THE USE OF NON-POTABLE WATER IN ROAD CONSTRUCTION

Report to the
Water Research Commission

by

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EXECUTIVE SUMMARY

Construction of a new bitumenised road typically requires upwards of 1 000 cubic metres (1 Mℓ) per day of fresh water, enough for 20 000 people at the Cape Town drought ration of 50 litres per person per day.

Only fresh water is normally permitted because of certain poor experiences in the past such as damage to the primed base course when even slightly brackish water was used and damage to the bituminous surfacing when salt water was used.

The water used cannot be recovered and is lost forever.

The causative damage factors have been identified and it is shown that, provided certain design and construction precautions are taken, even seawater can be used in all layers of most roads and other flexible pavements.

A short, practical users' guide is also provided.

Although the use of salt water cannot be considered in isolation from the inherent salinity of the pavement layer material itself, the scope of this project does not fully include this aspect and it is assumed that the materials initially comply with those of the conservative national salinity specifications for state roads.

Further projects are recommended using materials of greater inherent salinity, gold and copper mine waste rock containing sulphides with a potential to generate salts, acid mine, and effluent water.

The aims of the project were to conduct a critical state-of-the art review of southern African and some overseas experience of the use of non-potable water, to develop a practical guide and publish the results.

The methodology used was to carry out a limited literature survey, write up and review the author's experience and own practical experiments, produce a practical guide document and submit a paper to a local civil engineering journal or conference.

A critical state-of-the-art review of southern African and some overseas experience of the use of non-potable water in road construction is provided. However, the main focus of the project has been the writing-up and presentation of the results of the writer's unpublished practical, full-scale, long-term (over 30 years) road experiments and experience in southern Africa using seawater and other saline waters.

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This report has also drawn upon work by the author whilst in the employ of the then NITRR of the CSIR and for the Roads Authority of Namibia, for which permission has been granted.

The opinions expressed here are those of the author and not necessarily those of any of the above.

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LIST OF ABBREVIATIONS, ACRONYMS, DEFINITIONS AND SYMBOLS

AADE	Annual average daily E80 (per lane unless stated otherwise) over a 365-day year (TRH16: COLTO, 1991)
AADT	Annual average daily traffic in total vehicles per day in both directions over a 365-day year. (TRH16:1991), units usually omitted in text
AADTT	Average annual daily truck (i.e. heavy vehicle) traffic, units usually omitted in text
AAE	Average annual equivalent 80 kN standard axles (E80)
AASHO	American Association of State Highway Officials, now AASHTO
AASHTO	American Association of State Highway and Transportation Officials (formerly AASHO)
ADE	Average daily E80 in E80/lane/day over the axle load survey period (TRH16: 1991)
Asphalt	Bituminous concrete
ASTM	ASTM International, formerly American Society for Testing (and) Materials
Brackish	Slightly saline water with a salt content less than that of seawater [usually 3,5%]; sometimes defined as containing 1,5-3,0% (Monkhouse *et al.,* 1982)
BS	British Standard
Cape seal	See S4
Compaction	In South Africa always as a percentage of the MAASHO MDD reference density
C1 – C4	Cemented natural gravel of decreasing strength (see TRH 4 (COLTO, 1996) for abbreviated and COLTO (1998) for complete specifications and TRH 14 (NITRR, 1985a) and TRH 13 (NITRR, 1986a) for guidelines)
CBR	California bearing ratio. In South Africa unless noted otherwise always at 2,54 mm penetration under a 5,56 kg surcharge after four days soaking under a total surcharge of 5,56 kg; either after compaction at MAASHO OMC at the specified effort (see TMH 1-NITRR, 1986b), or a CBR value at a particular percentage of MAASHO MDD calculated from such test results; in other countries often substantially different from this. Units are in percent, usually omitted in text
Ch or ch	Section “chainage” in m from start of section
COLTO	Committee of Land Transport Officials (successor to CSRA)
CONCRETE	Portland cement concrete
COTO	Committee of Transport Officials (successor to COLTO)
CSRA	Committee of State Road Authorities
DCP	Dynamic cone penetrometer (TMH6:1984)
DoT	South African Department of Transport
DWAF	Department of Water Affairs and Forestry
DWS	Department of Water Affairs and Sanitation (successor to DWAF)
EC	Electrolytic conductivity of the passing 6,7 mm saturated soil paste (TMH1:1986) in S/m of soil or graded crushed rock, or of water in mS/m or S/m
ESA, E80	Equivalent standard axle (80 kN, single axle, dual wheel, assumed tyre contact stress 520 kPa) (TRH 4:1996)
Expt	Experiment
FN	Frank Netterberg
FWC, FMC	Field water (moisture) content in percent m/m

FWD	Falling weight deflectometer
GM	Grading modulus = (R2000 +R425 + R075) / 100 (Kleyn, 1955) = 300 – (P2 000 + P425 + P075) / 100 (i.e. essentially a coarseness index)
GVM	Gross vehicle mass in metric tons (1 000 kg)
GI	Group Index according to AASHTO M-145
G1-G3	Graded crushed stone of decreasing quality (see TRH4 (COLTO, 1996) for abbreviated and COLTO (1998) now COTO (2020) for complete **specifications** and TRH14 (NITRR, 1985a) for **guidelines**. This 'G' classification system came into use only after 1978 and in national specifications in 1998 and its use in this report for earlier work implies an estimated G classification with which it may not fully comply
G4-G10	Natural gravel or soil of decreasing quality (see TRH4 (COLTO, 1996) for abbreviated and COLTO (1998) for complete specifications and TRH14 (NITRR, 1985) for guidelines
hv	Heavy vehicle ("truck"). Vehicle with axle load > 4 000 kg, GVM > 7 000 kg and a carrying capacity greater than 3 000 kg (TRH4:1985; TRH16:1991; TRH4:1996), formerly > 2 700 kg (TRH4:1978)
hv/d	Heavy vehicles per day
Im	Thornthwaite's Moisture Index (Schulze, 1958, Emery, 1992) in cm, units usually omitted
LEF	Load equivalency factor in E80/heavy vehicle
LHS	Left-hand side
LL	Liquid limit in percent m/m, units usually omitted in text
LS, BLS	Bar linear shrinkage (in South Africa from the LL) in percent of initial wet length, units usually omitted in text
LTPP	Long-term pavement performance
LSM, SM	Linear shrinkage modulus or shrinkage modulus = LS x P425 (Netterberg, 1994)
LSP, SP	Linear shrinkage product or shrinkage product = LS x P075 (Netterberg, 1994)
MAASHO, MAASHTO	Modified **AASHO** (or, incorrectly, MAASHTO) compaction or compactive effort (2 413 kJ/m^3 in South Africa (DOT, 1970), i.e. lower than modern AASHTO T 180 (2 695 kJ/m^3), ASTM D 1557 (2 700 kJ/m^3) and BS 1377 heavy (2 671 kJ/m^3) efforts and equal to the original MAASHO effort
MDD	Maximum dry density in kg/m^3 (at MAASHO effort unless stated otherwise)
MESA, M E80	Million equivalent single axles
N	Weinert's (1980) climatic N-value, dimensionless
NIRR	National Institute for Road Research of the CSIR in Pretoria
NITRR	National Institute for Transport and Road Research (successor to NIRR)
n	Load equivalency exponent
n	Number of results
NP	Nonplastic (TMH1:NITRR, 1986b)
OWC, OMC	Optimum water (moisture) content for compaction, in percent m/m (usually at MAASHO effort in South Africa)
Pavement	Road, street, airport or other area covered with a bituminous (flexible pavement) or portland cement concrete (rigid) surfacing
PI	Plasticity index, in percent m/m, units usually omitted
PIM, PM	Plasticity index modulus or plasticity modulus = PI x P425 (Netterberg, 1994)
PL	Plastic limit, in percent m/m, units usually omitted

PSI	South African Present Serviceability Index (dimensionless) – see T
P425, etc.	Cumulative (i.e. total) percentage by mass **passing** the 425 µm sieve, etc., units usually omitted in text
R425, etc.	Cumulative (i.e. total) percentage by mass **retained** on the 425 µm sieve, etc., units usually omitted in text
ROC, RoC	Radius of curvature of deflection bowl in m
Salinity	As used here the TDS in mg/ℓ or % or the EC in mS/m or S/m
SANRAL, Sanral	South African Roads Agency Limited
SD, *s*	Sample standard deviation
Seal	Bituminous surface treatment (i.e. not asphalt), e.g. S1, S2, etc. (see TRH4 (COLTO, 1996) for abbreviated and COLTO (1998) for complete specifications and TRH3 (SANRAL 2020 for designs)
Soil	Loosely, any soil, gravel or uncrushed weathered rock used in road construction
Specifications	Always imply a minimum or maximum as appropriate, but loosely for example, compaction to 98% means compaction to a minimum of 98% MAASHO reference density
SV	Construction stake value (formerly chainage in chains of 100 English feet), now in km + m
S1	Single seal
S2	Double seal
S3	Sand seal
S4	Cape seal (13 or 19 mm S1 with one or two slurries (Sl), respectively
Sl	Slurry seal, grading not specified, S5,S6 or S7 if specified
SP	Slightly plastic (TMH1:NITRR, 1986b)
S5-S7	Slurry of increasing coarseness (see TRH 4 (COLTO, 1996) for abbreviated and COLTO (1998) for complete specifications)
TDS	Total dissolved or soluble solids (assumed salt) content in mg/ℓ or 1% m/v for water or % m/m for soils or graded crushed rock
TMH	Technical Methods for Highways series of documents
TRH	Technical Recommendations for Highways series of documents
Unsealed	Road or other area not covered with a seal (e.g. gravel road)
vpd	Vehicles per day, total, normally total of both directions
$\bar{x}$	Arithmetic mean

GLOSSARY

1. CLIMATE

As the likelihood of salt damage is very weather- and climate-dependent, particular attention is paid to these factors wherever possible.

The general climatic classification used is largely that of Köppen-Geiger's 1961 1:16M map 'Climate of the Earth' (Justus Perthes, Darnstadt) with a map of southern Africa (Schulze, 1947) and a later discussion by Trewartha and Horn (1980). In this system, for which worldwide data are available, the climate is abbreviated by three letters, the first giving a general description, the second an indication of the annual distribution of rainfall and the third the temperature. For example, the west coast of Southern Africa has a cold dry desert climate (BWk) with fog near the coast (BWk'n) grading eastwards into a hot dry steppe (BSh) climate with rainfall mostly in summer. Climatic statistics such as rainfall have been taken from Weather Bureau (1980, 1986) supplemented by Schulze (1997) and Kruger (2004, 2007).

For pavement design purposes the macroclimate is defined approximately according to Weinert's (1980) N-values as Dry (N >5), Moderate (N = 2-5) and wet (N <2) in TRH 4:1996 (Committee of Land Transport Officials (COLTO) 1998) taken from larger unpublished maps. However, Thornthwaite's Moisture Index (Im) (Schulze, 1958) as read from Emery (1992) and unpublished larger maps is also used.

2. PAVEMENT DESIGN

The structural design of flexible pavements for interurban and rural roads used in South Africa and essentially also in Botswana and Namibia is TRH4:1998, rehabilitation investigation and design TRH12:1997 and visual assessment TMH9:1992, which also define the terminology used in more detail than in the list of abbreviations, etc. provided, as well as its interpretation.

3. TEST METHODS

The laboratory and field test methods used were those of TMH1:1979 or 1986 or, prior to 1979 their earlier equivalents (DoT, 1970). The routine TMH1 A2IT electrolytic conductivity (EC) test was used from 1979 and the equivalent NITRR/CSIR research EC method CA21-74 (1980) from which it was derived was used for the earlier work. The CA21 soil paste method was used for all pH determinations, and not the TMH A20 suspension method, which are not equivalent. The dynamic cone penetrometer (DCP) test method for the in-situ strength was that of TMH6:1984.

4. SPECIFICATIONS AND MATERIALS

Although each road authority may have their own version, the general **specifications** for state roads from 1987 were those of the Committee of State Road Authorities (CSRA, 1987), from 1998 until February 2021 those of COLTO:1998, superseded by the draft COTO:Oct. 2020 of the Committee of Transport Officials (COTO) for all new work from 01 March 2021. Similar specifications are also used in Botswana and Namibia. The main **guideline** documents for materials are TRH14:1985 and TRH13:1986 of the National Institute for Transport and Road Research (NITRR) of the CSIR and for seals TRH3:2020 of SANRAL. All of the above except TRH3 included requirements for the soluble salt content and pH of graded crushed stone, gravel and soil and requirements for water.

CHAPTER 1: BACKGROUND

1.1 INTRODUCTION

South Africa is running out of water and has a projected 17% water deficit by 2030 unless immediate, bold action is taken (Department of Water and Sanitation (DWS) 2018).

Construction of a new, bitumenised flexible pavement with a granular base such as a road, street or airport requires at least about 500-1 000 m^3 (1 Mℓ) per day of water for the compaction of the soil, gravel and crushed rock. Allowing for the low moisture content and high absorption of many materials and the high evaporation and other losses in hot dry weather in a dry climate, for a road this can escalate to 2 Mℓ per day or more – up to 10 Mℓ per day piped from the Orange River was used for the new Springbok-Pofadder Road in 1981-1983.

As each megalitre per day is equivalent to the Cape Town daily domestic drought ration of 50 litres per person for 20 000 people, the use of non-potable water could render a significant saving of fresh water in times of drought and may make the difference between building the road at all.

However, even brackish water can lead to surface disintegration of the primed base course during construction (Figure 1.1) and the use of hypersaline or sea water even in the subbase also to the blistering and cracking of bituminous surfacing (Netterberg, 2013; Netterberg and Bennet, 2004) (Figures 1.2-1.4).

Figure 1.1: Typical mild surface disintegration of primed base course due to crystallisation of salt from brackish compaction water in the base course.

Figure 1.2: Very severe blistering and cracking of triple seal on old Walvis Bay airport due to crystallisation of halite (NaCl) in the upper base due to use of salt water in subbase (Netterberg, 2015)

Figure 1.3: Closer view of very severe blister 30 mm high and 400 mm wide on old Walvis Bay airport.

Figure 1.4: On opening the blister much white halite (NaCl) was evident and the full depth of the G3 crushed stone base course could be excavated by hand.

Such damage may require expensive remedial measures which may also lead to construction delays, disputes, contractual claims, and potential court cases.

Owing to such negative experience in the then Cape Province in the 1950s only potable (fresh) water preferably of drinking quality is normally permitted, especially in the base course.
This water is not recoverable and is lost forever.

However, research by the author and others over the last 40 years comprising personal investigations, examination of case histories and practical road experiments has shown that, with certain precautions, brackish and even seawater can be used in all layers including the slurry of a Cape seal without detriment to the road during construction or its usual design life of 20 years.

Although publications are available on the local salt damage problem (e.g. Netterberg, 1979; Obika, 2001; Netterberg and Bennet, 2004), no local publication appears to be available on the specific use of seawater except one case (Spottiswoode and Graham, 1982a, b) where preliminary results of the author's research (Netterberg, 1983) were applied with fair early success, one by him on a slurry only (Netterberg, 2004) with complete long-term success, and a few notes (Netterberg, 2007, 2011, 2013, 2020) to raise awareness and in his unpublished reports.

The research has aimed to understand the reasons for and the mechanisms causing the damage and by a study of case histories and the construction and long-term monitoring of purpose-built road experiments to derive practical guidelines for the successful use of even seawater, all of which have been achieved.

The innovation is the use of non-potable water where only scarce and expensive potable water is currently permitted.

This work has been particularly rigorous in that the factors apparently involved in a number of previous cases of both successful and unsuccessful use of non-potable water were identified and hypotheses developed therefrom. These hypotheses were then tested by means of full-scale road experiments involving several different types of material, climate, construction constraints, pavement layers, and primers, and were designed and monitored by the writer personally both during construction and for more than the usual road design life of 20 years.

The envisaged outcome is that, with the precautions to be derived, the road authorities will permit the use of non-potable and even seawater instead of scarce and expensive potable water.

Society will benefit as more potable water will be available, the bituminising of roads and streets will not be delayed due to lack of potable water, and their cost will be reduced.

As most of the compaction water evaporates or remains within the road or other pavement prism the use of brackish or even seawater is not expected to lead to any health, safety or environmental concerns, but waste waters would have to conform to the relevant treated standard.
The term 'pavement' is used in this report in its engineering sense to mean any facility such as a road, street, airport, sidewalk, etc. covered with (in this case) a flexible bituminous surfacing.
The term 'roads' as used in this report is intended to include streets but with due consideration of their different environments.

Although the use of non-potable water cannot be considered in isolation from the inherent salinity of the pavement layer material itself, the scope of this project does not adequately include this aspect and it is assumed that the materials (i.e. graded crushed stone, gravel or soil) initially comply approximately with the salinity specifications for state roads (COLTO:1998, COTO:2020), i.e. essentially a maximum electrolytic conductivity (EC) of 0,15 S/m and a minimum pH of 6,0.

1.2 PROJECT AIMS

The aims of the project were to:

1. Conduct a critical state-of-the-art review of Southern African and some overseas experience of the use of non-potable water in road construction
2. Develop a practical guide and publish a paper on the use of non-potable water in road construction

1.3 SCOPE AND LIMITATIONS

The scope of the project has been wide in that it has included climates ranging from moist subtropical to arid, weather from misty coastal to dry, materials from graded crushed rock of base course quality to finely graded calcrete of selected subgrade quality, and a range of inherent material and water salinities from very low (as controls) up to that of seawater and more.

As damage can occur both during and after construction his practical road experiments were monitored personally by the author for at least the full design life of the road, usually 20 years.

The sources of information were mostly unpublished case histories gathered by the author and his own laboratory and site experiments carried out over the last 50 years.

Some 200 cases of salt damage to bitumenised pavements of all types are known to the author to have occurred in southern Africa since about 1957 to the present under climatic conditions ranging from arid receiving a normal annual rainfall of less than about 50 mm, up to about 800 mm, i.e. more than half of South Africa alone.

The limitations of the work and recommendations described are that they are mainly applicable to the use of salt water used for the compaction of base course material of inherently low to moderate salinity. Although one case is described of the use of seawater for a calcrete base of high salinity, the extension of the work to the use of material of high inherent salinity (such as many calcretes) or potential salinity (such as sulphidic gold and copper mine waste rocks) would require a larger project and is beyond the scope of this one.

It is therefore recommended that further projects should be undertaken for the use of materials of higher inherent salinity, those containing sulphides such as gold and copper mine waste rocks which can develop both high acidity and salinity, and mine and effluent waters. Such work should also consider the potential environmental and safety implications of using such materials, which have actually been in use for some time.

CHAPTER 2: LOCAL EXPERIENCES AND LITERATURE SURVEY ON THE USE OF NON POTABLE WATER IN ROAD CONSTRUCTION

2.1 INTRODUCTION

Only sufficient local experience and literature are summarised here in order to illustrate the variability, severity and extent of the problem of using non-potable water in road construction.

The highly variable experiences reported both on a local and global basis showed the necessity for the construction of specially designed, controlled, full-scale, long-term road experiments under local conditions of climate, materials and pavement design.

Two such experiments using seawater were therefore constructed in the Cape Town area in 1975-1976, one at Lambert's Bay in 1975 and one at Lüderitz in 1976.

It is the main purpose of this project to review these experiments and experiences, to report the results of the controlled road experiments – in particular those in the Western Cape – to identify the factors leading to success or damage, and to derive practical, quantitative measures for the successful use of non-potable – particularly seawater – in road and other pavement construction.

2.2 WATER QUALITY SPECIFICATIONS FOR ROAD CONSTRUCTION

2.2.1 South African National Specifications for state roads

2.2.1.1 Water quality specifications

The South African national specifications for state roads (CSRA:1987, COLTO:1998) only included specifications for mixing water for concrete and a proviso that "where the salinity of the water used for compaction purposes is so high as to cause an increase in the salinity of the material, the engineer shall be entitled to determine the soluble salinity from samples taken from any section of the compacted layer within 24 hours, and also before the prime coat is applied" (COLTO:1998). This was made more specific in the current draft COTO:Oct. 2020 national specifications in which the author assisted:

"Water to be used for the construction of the earthworks and road pavement layers, including for bituminous stabilisation shall comply with the requirements set in Table A4.1.5-19" (Table 2.1).

"Purified wastewater, also known as effluent, and water from other sources that may contain visible quantities of physical and aesthetical, chemical or organic determinants expected to have a detrimental effect on the road layers, can be considered for the construction of the earthworks and the road pavement layers, provided that it complies with the requirements in the above table. In addition, for chemical stabilised layers, the stabilized material shall comply with the strength requirements for UCS and ITS at extended 28-day curing periods and the durability (WDD) requirements specified in Table A4.4.5-2 [Not shown]. Water containing raw sewage shall not be used anywhere in the construction

Table 2.1: Construction water for earthworks and pavement layers (COTO Oct. 2020)

Purpose	Electric Conductivity (EC) at 25°C (maximum)	Total dissolved solids (TDS) (maximum)	pH range at 25°C	Sulphate as SO_4 (maximum)
1. Crushed stone base layer compaction and slush-compaction	170 mS/m	1 200 mg/ℓ	5,0-9,7	-
2. Chemical stabilisation compaction and curing	170 mS/m	1 200 mg/ℓ	5,0-9,7	450 mg/ℓ
3. Bituminous stabilisation	170 mS/m	1 200 mg/ℓ	5,0-9,7	-
4. Other layers and materials	370 mS/m	2 400 mg/ℓ	4,0-10,0	-

Note:

(1) Siemens per metre (S/m) is the standard SI unit. The salinity of water can also be expressed in milligrams per litre (mg/l) or percentage (%). The relationship between the units are: 1 000 mg/l = 0,1% = 150 mS/m

"The use of brackish water and seawater can be considered for the compaction, provided that the saline water shall be used instead of the prescribed distilled water in the pH and EC tests and that the requirements for soluble salts in Table A4.1.5-18 shall not be exceeded, except with recommendations by an experienced road materials specialist and proved in a trial experimental section. Brackish water and seawater shall not be used in the curing of any stabilized layer".

"Turbid water can be used for compaction without further testing in addition to the test specified in Table A4.1.5-19 [not shown] except for crushed stone bases where strict requirements for the PI and the P0,075 apply. In this instance a sample shall be prepared of the crushed stone material mixed with the required quantity of turbid water to reach optimum moisture content. After drying out of the material the PI and P0,075 shall be tested and evaluated for compliance with the specified limits in Table A4.1.5-5" [not shown].

"Effluent, brackish water and seawater shall not be used for bituminous stabilisation or for diluting a bitumen emulsion, unless the use of the water has been approved in writing by the supplier".

2.2.1.2 Comment

Some of the test methods specified in the **draft** COTO:2020 are inappropriate or not listed and until the **new** SANS methods are approved the correct ones are those in the COLTO:1998 specifications and, for cement and lime stabilized materials, the TRH13:1986 guidelines.

The limits of EC and TDS in Table 2.1 are within those for the previous SANS 241:2006 Class II drinking water of 1 000-2 400 mg/l and 150-370 mS/m, although the pH limits are more restrictive except for other layers and materials which are the same at 4,0-10,0. Specification of both EC and TDS is unnecessary; EC alone is sufficient, cheaper and more suitable for construction control.

The use of EC at 25°C with the usual conversion factor of 6,5 for the estimation of TDS is an acceptable alternative to measuring the TDS (DWAF, 1996a).

2.2.2 The Botswana Guide

2.2.2.1 Water quality specifications

Botswana is also a water-stressed country and "the prevention of soluble salt damage to roads and runways plays a vital role in reducing the cost of both construction maintenance of roads in Botswana" (Mr Andrew Nkaro in the Foreword to the Botswana Roads Department Guideline No. 6 (Obika, 2001) on salt damage to roads and runways).

At that time at least ten cases of damage to roads and airport runways attributed to salt were known in Botswana and this had led to the construction of experimental sections of saline materials alongside the Nata-Maun road in Botswana (Woodbridge et al., 1994). Although primarily concerned with the salinity of the construction material, much of the groundwater in Botswana is saline and this was also considered.

For example, after drilling 86 boreholes to an average depth of 180 m for the 600 km Trans-Kalahari road most sources found were brackish or saline (Pinard *et al.,* 1999).

The guide (Obika, 2001) includes a comprehensive risk evaluation protocol incorporating consideration of the salinity of the pavement material, subgrade and compaction water, the climate, and construction time delays between priming and surfacing, as well as the type of prime and surfacing.

A risk of salt damage is considered if the salinity of the material exceeds 0,2% and/or the water 2 000 mg/ℓ and a chart is provided in which the water is classified as fresh (0-0,5%), brackish (0,5-1,0) % and saline (over 1,0%) against the soluble salt content of the pavement or subgrade.

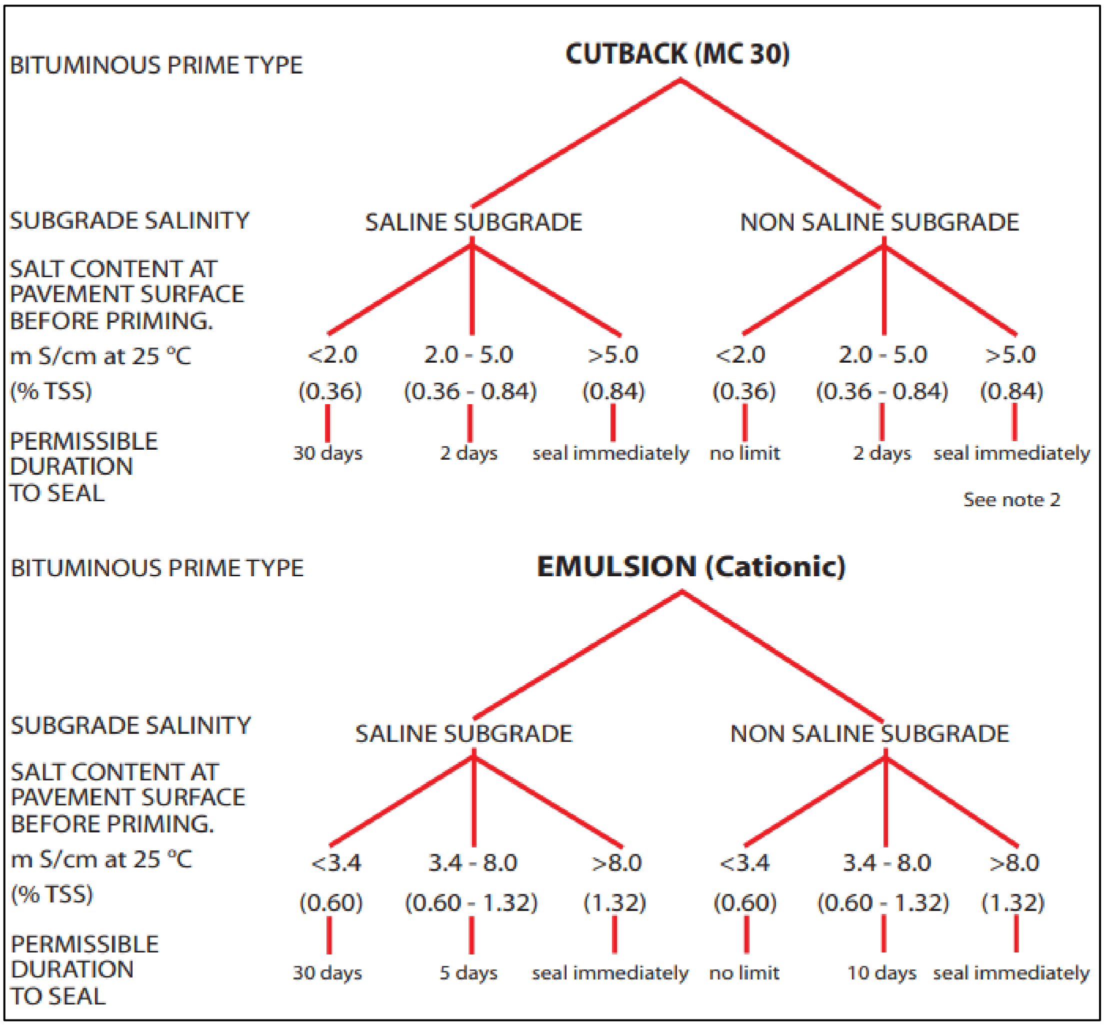
A risk value derived from this chart is then added to one for the climate to give a combined risk value for the project.

A decision tree according to the type of prime, the salinity of the subgrade and that of the upper 50 mm of the base course to arrive at the permissible time duration between mixing and sealing is then used (Figure 2.1). A table showing the estimated soluble salt content tolerable by different surfacings is also provided.

2.2.2.2 Comment

On the whole this is an excellent, well-illustrated and user-friendly guide, which is unfortunately marred by confusion over the EC and TSS methods to be used for the decision tree, which remains to be cleared up. The time constraints and the guide to the soluble salt contents tolerable by different types of surfacing must therefore be used with caution.

The extent to which this guide has been used is uncertain. Although it was specified for the Trans-Kalahari road the salinity of most of the water (and presumably the materials) found resulted in all of the contractors opting where possible to pump water long distances and to store it in temporary reservoirs rather than complying with the stringent time constraints (Pinard *et al.,* 1999). In spite of this some salt damage did occur.



Notes:

[1] A safety factor of 2 was applied

[2] A conversion equation for total soluble (salt TSS) content is provided

Figure 2.1: Permissible intervals between prime and surfacing in relation to subgrade salinity and pavement surface salinity (Obika, 2001).

2.2.3 The Namibia Guide

2.2.3.1 Water quality specifications

A comprehensive guide to the use of saline materials and waters in bitumenised road construction in Namibia written by the author has been included in the materials manual of the Roads Authority (2014).

This guide will not be repeated here but only those aspects related to the use of salt water for the compaction of inherently only slightly saline materials highlighted.

The inherent salinity of G1, G2 or G3 graded crushed rocks, gravels and soils varies widely from about 0,01 to several percent (equivalent to an EC of about 0,02 to 3 S/m) and under an adverse combination of circumstances even as little as 0,2% salt can cause surface disintegration of a primed base course. The soil binder added to a G3 material is also often a source of salt.

The salinity of water varies from about 1 mg/ℓ (0,15 mS/m) for clean rainwater through up to about 1 000 mg/ℓ (0,1% and 150 mS/m) for drinking water to 3,5% (5,3 S/m) for seawater, or more. Under an adverse combination of circumstances compaction and slushing water with a salinity of only 0,2% has caused surface disintegration of a primed G3 base course.

The use of other than potable water must always be considered together with the salinity of the material by using the proposed water instead of distilled water in the EC and pH tests and/or by adding it to the full grading of the material, air-drying it, and carrying out the standard EC and pH tests with distilled water.

In general a compaction water with a maximum EC of about 0,5 S/m (0,33% TDS) is recommended for general use without any special precautions except for thorough removal of all excess fines from the base course after compaction and not leaving the primed base to stand for several months.

The accelerated construction method is recommended when water with a higher salinity and/or material with an inherent salinity of more than about 0,15 S/m must be used.

In principle this involves covering each layer with the next – even just by dumping and spreading to act as a mulch – to minimise upward migration during construction and priming and sealing the base with an impermeable surfacing as soon as practicable after compaction.

The EC of a G2-G6 material of inherently low salinity compacted with seawater is likely to be about 0,5 S/m.

The maximum delay recommended between completion of a G4 or G5 subbase compacted with seawater with an EC of about 0,50 S/m and covering it with the base is three weeks, but two weeks if it exceeds 0,50 S/m.

The maximum delay recommended between completion of a base compacted with seawater with an EC of about 0,50 S/m and priming is 14 days for a G2 and G3 and 7 days for a G4 and G5.

The maximum delay recommended between priming and surfacing for a base compacted with seawater with an EC of about 0,60 S/m is 14 days for a G2 and G3 and 24 hours for a G4 or G5.

Further details and a comprehensive list of precautionary measures – mostly just emphasising the necessity of good workmanship – are provided.

2.2.3.2 Comment

This guide is intended both for compaction with sea or other salt water and for the use of material with an inherent salinity up to about ten times that normally permitted.

As methods of determining the salinity of both waters and materials – especially the latter – are very method-dependent it is important to only use the test methods used to derive the specifications and guidelines, i.e. essentially the paste EC and pH methods prescribed by COLTO:1998 and TRH13:1986.

The paste pH test should initially also be carried out in addition to the EC as it provides a guide to the presence of unusual salts or acidity which requires further consideration; often only the EC need then be used.

Whilst the limit of 0,5 S/m (i.e. 500 mS/m) for compaction water is more than the 170 and 370 mS/m specified by COTO:2020 the latter do not require the use of any special precautions.

It is emphasised that the content of soluble salt is only one factor involved and the allowable time delays are conservative guidelines rather than specifications and should be tempered with experience gained during the particular project.

A quick guide to the method developed is provided in Chapter 5.

2.3 USE OF SEAWATER IN ROAD CONSTRUCTION

2.3.1 Composition

Two arbitrarily defined quantities, the **salinity** and the **chlorinity** are usually used in discussing the composition of seawater. The salinity is slightly less than the total mass of dissolved solids per kilogram and is usually reported on a mass/mass (m/m) basis at 480°C in parts per thousand (per mille, ‰) and that of other waters on a mass/volume (m/v) basis in mg/l and the test methods used also differ. As the authors quoted did not always provide such details, for ease of comparison the figures quoted for salt water are mostly total dissolved solids (TDS) in percent (%) presumably usually as m/v, as mostly provided by the authors. However, this is sufficiently accurate for pavement engineering purposes.

The chlorinity is determined by precipitation of the halides with a silver salt and will not be used in this report.

The average near-surface, open ocean seawater has a salinity of about 35‰ (i.e. 3,5%) m/m with a range of only 32 to 37‰ , but may range up to about 38‰ in the Mediterranean and 41‰ in the Red Sea and the Persian Gulf and down to less than 32‰ where fresh water from rivers, melting ice or underground water flows into the sea such as the Baltic, which can be as low as 4-8‰ (Harvey, 1980; Mason and Moore, 1982). Salinities much higher than 35‰ may be encountered in landlocked estuaries, rock pools, tidal basins and coastal salt pans due to concentration by evaporation. However, at least in other than such cases the relative proportions of the major ions remain almost constant (Mason and Moore, 1982; Riley and Skirrow, 1965).

The determination of only one component such as the salinity – most conveniently by simple electrolytic conductivity measurements – is therefore usually sufficient for practical civil engineering purposes. For this purpose the usual average factor for most natural waters (e.g. DWAF, 1996a) can be used:

TDS =	6,5 EC	[2.1]
	where TDS = total dissolved solids in mg/ℓ	
	EC = conductivity at 25°C in mS/m	
=	0,65 EC	[2.2]
	where TDS = total dissolved solids in %	
	EC = conductivity at 25°C in S/m	

This factor varies between 5,5 and 7,5 and for more accurate work should be determined for the specific water.

The same factor of 0,65EC in S/m can be used for seawater within the usual range of 3 and 4% to estimate the TDS on a m/m basis. For more accurate work on seawater or other salt waters of similar composition Figure 5 of NITTR Test Method CA21-74 (1980) or factors derived from it can be used.

The accuracy of the simple EC method is probably about ±10% of the TDS in the case of natural waters of unknown composition and much better in the case of seawater. This is adequate for most road works and the difference between TDS expressed as m/m and m/v is also of little concern.

Other convenient methods of sufficient accuracy for waters of high salinity such as seawater are the determination of relative density (specific gravity) using a density bottle or an hydrometer using the graph in Harvey (1960) or the table in Riley in Skirrow, 1975, **3**) or Weast (1977).

Areometers (hydrometers) graduated in degrees heavy Baumé correspond closely to TDS m/m and are available in ranges of 0,1, 0,5 and 0,40°. The 0-5° range should be used for seawater and the 0-40° range for hypersaline waters such as are used for the selection of salt water in the construction of “salt” roads.

For a quick estimate a pocket refractometer of the type used by winemakers for testing the sugar content of grapes can be used.

A convenient table listing the electrolytic conductivity of seawater for salinities between 5 and 60‰ and the density and refractive index for up to 150‰ is that of Weast (1977, p. D249).

The major chemical composition of the average near-surface open ocean water is shown in Table 2.2. Although the major constituents occur as ions it is convenient to regard it as a solution of the anhydrous salts shown. Thus the 3,5% sea salt is largely composed of NaCl, with minor $MgCl_2$ and $MgSO_4$ and only about 0,1% $CaSO_4$ and K_2SO_4. Unless polluted or drawn from the surface the content of organic and suspended matter is usually low. The pH is generally between 8,0 and 8,35 and is near calcite saturation (Walther, 2005).

Table 2.2 also shows the salts precipitated during evaporation, such as would occur in a pavement layer compacted with seawater on standing waiting to be covered with the next layer, prime or surfacing.

In this connection a prime coat applied at the usual rate of about 0,7 ℓ/m^2 cannot be relied upon to prevent a primed base from drying out as it becomes permeable to air and water vapour as soon as it dries.

Neglecting other factors such as nuclei, mineral and solute interactions, and the minerals inherent in the layer, the minerals will precipitate out more or less in order of their increasing solubility, the least soluble first: firstly some form of iron oxide, then $CaCO_3$ probably as calcite and/or dolomite $CaMg(CO_3)$, followed by gypsum ($CaSO_4{\bullet}2H_2O$) and/or anhydrite ($CaSO_4$), which normally hydrates to gypsum.

At this stage 90% of the seawater has evaporated but only 3% of the salts have precipitated. These will do no harm and may even act as weak cements.

Only then does halite (NaCl), which comprises about 74% of the salts present – and which is the chief potentially disruptive agent present – crystallise, together with minor anhydrite, polyhalite ($K_2Ca_2Mg(SO_4)_4{\bullet}2H_20$) or glauberite ($Na_2Ca(SO_4)_2$).

Only very small amounts of these latter three salts can form owing to the very small amounts of calcium and potassium in the seawater.

At this stage about 98,5% of the seawater has evaporated and 78% of the salts have crystallised out of solution, including all the halite.

Table 2.2: Composition of average open ocean seawater and salts precipitated during evaporation

Major constituents at 35‰ salinity				Salts precipitated during evaporation [3]				
[1]		[2]		Stage	Approximate volume of seawater	Approximate density of seawater	Dominant minerals precipitated	Total salts precipitated
Ion	% m/m	Salt	% m/m		%			%m/m
Cl	1,93			1	100-20	1,026-1,126	Iron oxide, $CaCO_3$ and/or dolomite	1
Na	1,08	NaCl	2,72	2	20-10	1,126-1,21	Gypsum and/or anhydrite	3
SO_4	0,27	$MgSO_4$	0,17	3	< 10	>1,21	Halite, minor anhydrite	56
Mg	0,13	$MgCl_2$	0,38	4	< 5-1,5	>1,23-1,31	Halite, minor polyhalite or glauberite	18
Ca	0,04	$CaSO_4$	0,13				15°C < > 55°C	
K	0,04	K_2SO_4	0,09		< 1,5	>1,31	Epsomite Blodite Loweite	
HCO_3	0,01	$CaCO_3$	0,01				Schoenite Kieserite Langbeinite	
Br	0,01	$MgBr_2$	0,01				Kainite Kainite Kainite	
Total	3,51		3,51				Epsomite Kieserite Kieserite	
etc.	<0,01	etc.	<0,01					
							Carnallite Carnallite Carnallite	
							Hexahydrite Kieserite Kieserite	
							Carnallite Carnallite Carnallite	
							Bischofite Bischofite Bischofite	22

Notes:

[1] Riley in Riley and Skirrow, 1975, **3**). HCO_3 actually carbonate alkalinity expressed as HCO_3^- (Stumm and Morgan, 1981)

[2] Hypothetical combination after Lotze in Borchert (Riley and Skirrow, 1965)

[3] Compiled and simplified from Borchert in Riley and Skirrow (1965), Mason and Moore (1982) and Stewart (1963)

[4] Some other properties of seawater are: pH 7,8-8,3 (Skirrow in Riley and Skirrow, 1965); and at a salinity of 35,00‰ and 25°C: EC 0,530 S/m (U.S. Navy 1951, Riley in Riley and Skirrow, 1975, **3**, NITRR/CSIR Test Method CA71-74 (1980); density 1,023, chlorinity 19,37‰, osmotic pressure 2,6 MPa, vapour pressure lowered to 69% of that of pure water, absolute refraction index 1,338 (Riley in Riley and Skirrow, 1975, **3**); some summarised vs salinity in Weast (1977 1, p. D249)

With the exception of schoenite (a form of picromerite $K_2Mg(SO_4)_2 \bullet 6H_20$)), langbeinite ($K_2Mg_2(SO_4)_3$ and carnallite ($KMgCl_3 \bullet 6H_20$) – all of which can also only occur in very small amounts – and bischofite ($MgCl_2 \bullet 6H_20$) all the remaining salts are various hydrated forms of $MgSO_4$ or Na,$MgSO_4$ and depend upon the temperature.

Although all forms of soluble sulphates are potentially damaging to portland cement and lime, only halite and magnesium and sodium sulphates have been implicated in pavement salt damage due to crystallisation.

Whilst magnesium and, hypothetically, $MgCl_2$ is a significant constituent of seawater, most of the Mg will have crystallised out as sulphates leaving little to form bischofite. Bischofite is so highly hygroscopic, soluble and deliquescent that it rarely occurs in Nature and would hold water and remain in solution in a pavement layer.

2.3.2 Comment

In practice the dominant salts of concern in pavement construction are likely to be NaCl and some kind of $MgSO_4$ hydrate and most of them can be kept safely in solution by ensuring that the volume of seawater present does not fall below about 5-10% of that added.

The amount of water added to a pavement material for compaction is equal to the optimum water content (OWC or OMC) – which depends upon the material but is usually between 5 and 10% by mass – plus a few percent to allow for slushing and evaporation.

The addition of say 10% seawater would add 0,35% sea salt – mostly NaCl – to the layer and to whatever salt might already be there. This is substantially more than the lower limit of 0,1-0,2% NaCl found to cause surface disintegration of primed gravel base courses on drying out, although not all cases of surface disintegration or blistering are due to salts (Netterberg, 1994).

The presence of a concentrated solution of sea salt in say 2% of water in a pavement layer would actually be beneficial if the extremely high solute suction also adds to the shear strength of an unsaturated soil in addition to the matric suction. (Whether or not it does appears to be unknown.) On the other hand, the high sodium content may increase the Atterberg limits of any clay minerals present, especially smectite.

Whilst it is always best to use the proposed source of seawater for laboratory testing of its effect on engineering properties an artificial seawater adequate for most civil engineering purposes can easily be made up (e.g. ASTM:D1141 2013) with the omission of the unnecessary heavy metals.

Case studies of the local use of seawater are presented in Chapter 4.

2.4 USE OF OTHER NATURALLY SALINE WATERS IN ROAD CONSTRUCTION

2.4.1 Composition

The waters of saline lakes are of five types: Salt, alkali, bitter, nitrate and borate, the dominant salts being NaCl, Na_2CO_3, Na_2SO_4, $NaNO_3$ and $Na_2B_4O_2$, respectively (Bateman, 1950). Salt lake water approximates that of seawater, although the salinity may be higher.

The composition of river water in its upper reaches depends largely on geology and climate, and the TDS of world mean river water is only 110 mg/l including pollution, with African rivers at 61 mg/l and the dissolved species about half bicarbonate, with far lesser Ca, SO_4, $SiO_{2,}$(aq.), Cl, Na, Mg and K (Berner and Berner, 1996 in Walther, 2005).

However, the salinity of rivers increases with decreasing flow, increasing aridity, saline irrigation returns and pollution, and in landlocked and even tidal estuaries it may approach or possibly exceed that of seawater.

Two cases were investigated by the author in South Africa in which the use of estuarine water resulted in the surface disintegration of the primed base course, and a river and dam in Namaqualand both had an EC of about 0,5 S/m.

The species of salts present in groundwater also depends largely on geology and climate and the pore water in deep sedimentary rocks is often highly saline (Walther, 2005). Highly mineralised chloride-sulphate water and to a lesser extent alkaline sodium carbonate waters are characteristic of the South African semiarid and arid zones (Bond, 1940). NaCl is dominant in chloride-sulphate waters with sulphate being usually less than 10% of the TDS. $NaHCO_3$, Na_2CO_3 and SiO_2 are the most important constituents of the sodium carbonate waters and chlorides and sulphates are low.

The salinity of groundwater also varies with time and the rate and depth of abstraction and should be monitored regularly during construction.

In one such case investigated by the author the salinity of the borehole water exceeded that of seawater and led to extensive surface disintegration of the lime-stabilised calcrete base course when it had to be used as both compaction and curing water.

In short, the deleterious salts likely to be present in construction waters are NaCl, Na_2SO_4 and $MgSO_4$ and less often $NaHCO_2$ and/or Na_2CO_3 (Netterberg, 1970).

The salts precipitated on evaporation obviously depend upon the type of water and the extent to which it is evaporated: chloride-sulphate waters will precipitate mostly NaCl and possibly $MgSO_4$ and so on. In practice, as in the case of seawater, NaCl in the form of halite will usually be the dominant disruptive salt.

High concentrations of chlorides in excess of about 200 mg/ℓ are likely to be highly corrosive and a comprehensive guide with respect to both potential corrosion and scaling according to the composition of the water and the equipment is available (DWAF, 1996a).

A useful quick field test for salinity is taste: water has a noticeably salty taste above a TDS of about 450 mg/ℓ (70 mS/m), is marked above 1 000 mg/ℓ (150 mS/m) and would not normally be used for drinking water (DWAF, 1996b). The South African upper limit for drinking water is 1 200 mg/ℓ (170 mS/m) (SANS 241-1:2011).

2.4.2 Case studies on the use of other naturally saline waters in road construction

2.4.2.1 Walvis Bay Airport: A case study in an arid coastal area

The old Walvis Bay (Rooikop) airport was constructed using salt water said to have been more saline than seawater for the subbase and lower layers but fresh water was used to compact the G3 crushed stone base with a saline, gypseous binder, and surfaced with a triple seal without priming the base in 1962 (Netterberg, 2015).

Blistering, cracking and salt staining of the surfacing were observed within one year of construction, after a few years the entire runway was affected (Figure 1.2) and became so rough that it had to be closed to fast jet aircraft (Figure 1.3).

The blisters were filled with mostly halite salt and the base had ECs of up to 4,0 S/m and could be excavated with bare hands (Figure 1.4).

The salt was apparently mostly derived from the salt water used below the base, although all layers were also saline to some extent.

The airport was kept open for other aircraft simply by periodic rolling with a towed steel-wheel roller until rehabilitation and upgrading with a 70 mm-thick overlay of asphalt after 16 years in 1978.

Although this was 98% successful it was resealed with a bitumen-rubber single seal in 1985, which apparently performed well.

This case showed that salt can rise from a subbase compacted with salt water to severely damage even a triple seal and to slightly damage a subsequent 70 mm-thick asphalt overlay.

Although the high ECs were ascribed to halite and the crystallisation of this salt was certainly the cause of the damage, some gypsum was found in at least one blister. As the solubility of gypsum is increased by the presence of large amounts of NaCl the contribution of this normally only very slightly soluble salt cannot be entirely neglected.

Whilst salt damage to surfacings due to sulphate salts in bases made from crushed copper and gold mine waste rocks was known (Weinert and Clauss, 1967) this was the first case of damage to a surfacing due to NaCl in southern Africa and at that time the problem was poorly understood.

2.4.2.2 Sua Pan Airport: A case study in a semiarid inland area

Salt water with a TDS content of 4-15% and a pH of up to 10 containing mostly NaCl with much lesser amounts of $NaHCO_3$, Na_2CO_3 and sulphate from trenches next to the pan was used for compaction of the earthworks and subbase of this airport in1989 (Netterberg and Bennet, 2004).

In spite of potable water having been used in the base course, within one month of completion and no damage having occurred to the primed calcrete base, the Cape seal surfacing started to exhibit damage in the form of halite-filled blisters with associated cracking. Within six months most of the runway was affected (Figure 2.2) and after six years the entire runway was affected and those parts of the surfacing subjected to turning movements had completed disintegrated.

The EC of the base course under the five blisters tested was 1,4-3,2 S/m (averaging 2,0 S/m, equivalent to about 2,2% NaCl), with pHs of 8,2-9,8 and even under intact, uncracked surfacing 0,5-2,0 S/m, averaging 1,4 S/m, equivalent to about 1,4% NaCl.

Whilst both NaCl and trona ($Na(CO_3)(HCO_3)\bullet 2H_2O$) were initially reported to be the cause of the damage (Obika, 2001), in this more comprehensive investigation only halite was found.

Although the precautions specified included accelerated construction and a salt-resistant surfacing (a 13 mm Cape seal with one slurry) neither of these were achieved due to unavoidable construction delays and an inferior seal.

Figure 2.2: Severe blistering of 13 mm Cape seal on Sua Pan airport due to crystallisation of halite in upper base from salt compaction water used in subbase and lower layers (Netterberg and Bennet, 2004).

Due to shortage of funds for rehabilitation the runway was successfully kept in service for over seven years by periodically rolling the blisters flat, brooming off any loose material and patching any scabbed areas. A reseal with another 13 mm Cape seal was only partially successful and the worst section was replaced with interlocking block paving.

This case showed that restriction of the saline compaction water to the subbase probably prevented salt damage to the primed base on the runway during the 17 days between priming and tacking and chipping during which no rain fell, but did not prevent damage to the surfacing.

The three to four weeks delay between completing the base course and tacking and chipping probably permitted significant upward movement of salt into the base course to take place.

The five weeks taken to complete the seal during which some rain fell probably allowed salt migration into the upper base to take place and contributed to the poor performance of the seal.

This case also showed the difficulty of rehabilitating salt damage by resealing, even with a Cape seal.

2.5 USE OF TREATED EFFLUENT WATER (WASTEWATER) IN ROAD CONSTRUCTION

2.5.1 Introduction

Although wastewaters are of many types only treated effluent water (TE) will be discussed in this section. Mining water is discussed separately in section 2.6.

The background to and regulation of wastewater in South Africa has been well dealt with by Wall (2018) and will not be discussed further.

2.5.2 Wastewater quality specifications for treated effluent water

Effluent from wastewater treatment plants is required to comply with either the General or the Special Standard for effluent as set by DWA (1999) (Table 2.3), from which only those parameters thought to be relevant to road works are shown here. Site-specific, health, spillage, runoff, seepage and disposal of surplus water requirements may also apply and may differ for municipal, industrial and mining water. These are beyond the scope of this report.

Table 2.3: Wastewater limit values applicable to discharge of wastewater into a water resource (DWA, 1999)

Substance / Parameter [1]	General Limit	Special Limit
Faecal Coliforms (per 100 mℓ)	1 000	0
Chemical Oxygen Demand (mg/ℓ)	75 [2]	30 [2]
pH	5,5-9,5	5,5-7,5
Ammonia (ionised and un-ionised) as Nitrogen (mg/ℓ)	3	2
Chlorine as Free Chlorine (mg/ℓ)	0,25	0
Suspended Solids (mg/ℓ)	25	10
Electrical Conductivity (mS/m)	70 mS/m above intake to a maximum of 150 mS/m	50 mS/m above background receiving water, to a maximum of 100 mS/m
Soap, oil or grease (mg/ℓ)	2,5	0

Notes:

[1] Many other requirements are listed, such as phosphate, fluoride, arsenic and many other metals

[2] After removal of algae

2.5.3 Use of treated effluent water in road construction

Disposal of grey (domestic slop, washing, etc.) water containing detergents, etc. onto the street has caused significant damage in the form of ravelling and cracking to asphalt surfacings in low income settlements in Cape Town (Hattingh and Mupra, 1992; Nel et al., 2019), requiring the development of a greywater resistant-asphalt in addition to other measures.

As the quantities of greywater are too small for use in other than small areas of paving, are extremely variable, pose health risks (Carden et al., 2018) and would probably have deleterious effects on all forms of road construction it will not be discussed further.

The extent to which TE is used for roads in South Africa is unknown, but may be extensive.

Since January 29, 2018 no contractors have been allowed to use potable water on road contracts for the City of Cape Town (Erasmus, 2018).

The City of Cape Town put much effort into making the availability of TE known and as a result significant use was made by contractors collecting TE at the wastewater plants and other collection points along the TE reticulation network (Flower, 2018) and is controlled by a treated effluent by-law (Cape Town 2010).

This proved very popular and was used for the compaction all layers except for cement stabilisation (McDonald, 2020) but in at least one case also for cement stabilisation (Bowker, 2021).

TE has been used in Windhoek in sealed roads up to selected levels and for gravel roads with no problems having been reported (Böhmer, 2021). It has also apparently been used in the Beaufort West area, but no details have been obtained.

2.5.4 Comment

The wastewater quality limits, especially the Special Limit (Table 2.3) for treated effluent water are quite severe and in some respects are more so than those of SANS 241-1 : 2011 for drinking water. For example, the maximum EC of 150 mS/m and pH of 5,5-9,5 for the General Limit is slightly more severe than the EC of 170 mS/m and pH of 5,0-9,7 for drinking water

This means that deleterious amounts of sulphates injurious to portland cement should be absent. However, the limit of 1 000 faecal coliforms / 100 mℓ for the General Limit far exceeds the zero limit for drinking water and may be a health hazard for construction workers.

The current draft specifications for state roads (COTO:2020) include the following:

"Purified waste water, also known as effluent, and water from other sources that my contain visible quantities of physical and aesthetical, chemical or organic determinants expected to have a detrimental effect on the road layers, can be considered for the construction of the earthworks and the road pavement layers, provided that it complies with the requirements ….. in the above table". [These are a maximum EC of 170 or 370 mS/m, a maximum TDS of 1 200 or 2 400 mg/ℓ and a pH of 5,0-9,7 for crushed stone bases, chemical stabilisation including curing and bituminous stabilisation, and 370 mS/m, 2 400 mg/ℓ and 4,0-10,0 for other layers and materials.] "In addition, for chemical stabilised layers, the stabilised material shall comply with the strength requirements for UCS and ITS at extended 28-day curing periods and the durability (WDD) requirements specified in Table A4.4-5-2." [Not shown here] "Water containing raw sewage shall not be used anywhere in the construction. Effluent, brackish water and seawater shall not be used for bituminous stabilisation or for diluting bitumen emulsion, unless the use of the water has been approved in writing by the supplier:"

As a general rule it is not expected that the use of TE complying with the General Standard will give rise to any engineering problems in road construction. However, it is known that many TE plants are dysfunctional (Wall, 2018) and that the TE from even well-functioning plants can vary from day to day. One such set of results seen by the author vastly exceeded the faecal coliform limit and must have constituted a health hazard to users.

It is therefore recommended that in cases of doubt comparative tests with TE and drinking water should be carried out and the quality of the TE also be regularly monitored.

It is recommended that a project on the use of wastewaters in road constriction be undertaken. This should include the extent to and for what exact purpose they have been used, any problems encountered, and guidance on their specific uses, as well as potential health, safety and environmental issues.

One such study, in which work on other wastewaters was also reviewed, in the United Arab Emirates concluded that the use of the two TEs used had negligible effects on the compaction characteristics and unsoaked CBR of three apparently nonplastic, graded crushed stone base course materials and a mechanistic pavement analysis also indicated that the use of TE would have negligible deleterious effects on a pavement with 75 mm of asphalt surfacing (Abed et al., 2018).

2.6 USE OF MINE WATERS IN ROAD CONSTRUCTION

2.6.1 Composition of mine waters

Mine water is essentially of two varieties: acid mine drainage and water which is pumped from the mine.

Acid mine (AMD) or rock (ARD) drainage is of localised extent and often extremely acid, pHs as low as -1 being known (Bowell, 1998) and 3-4 being not uncommon (Clark, 1998) and has probably never been used in any form of road construction. However, by 2040 after treatment it is expected to make a significant contribution to South Africa's water mix (DWS, 2018) and may then be suitable. The main pollutants in acid mine water are sulfuric acid, H_2SO_4; acidic hydrated iron (III) ion, $Fe(H_2O)_6^{3+}$; and precipitates of amorphous, hydrated iron (III) oxide, $Fe(OH)_3$ (Manahan, 1997).

The quality of water pumped from a mine can be expected to vary between fresh and saline and/or acid depending upon the nature of the rock, the mineral being mined, whether the mine is active or disused, the depth from which it is pumped, and other factors.

For example, that pumped out of Free State gold mines varies in TDS between about 0,1 and 0,16%, is composed mostly of Na and Cl (Brink et al., 1990), that from Witwatersrand and gold mines 0,6-0,7% with up to 0,1% sulphate and a pH of 3,3-5,7, and that from Witbank coal mines about 0,25% with 0,03% sulfate and a pH of 2,4 (Wagner and Van Niekerk (1987). On the basis of salinity (max. 170 or 370 mS/m) and pH (min. 4,0 or 5,0) depending on use most would be unacceptable for construction water for earthworks and pavement layers (COTO, 2020). At pH values below 5 waters may be highly corrosive both to metals and concrete (DWAF, 1996a).

2.6.2 Use of treated effluent water in road construction

The extent to which such waters have been used in road construction is unknown, but has probably been quite extensive on mine property and possibly nearby municipal and state roads. It is recommended that research into tie use of pumped mine water and treated AMD in road construction should be carried out and guidelines for their use developed.

The suitability of mine water should be assessed according to the specifications for water for state roads, currently COTO (2020), and the General Effluent Standard (DWA, 1999). Further guidance, including sample preservation, analysis, interpretation, possible toxicity, and regulatory requirements have been given by Lenahan and Murray Smith (1980), DWAF (1996a) and in more detail by Pulles et al. (1996).

To date, the author has details of and was involved in only one case on the use of mine water and details of this case are described in section 2.6.3 below.

2.6.3 The Springbok-Komaggas road : A case study in an arid inland area

The upgrading of the existing gravel road between Springbok and Komaggas (MR 745 and DR 2955) in Namaqualand to bitumen standard from September 2003 to February 2005 used water pumped from the incline shaft of the disused Spektakel copper mine for compaction over most of the lengths between km 17,4 and 36,2 on MR 745 and km 0 and 20,8 at Komaggas on DR 2955, and probably others.

Three samples of the mine water taken at different times had a TDS of 5 477-6 439 mg/ℓ (i.e. 0,55-0,64%), an EC of 843-908 mS/m and a pH of 6,7-7,3, with 1 330-1 352 mg/ℓ of Na, 2 978-3 082 Cl, 937-1 106 of SO_4, 459-540 Ca, and 245-282 Mg: with a total hardness of 2 303-2 368 mg/ℓ, a corrosivity index of 35-43, and a

Langelier saturation index of -0,4 to +0,7. No bacteriological or toxicity determinations were carried out, but metals such as Cu were within the General Effluent limits.

All three failed to meet the requirements for human drinking water according to SANS 241:2011, treated effluent, or compaction water. Only the pH was acceptable.

For comparison, a single analysis of the Eselsfontein River water had a TDS of 4 147 mg/ℓ (0,41%), EC of 638 mS/m and a pH of 7,3 and was also of the chloride-sulphate type with a similar hardness and indices. The EC and pH of a local dam water were 0,45 S/m and 8,3, respectively.

For practical road engineering purposes the mine water was of the chloride-sulphate type with a TDS of about 0,60% an EC of 0,90 S/m, and a pH of about 7,0.

The new draft COTO 2020 requirements notwithstanding, the use of such waters for the compaction of a weathered granite gravel base course should not be of much concern as the addition of say 10% such water would only add about 0,06% salt to the completed layer, well below the level of about 0,1-0,2% above which damage is likely. However, a later EC of the mine water was 1,74 S/m, equivalent to a TDS of about 1,1%, which was marginal.

The value of treatment of such a water with lime was considered doubtful and a laboratory test confirmed that it did not lower the EC but actually increased it.

Of more concern was the EC of some of the completed base course which varied between about 0,42 and 0,6 S/m, with a pH of 7,8-8,1.

Whilst the pH was of no concern the high ECs were, and at ECs of 0,26-0,50 S/m the author recommended priming within 7 days and sealing with a rich, impermeable seal within 1-2 days thereafter and that omission of the prime above about 0,5 S/m be considered in order to avoid damage.

Additional precautions recommended including keeping the base grading as coarse as possible within the envelope, sweeping it thoroughly of all excess fines, and the importance of a good seal.

The climate of this area is desert (BW). The normal annual rainfall along the road is between 100 and 200 mm. but is extremely variable, falling mostly between April and September. Weinert's N-value is about 20, Thornthwaite's Moisture Index about minus 50, and the macroclimate Dry for pavement design purposes.

The weather during construction was almost always dry, with a monthly total of between nil and 20 mm from October 2003 to March 2004, and nil to 25 mm from November to September 2004, with a total of 95 mm for the 12-month period.

Such dry conditions, especially during the hot summer months are highly conducive towards soluble salt damage.

The weathered granite gravel base had a marginal EC of about 0,14 S/m before adding water in comparison with the maximum of 0,15 S/m (COLTO, 1998) normally permitted at that time for a completed base course.

The primer used was MSP1 at 0,8 ℓ/m^2 and the surfacing a 9,5 or 13,2 mm on 19 mm granite chip double seal.

Fifty ECs of the completed base course of MR745 between km 29,1 and 40,5 as at 27 April 2004 varied between 0,10 and 0,32 S/m with pHs mostly between 7,0 and 8,0. However, 10 pHs between 6,0 and 7,0 and 13 between 9,6 and 11,0 were recorded. The reason for this was not investigated.

An inspection of the completed and ongoing work by the author together with the Resident Engineer was carried out in late October 2004.

No significant salt damage was seen anywhere in spite of primed base course having stood for up to two weeks on DR 2955 and ECs of up to 0,58 S/m having been reported in a few places on MR 745. Only insignificant looseness or blistering of the primer overspray could be found at such places.

The reasons why no significant salt damage occurred during or within a few months of sealing were probably that the base was generally well swept of excess fines, was generally sealed sooner rather than later, that the GM (a measure of coarseness) was generally high, and possibly because it was not calcrete, which is particularly susceptible to salt damage.

This case should be followed up in order that the time delays during construction and the long-term performance can be recorded.

Research into the use of pumped mine water and treated AMD in road construction should be carried out and guidelines for their use developed.

2.7 SUMMARY

Whereas previously the use of seawater in the base course was not permitted in either South Africa or Namibia the new **draft** COTO:2020 specification for state roads in South Africa allow both seawater and treated effluent water to be **considered** for use in all layers subject to certain provisions.

Sodium chloride (NaCl) crystallising as halite is the dominant salt in seawater and most other waters and is also the disruptive salt generally found in salt-damaged pavement layers other than those made from rocks such as mine wastes containing sulfides.

Salt from seawater restricted to the subbase can rise into the base course and cause severe long-term damage to the surfacing.

However, restricting the seawater to the subbase probably has short-term value in preventing damage to the primed base during construction.

Short-term salt damage during construction can be prevented by priming and sealing as soon as practicable and long-term damage to the surfacing by the provision of an impermeable seal.

Valuable local guides to the use of saline materials and waters in Botswana and Namibia are available.

The use of treated effluent and pumped mine water can potentially make a useful contribution to the water problem and should be researched

Suggested methods of evaluating doubtful waters for road construction are provided in Chapter 5.

Water for concrete is not discussed in detail, for which see Roxburgh (2021).

Salt roads require water with a TDS of at least about 18% and should not be surfaced with any kind of bituminous surfacing without expert advice.

CHAPTER 3: INTERNATIONAL EXPERIENCE AND LITERATURE SURVEY ON THE USE OF SALT WATER IN ROAD CONSTRUCTION

3.1 INTRODUCTION

As Australian conditions of climate, materials, traffic and construction practices most closely resemble those in Southern Africa this survey is largely limited to their findings.

3.2 AUSTRALIA

In Australia the problem of soluble salt damage to sprayed bituminous seals has been known since at least the 1950s (e.g. Chester, 1959; Cole and Lewis, 1960) and recent publications (e.g. De Carteret et al., 2010, 2012; Neaylon, 2014) show that it is still of concern

In South Australia the use of seawater or more saline water led to “powdering” or “fluffing” of the primed base course (Fleming, 1973; Januszke and Booth, 1984, 1992; Foord, 1984). Experience on the Stuart Highway in which the author was involved and which used saline chloride-sulphate water with a TDS of up to 23% is described in more detail in Appendix A1.

On the basis of experience over the previous 25 years on a number of roads affected by salt (mostly sodium chloride) a number of treatments were recommended by Januszke and Booth (1992) according to the salt content of the finished base course (Table 3.1).

Table 3.1: Recommended treatment for roads affected by salt (Januszke and Booth, 1992)

Salt level	Treatment
0-1,5%	As for normal pavement
1,5-2,5%	Primer seal followed as soon as possible by final seal
2,5-3,0%	Sacrificial primed surface swept and followed by a light holding seal and final seal or, if not satisfactory, as for > 3,0%
>3,0%	Emulsion “prime” followed by a C170 light holding seal and double seal

In addition, surface slurrying of fines must be avoided, the “prime” and light holding seal must be applied as soon as possible (but within 48 hours) after completion of the base course, immediately rolled with a 10-15t pneumatic-tyred roller and exposed to as much traffic as possible, the double seal applied as soon as possible (but no longer than three months) with construction traffic routed over its full width.

Essentially the same guidelines have been advocated by the Australian Asphalt Pavement Association (AAPA) (2008).

In short, the key precautions were to have a coarse-textured base surface low in fines followed as soon as possible by an impermeable seal and immediate trafficking.

Owing to the different test methods used these salt levels cannot accurately be converted to the equivalent minus 6,7 mm saturated paste EC (electrolytic conductivity) used in South Africa, but are probably approximately 0,5, 1,0, 2,0 and 2,5 S/m respectively.

In contrast, in Western Australia seawater (Spender, 1973) and even water containing 7,3% salt (Cocks, 1982; Neaylon, 2014) have been successfully used.

In the Australian state of Victoria seawater was found to be satisfactory only for the mixing and compaction of cement-treated materials, but not for curing (Chester, 1959, 1962).

Recent reviews of the salt damage phenomenon in general relevant to Australian conditions are those of De Carteret et al. (2010) and Neaylon (2014) on the use of seawater.

A risk assessment approach to the use of seawater was used for the construction of the airport runway at Onslow in Western Australia. The salinity risk management flowchart used for Western Australian roads (Cocks et al., 2018) includes specific reference to the salinity of the compaction water (Figure 3.1).

The pavement design included 50 mm of dense-graded asphalt on a 200 mm-thick crushed granite base course over 200 mm of subbase.

The runway was constructed in what was essentially a salt pan fed by a tidal salt water creek, with saline groundwater at depths between 0,1 and 3,0 m below natural surface level and also with potential flooding by salt water.

The selected fill was covered with a thick geofabric (Bidim A34) prior to placing the subbase to serve as a drainage and possible salt cut-off layer.

The salinity of the compaction water varied between about 43‰ (4,3%), i.e. more than the 35‰ of seawater, to 91‰ (9,1%) with pHs of 5,5-7,7.

The accelerated pavement construction method was adopted and the actual timing was regarded as critical to success:

“1 Avoid unnecessary repeated wetting of the pavement material during construction (if possible).
2. Should there by any short delays in supply of base and subbase materials, keep the existing layers moist with fresh water, to prevent capillary rise and to avoid the addition of further salt during these delay periods.
3. Do not slurry the basecourse surface. Leave it ‘bony’. Fines aid capillary action and attract salt crystals, and slurried final surfaces are usually provided to meet surface roughness specifications.
4. Advise the asphalt contractor that surface roughness may need to be taken out through the asphalt layer.
5. Prime with an emulsion prime as soon as dry-back has been reached.
6. Within (say) one day of priming, apply a polymer modified binder (PMB) waterproofing seal. The recommended seal was a 14 mm PMB site blended S10E (i.e. about 3% SBS). Do not use cutter
7. Back-roll the waterproofing seal every day with pneumatic multi-tyred rollers until the asphalt is laid.
8. Asphalt (site blended A35P – an EVA plastomer) to be laid as soon as practicable after the waterproofing seal.”

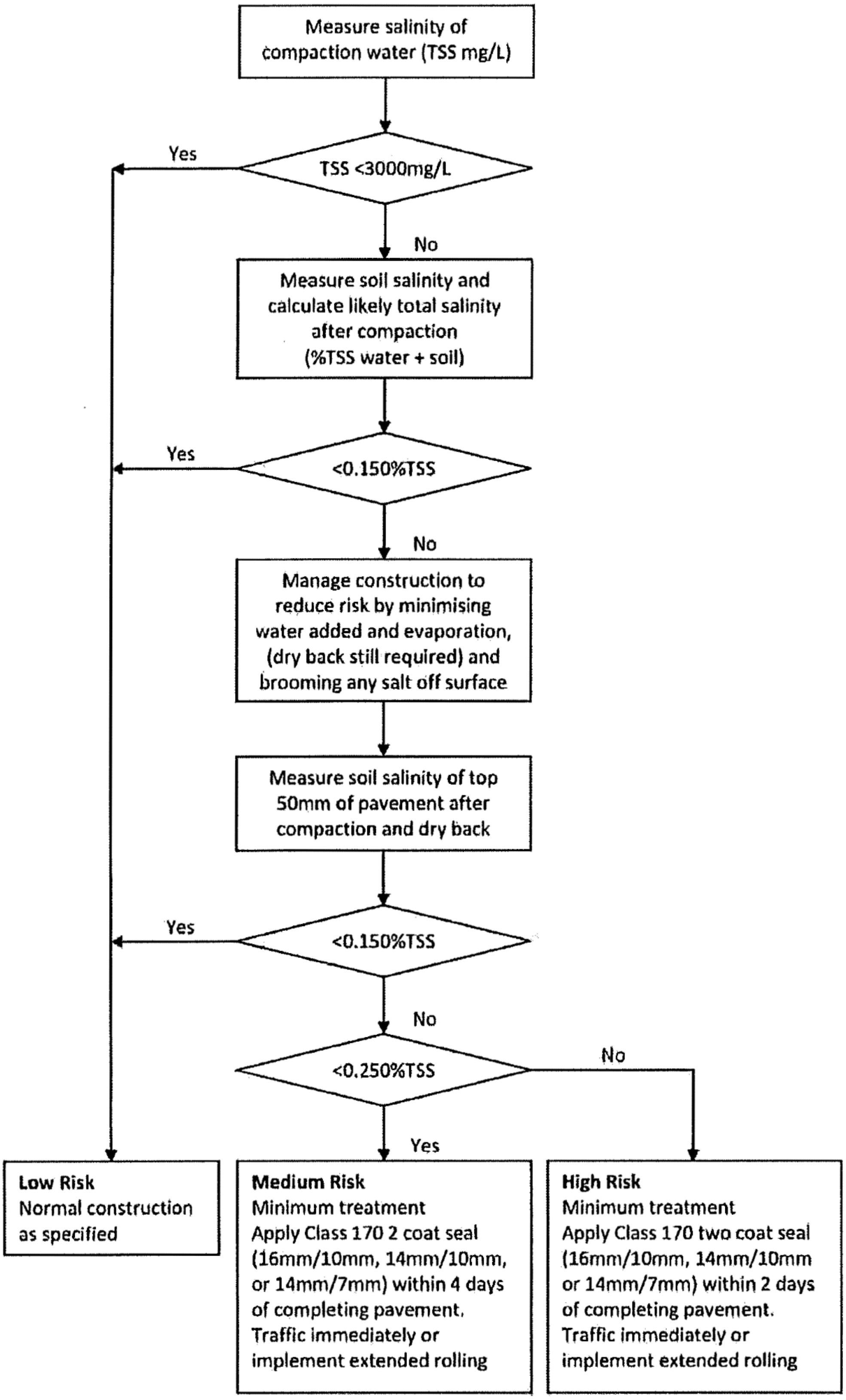


Figure 3.1: Salinity risk management flowchart (Cocks *et al.,* 2015).

In general these timings were adhered to, the prime was applied as soon as dry-back was achieved and the waterproofing seal applied as soon as the prime had dried. The proprietary emulsion prime was found to be unsatisfactory and an on-site mix of 40 parts of C170 bitumen to 60 parts kerosene was then used for the rest of the work.

This was successful until 9 mm of rain fell overnight after which salt crystallisation damage to the prime occurred on drying. However, this was broomed off and the remaining prime remained intact and adhered well to the base course. It was calculated that 8 000 tons of salt were contained within the runway earthworks and pavement at an average concentration of 1% w/w. After seven months there was no salt damage to the runway surface. However, the author has been unable to ascertain the longer term performance of this airport.

The total soluble salt content of the water and soil is determined by evaporating the water and a 4:1 water : soil extract to dryness at 105-110°C, respectively.

A cutback prime may be omitted, but the base is always dried back to 60-85% of MAASHO OWC depending upon the type of base material and the surfacing (Cocks *et al.,* 2021) and water with up to 8% TDS has been used successfully (Keeley, 2021). Further requirements and guidelines are given in official specifications and engineering road notes.

3.3 USE OF NON POTABLE WATER IN ROAD CONSTRUCTION IN OTHER PARTS OF THE WORLD

3.3.1 Far East

Seawater was used during World War II on islands in the Pacific and later in Vietnam without any problems being reported (Brown, 1976). The island runways were either left unsurfaced or typically surfaced with about 75 mm of asphalt on a MC-30 primed, unstabilised coral base. On Tinian Island the use of seawater was actually found to be advantageous as it promoted cementation of the coral (Halloran, 1945).

3.3.2 North Africa and Middle East

In the Middle East seawater has been used successfully for compaction of asphalt-surfaced airport runways in Aden (Vail, 1976), Oman and the United Arab Emirates (Mozley, 1974), Yemen (Murphy, 1986), and in Israel (Nedavia, 1978). Laboratory tests of a road base material in Oman generally showed a decrease in California bearing ratio (CBR) and cohesion but an increase in friction angle when brack, oil-contaminated oilwell production water was used instead of tap water (Taha et al., 2005).

Experience in the dry climate of North Africa including the Sahara (Hamrouni, 1975; Horta, 1985) indicates that seawater can be used successfully, but in other cases salt water of unknown composition appears to have led to priming problems.

If prime damage or efflorescence occurs on the base it is easily removed by brooming when dry and is not likely to recur in the absence of a shallow salt water table. The general practice appears to be to prime and seal as soon as possible in order to prevent damage to the prime.

3.3.3 United States

From personal visits by the author to the United States, seawater and salt water of comparable or higher salinity have been used successfully in gravel bases as well as those stabilised or modified with cement or lime, and probably also in crushed stone bases under a variety of climatic conditions ranging from arid to humid. Whilst primer damage is known, damage to any kind of surfacing due to the use of saltwater does not appear to have been recorded.

Under dry climatic conditions, in west and southwest Texas the primer is usually omitted altogether or the upper base treated with emulsion instead of priming.

In addition, a single or double surface treatment would usually be provided under the asphalt or used alone.

The salt content of such bases seen when completed and also when surfaced must have been in the vicinity of 0,5% or more (equivalent to a < 6,7 mm EC of 0,5-1,0 S/m or more) in many cases. Damage to the surfacing due to the use of seawater or other salt water appears to be unknown.

3.4 SUMMARY

Experience in Australia, North Africa and Texas with roads surfaced with sprayed seals or thin asphalt is most relevant to local conditions as thick asphalt is seldom used here except on major airports.

An accelerated construction method in which the time delay between compaction and sealing according to the salinity of the compaction water and the finished base course, with or without priming, and the type of seal to be used is particularly relevant.

Any damage to the primed base should be broomed off and damage to the seal is unlikely in the absence of a shallow salt water table.

Key factors were to have a coarse-textured base course surface free from excess fines followed as soon as possible by an impermeable seal and immediate trafficking or rolling.

CHAPTER 4: CASE STUDIES OF LONG-TERM ROAD EXPERIMENTS AND OBSERVATION SECTIONS ON THE USE OF SALT WATER IN ROAD CONSTRUCTION

4.1 INTRODUCTION

Because of the varying experience with salt water compaction purpose-built, full-scale, long-term road experiments using seawater for compaction designed and monitored by the author both during construction and for more than the usual design life of 20 years and the analysis period of 30 years were built on the False Bay coast near Swartklip (Appendix A.4) and at Lüderitz in Namibia (Appendix A.6).

In addition, the performance of roads and streets compacted with seawater in the Gqeberha (Port Elizabeth) area (Appendix A.2), the Milnerton-Ysterfontein road (Appendix A.3) in which brackish water was used and a section of road at Lambert's Bay (Appendix A.5) in which seawater had been used was investigated. No cement- or lime-stabilisation was used in any of these cases.

The use of saline borehole water in a lime-stabilised calcrete base is illustrated by the Oshivello-Ondangwa road in Namibia (Appendix A.7) and the full-scale application of the author's preliminary findings on seawater compaction (Netterberg, 1983) by the long-term performance of the Kleinzee-Koingnaas road in Namaqualand (Appendix A.8).

These case histories are described in detail in appendix A and only the most important findings and conclusions summarised here for convenience. The Stuart Highway in Australia (Appendix A.1) has already been summarised in Section 3.2.

4.2 GQEBERHA (PORT ELIZABETH) AREA ROADS (APPENDIX A.2)

4.2.1 Overview

Negative experience in the Gqeberha area in the late 1950s with the use of seawater and tidal estuarine water in all layers including the crushed sandstone base course led the then Cape Provincial Roads Department to restrict the use of seawater to the subbase and lower layers and the salt content of water for the base to 1,2%.

This damage took the form of looseness of the base course before or after priming with gas tar and in one case salt damage to the old-type Cape seal (i.e. with crusher dust premix) which was difficult to repair.

4.2.2 Comment

Seawater and water of apparently greater but unknown salinity was successfully used for the compaction and slushing of graded crushed stone bases without added binder in three out of the five sections of base considered. Both cases of seawater compaction using calcrete as base course were successful.

In both of the earlier roads the use of seawater led to the "fluffing" of the tar primer after it stood for some time. In both cases the loose material was broomed off and the roads surfaced with an old-type Cape seal. In the case that was reprimed no damage to the seal took place. In the other case the road was not reprimed and the old-type Cape seal suffered damage which apparently required three reseals in eleven years to cure.

The later construction both with crushed stone and with calcrete bases, was all entirely successful. It is believed that this was due to priming and sealing as soon as possible, although the season of the year and the weather during construction would also have played a role. In the case of the calcrete it is known that it was sealed within two or three days of completing the base.

The use of seawater probably added approximately 0,3 to 0,5% sea salts to both the crushed stone and the calcrete base materials and in both cases the < 6,7 mm EC of the completed base was probably between 0,5 and 1,0 S/m.

The Sundays River-Colchester road appears to be the only known case where an **unprimed** graded crushed stone base (possibly equivalent to a modern G2) became loose due to salt in the base. No soil binder was added in this case.

The Sundays River-Colchester road, together with a section of the Oshivello-Ondangwa road and an airport in Namibia are the only three pavements known to the author in southern Africa in which removal of the loose surface of the primed base was **not** sufficient to prevent subsequent damage to the surfacing. The reasons for this are unknown.

4.3 MILNERTON-YSTERFONTEIN-LANGEBAAN ROAD (APPENDIX A.3)

4.3.1 Overview

The use of water with a salinity of 0,15-0,8% for the compaction of all layers and the slushing of the crushed G3 base on the Milnerton-Ysterfontein-Langebaan road in 1975-1976 led to surface disintegration of several km of the primed base on this road.

4.3.2 Comment

The Consultant concluded that it was more economic to accept and repair the slight salt damage to the primed base encountered than to avoid the use of the saline materials.

The Consultant also concluded that most of the damage could have been avoided if, in terms of the specifications, the Contractor could have been required to cover the completed subbase with loose basecourse material, to prime the completed base as soon as possible after compaction and to tack and chip as soon as the primer was dry enough to receive it. In cases where the base could not be surfaced soon after completion it should be possible in terms of the specifications to require the Contractor to omit the prime or to delay it until he is able to start surfacing.

A relatively low salt content in the slushing water, perhaps as low as 0,15% was sufficient to add enough salt to the upper few mm of the base to cause salt damage to the primer if left long enough. However, 0,8% salt water was successfully used if the primed base was sealed quickly.

An analysis of some of the records for the southern contract clearly showed the dependence of the damage on both the total salt content of the completed base just before priming and on the length of exposure of the primed base (Figure 4.1) from which a conservative limit of 0,15 S/m was derived. This, together with others (Netterberg, 1979) is the sources of this limit in the CSRA: 1987 and later specifications.

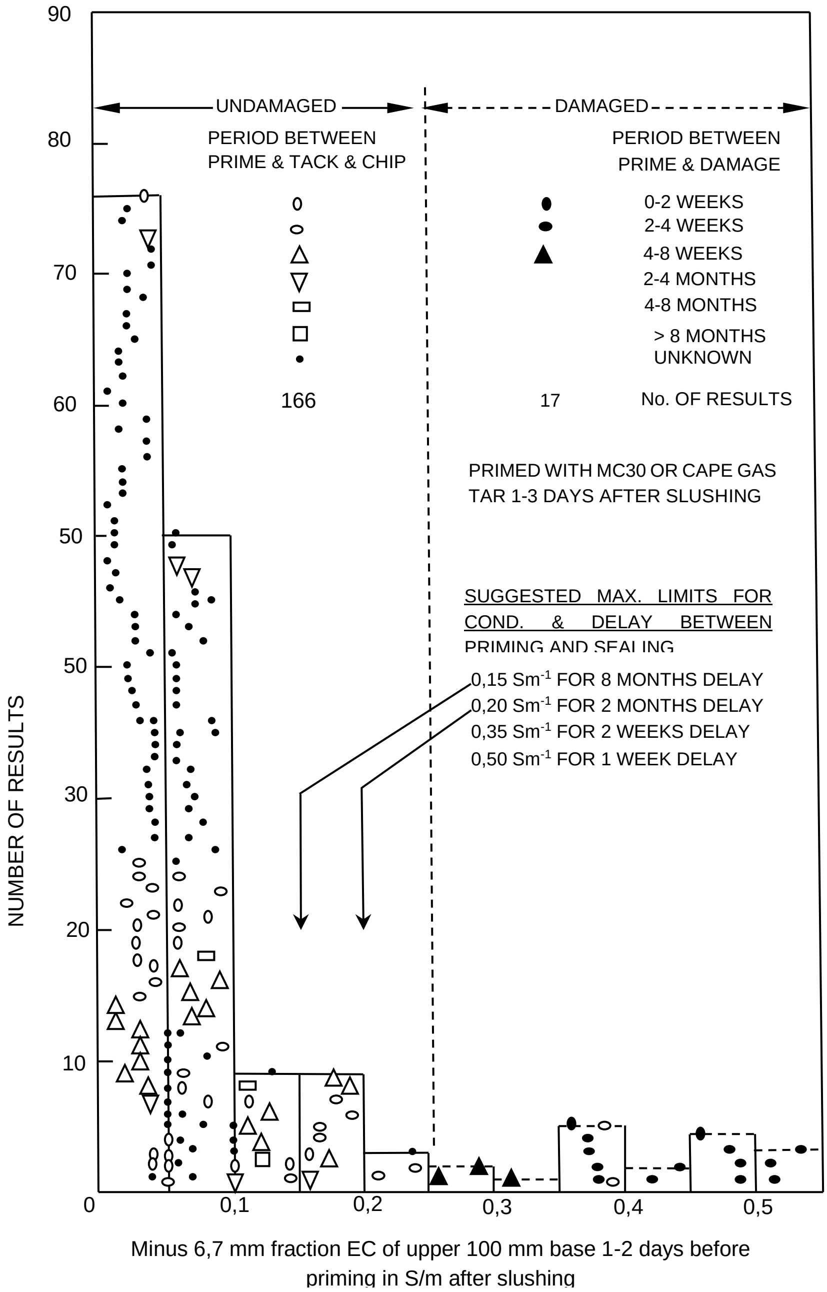


Figure 4.1: Comparison between paste conductivity of crushed stone base course and prime damage for Trunk Road 77 Milnerton-Ysterfontein.

A histogram of 131 EC results from undamaged areas and 33 from damaged areas on the northern contract (not shown) did not show such a clear distinction as those on the southern contract and all damage occurred within 4 weeks. An application of an upper limit of 0,15 S/m would have resulted in acceptance of about 85% of the undamaged areas at the risk of accepting 14% of damaged areas and rejecting only 1% of good areas.

Results from the northern contract further indicated that the crushed stone base became damaged if left long enough when the EC exceeded 0,3 S/m in the upper 15 or 25 mm or 0,15 S/m in the upper 70 mm and that the upper part of a base became damaged when its EC **increased** by more than about 0,15 S/m. (Only that depth over which this actually occurred became loose and/or blistered).

Although this unexpected – and then relatively new – form of damage caused great concern at the time it was mostly easily repaired by brooming and only resulted in insignificant damage to the edges of the seal in the long term.

Experimental sections on the link road between Langebaanweg and Saldanha Bay showed that seawater could successfully be used in both of the two cationic slurry seals forming part of the 19 mm Cape seal (Netterberg, 2004).

4.4 SWARTKLIP EXPERIMENTS (APPENDIX A.4)

4.4.1 Overview

The purpose-built Swartklip experiments were constructed in 1975 as part of an extension of Baden Powell Drive along the southern coast of False Bay from Muizenberg towards Somerset West.

The basic objective was to ascertain how to use seawater in a G3 crushed stone base as well as in a G6 natural gravel calcrete base normally used only as subbase or selected subgrade.

The experiments included control sections in which only fresh water of drinking water quality was used in the base course and a comparison between sections primed and sealed with a 13 mm Cape seal as soon as practicable and those with a longer delay such as would be normally used.

After compaction both the seawater crushed stone bases and the calcrete base had an EC of 0,5 S/m in comparison with the fresh water equivalents of 0,06 and 0,13 S/m, respectively.

No significant damage to the primed base occurred on any of the crushed stone base sections even after a six-week delay between compaction and priming and a two-week delay between priming and resealing.

The calcrete base did not become loose during the two-week delay between compaction and priming, even though the EC had increased from 0,5 to 0,9 S/m just before priming. However, within about one week both the seawater and the freshwater sections experienced surface disintegration, after two weeks extending to 80% and 40% of the surface area of their bases, respectively.

Both of the sections were then immediately broomed and sealed without repriming and the seal completed within five days.

The long-term performance was monitored for more than 20 years with negligible difference being found between the seawater and freshwater sections and only negligible salt damage to the edges of most seals being found, including those in which freshwater was used (Figures 4.2 and 4.3).

The reason for this difference in performance between the crushed stone and the calcrete bases may be because the more deleterious whisker habit of halite is favoured by finer materials (Obika et al., 1997).

Figure 4.2: View towards Muizenberg of SV 200-300 seawater calcrete base section after 19 years in 1994.

Figure 4.3: View towards Faure interchange of seawater crushed stone base section after 19 years in 1994.

4.4.2 Comment

It was agreed by the inspection panel that seawater could be used for base compaction under controlled conditions for both calcrete and for crushed stone bases.

Under controlled conditions the only damage that would be likely would be the loss of the primer overspray within a year and possibly some edge fretting and looseness at the edge of the slurry even if seawater was used only in the subbase, lower layers and the shoulders. The panel considered this to be acceptable.

Considerable salt migration is possible within a finely graded (GM about 1,5) calcrete base and from the subbase if the base is left exposed in hot dry weather. In 26 days the EC of the upper 25 mm of base compacted with seawater doubled, apparently due to concentration within the layer. Over the same period the EC of the full thickness of base compacted with fresh water increased from 0,13 S/m after compaction to 0,2 S/m. Over a further 13 days, during which time the primer was damaged, this rose further to more than 0,3 S/m.

The delay allowable during hot dry weather between compacting and priming a finely graded (GM about 1,5 or less) calcrete base with seawater with a compacted EC of up to and including about 0,5 S/m is at least 26 days. During this period no looseness of the upper base would be expected even if the EC of the upper 25 mm should rise to 0,9 S/m.

The allowable delay during hot dry weather between priming and sealing a similar calcrete base, whether seawater is used in the base or only up to the top of the subbase, with an EC of 0,2-0,5 S/m before priming, is about seven days or less for a gas tar primer.

Rain falling on such a primed base is likely to lead to immediate salt damage.

The best procedure for such a base is therefore to rather leave it unprimed and then prime and seal thereafter as soon as possible, omitting the dampening water spray if possible.

The delay allowable during warm dry to misty or light rainy weather for a G2 or G3 graded crushed stone base between compaction with seawater and priming with gas tar and with an EC after compaction of up to about 0,5 S/m after compaction is six weeks to 3,5 months or more. Under such conditions the EC of the upper 25 mm of base would be expected to decrease substantially and the EC of the full thickness of base to decrease slightly. Under these conditions the base would not become loose.

The delay allowable during hot dry weather for a similar base cannot be established from the results of the experiment, but can be expected to be less.

The allowable delay in warm dry to slightly rainy weather between priming with gas tar and sealing a graded crushed stone base with binder with an EC of about 0,5 S/m after compaction and before priming (and/or an EC of the upper 25 mm of base of up to 1,0 S/m) is nine days or more. For one with an EC of 0,3 S/m before priming (and/or an upper 25 mm EC of up to perhaps 0,5 S/m) this can be extended to 15 days to perhaps two months. Any primer damage at two months is likely to be slight during the wet winter. The delay allowable during hot dry weather between priming and sealing a similar base cannot be established from the results of this experiment, but can be expected to be less, i.e. possibly less than nine days and less than 15 days to two months respectively.

A longer delay is permissible between compaction and priming than between priming and sealing for both calcrete and crushed stone.

These conclusions may be somewhat optimistic owing to the cool misty and drizzly weather experienced during part of the construction period.

In order to minimise upward migration during construction the base course should be primed and sealed as soon as practicable.

Priming a seawater subbase at the normal rate of about 0,74 l/m^2 will have little effect on migration of salt into a freshwater base.

The surfacing used on this road was very impermeable with nearly all values being less than 0,8 mℓ/cm^2/minute soon after construction and dropping to zero within two years.

4.5 LAMBERT’S BAY EXPERIMENT (APPENDIX A.5)

4.5.1 Overview

The approach ramps to the road-over-rail bridge on the Clanwilliam-Lambert’s Bay at Lambert’s Bay were accidentally constructed in 1975 with seawater in the G2 crushed stone base instead of it being restricted to the subbase and lower layers as specified.

Within one month of compaction, during which 16 mm of rain fell, the base of the western ramp primed with an invert cationic emulsion became loose and powdery and was found to have an EC of 0,5 S/m in comparison with the 0,7 S/m of the unprimed eastern ramp which had **not** become loose.

The base of the western ramp was rejected and reconstructed with fresh water.

However the eastern ramp was accepted as an experiment in order to extend the seawater research to a desert climate and broomed, primed and surfaced six days later. It had not become loose at any stage.

The performance of the eastern ramp was monitored visually for seven years until it was resealed at the same time as the rest of the road built in 1970, and for four years thereafter and no salt damage was seen other than minor damage to the slurry overspread and the prime overspray.

4.5.2 Comment

Seawater can be used in a cold desert climate for the compaction and slushing of a G2 crushed stone base course without added soil binder with an EC at any time between compaction and sealing of up to at least about 0,6 S/m, to be used under a new-type 19 mm Cape seal with two slurries.

The completed base course can be left for at least a month unprimed during warm dry weather in autumn and for at least an additional three months during winter without experiencing any significant looseness.

Such a base primed with an invert emulsion primer is likely to become powdery and loose to a depth of about 10 mm within six to 30 days during warm dry or slightly rainy weather. It should probably therefore be sealed within about two weeks if damage is to be avoided.

In the long term the only damage likely is looseness and/or blistering and powdering along the slurry overspread and primer overspray which was regarded as acceptable if seawater had to be used.

A primer alone or a double slurry seal alone is inadequate as a seal for a base compacted with seawater or, probably, for any base with an EC at the time of sealing of much more than about 0,15-0,20 S/m.

The development of holes in a 19 mm Cape seal (due for example to the weathering out of some of the chippings) will not lead to salt damage if it is used on a primed, crushed stone base without soil binder with an EC at sealing of up to at least 0,6 S/m.

4.6 LŪDERITZ EXPERIMENT (APPENDIX A.6)

4.6.1 Overview

As fresh water is extremely scarce along the desert coast of Namibia experimental sections of G3 crushed stone base were constructed in 1976 at Lüderitz as part of the new road to Aus and Keetmanshoop.

These sections included sections with seawater only up to subbase level and seawater in all layers, with and without 5% added gypsum in the base in order to simulate a gypseous binder, using freshwater base sections with and without 0,5% added NaCl as control sections.

In addition, the left lane was primed and the 19 mm Cape Seal completed as soon as practicable up to the first slurry within five days after compaction and the right lane only after 20 days including 18 days before priming in order to simulate more normal construction.

The mean EC of the bases after slushing with sea or fresh water as appropriate were 0,4-0,5 S/m in the seawater sections in comparison with 0,15 S/m in the freshwater section with seawater in the subbase and lower layers and 0,09 S/m in the section with fresh water in all layers.

After about one year the average EC of the gypsum-seawater base (initially at 0,43 S/m) and the freshwater base with the seawater lower layers (initially at 0,16 S/m) were found to be similar at about 0,3-0,4 S/m in both lanes, indicating that salt from the subbase had migrated into the base course.

From this it was concluded that there was little or no long-term value in restricting the seawater to the top of the subbase, although it should prevent salt damage to the primed base before sealing.

Although white salt stains and/or glazing were evident both before and after priming with MC-30 on all the seawater sections the base did not become significantly loose although both it and the primer took longer to dry.

Experiments with the usual primers showed little or no difference between those tried.

A pavement evaluation carried out after three years showed little or no difference between any of the sections in either condition or the predicted structural capacity as indicated by visual inspection, rut depth, smoothness, riding quality, and radius of curvature measurements using a Dehlen curvature meter.

At the final monitoring in 2012 there was little difference between the main seawater section (Figure 4.4) and the freshwater control section and only two rejuvenation sprays and no resealing had been carried out in 36 years.

Figure 4.4: After 36 years without a reseal : Section 11 (seawater in all layers), view towards Aus.

4.6.2 Comment

Seawater can be successfully used for the compaction and slushing of a G3 crushed stone base with added soil binder whether containing gypsum or not, as well as for the underlying layers and for the shoulders, in a climate such as at Lüderitz (BWk'n: cold dry desert with frequent fog).

The performance of the seawater bases was similar to that of sections of similar EC (about 0,5 S/m) to which 0,5% salt (NaCl) had been deliberately added.

The performance of the crushed stone base sections containing added salt at Haalenberg was similar to that at Lüderitz (not shown here).

Seawater could therefore also have been used for compaction of all layers at Haalenberg (BWh: hot dry desert) and therefore for the whole road as far as would have been economic, and probably under any climatic conditions anywhere.

At any time of the year under conditions similar to those at Lüderitz a crushed stone base with soil binder compacted with seawater can be left unprimed for at least 18 days (at least 15 days at Haalenberg) without suffering significant loss of density. The presence of seawater and/or other salt will cause the base to dry out more slowly than normal.

Also under Lüderitz conditions a primed crushed stone base with soil binder compacted with seawater and/or with a similar EC can be left for at least 17 days (14 days at Haalenberg) without suffering significant primer damage. However, very slight and acceptable looseness may occur within ten days when the EC is more than about 0,5 S/m. This should not require any special treatment. [However, experience on the other roads indicates that provision for sealing within one week is advisable at an EC of more than 0,5 S/m, especially (and possibly only) if excess fines are present on the top of the base.] The presence of seawater and/or salt may cause the depth of primer penetration to be less than normal, the primer to take longer to dry and to a reduced application rate of primer being desirable. Any reduction in the primer rate should probably be compensated for by increasing the tack rate in order not to compromise surfacing permeability, durability and bond.

Under Lüderitz conditions a highly saline crushed stone base can be left for a total of at least 19 days after compaction before sealing without suffering damage, whether primed or unprimed. Under Haalenberg conditions a delay of at least 15 days is permissible.

Little difference was found between the performance of the different types and application rates of the primers tried. However, none became significantly damaged on the crushed stone bases.

Under these conditions a primer coat can be omitted. However, the tack spray should then be increased by 0,15 ℓ/m^2.

In the longer term, salt damage to such bases is likely to be limited to looseness or loss of all or most of the primer overspray within one year and, within three years, some looseness, cracking or loss (10%) of the slurry overspread on the base, and general looseness of the gravel shoulders to a depth of about 25 mm.

The foregoing conclusions do not apply to more finely graded or natural gravels such as calcrete, which are likely to suffer severe primer (and possibly some surfacing) damage under these conditions.

A primer alone or a slurry seal on a primer are inadequate surfacings even on a crushed stone base under these conditions and will be rapidly destroyed on a gravel base or shoulder.

Gypsum was not harmful in base courses under the conditions of the experiment and may actually exert a weak cementing effect (Netterberg, 2021).

The use of seawater and/or high salt contents and/or high gypsum contents does not reduce the structural life of a pavement under these conditions.

The salt content of a base of initially low salinity will increase both during exposure and after sealing if placed on a subbase of higher salinity.

Seawater restricted to the subbase will migrate into the base course and within one year the salt content of the base course will be nearly as high as that which it would have been if it had been compacted with seawater in the first place.

There is therefore little long-term value in restricting the use of seawater or excessively saline materials to the subbase unless an intervening impermeable membrane is utilised. However, in the short term (i.e. during construction) such restrictions may prevent or minimise damage to the primed base if it is left exposed for more than two to four weeks before sealing.

Under a relatively impermeable surfacing the salt in a base becomes evenly distributed vertically within the base within one year.

Under the conditions of the experiment at Lüderitz any thermal salt pumping action, diffusion or any other process was acting to even out and not to concentrate the salt in the upper base.

Calcrete natural gravel shoulders compacted with seawater on lower layers also compacted with seawater became loose to a depth of 10 mm within one year and to 25 mm within three years.

The salt content of these shoulders increased from an initial EC of 0,3 S/m after compaction to 0,6 S/m after one year.

Seawater should not be used for the compaction of gravel shoulders if such looseness is not tolerable.

4.7 OSHIVELLO-ONDANGWA ROAD (APPENDIX A.7)

4.7.1 Overview

The Oshivello-Ondangwa road in a semiarid area in the north of Namibia was completed in 1968 with a 25 mm-thick asphalt surfacing on a lime-stabilised, saline calcrete base course.

The upper base became loose and dusty soon after priming and such areas were broomed and one section reconstructed.

However, blistering accompanied by cracking on a starburst pattern then appeared over a 1,6 km section a few months after surfacing.

The chloride-sulphate compaction and curing water used on this section had a TDS of 5,0% and resulted in an estimated EC of 0,8 S/m of the base before curing and the curing water is estimated as having added an additional 3,6% salt to the upper base.

Further investigation of the whole road revealed the upper 25 mm of the base to be significantly weaker than the rest of the base, presumably due to salt crystallisation.

Although it was concluded that the cause of the damage was the crystallisation of halite (NaCl) (Netterberg, 1970) – the first such case recognised in southern Africa – later work (Netterberg and Paige-Green, 1983; Netterberg, 1991) indicated that carbonation of the lime and the lime-soil reaction products during exposure would have been a contributory factor.

Although there was great concern over this problem at the time and an additional 25 mm of asphalt placed over the worst section also exhibited mild salt damage the road nevertheless gave satisfactory service.

4.7.2 Comment

Chloride-sulphate water with a TDS of up to at least 5% m/v can probably be used for compaction of a lime-stabilised base of initially low plasticity but should not be used for curing.

Curing water used two or three times a day for four days should probably be limited to a TDS of less than 1,8%.

The theoretical < 6,7 mm EC of a lime or cement-stabilised base calculated from the initial EC of the material and the amount of salt added during compaction (or measured experimentally but without adding the stabilizer) should possibly be additionally limited to a maximum of 0,5 S/m.

Other possible factors such as carbonation, inhibition by a penetrating primer, and over-compaction due to the rapid-hardening calcrete when stabilised with lime make it difficult to define the role of salt in the distress on this road more closely. However, the combination of curing an already saline base (estimated EC of 0,8 S/m after compaction) with 5% saltwater was definitely implicated in the blistering and star cracking of the asphalt.

Curing any stabilised layer with salt water is undesirable and curing any saline base with even fresh water is also undesirable unless the upper base can be prevented from drying out between water sprays.

The surface “powdering” of the primed, lime-stabilised base on this road as well as the blistering and star cracking of the asphalt surfacing were similar to the salt damage experienced by pavements with unstabilised gravel and graded crushed stone bases.

This case showed the necessity of having a rapid site laboratory test for soluble salts and led to the adoption of a modified form of the soil paste electrolytical conductivity test used in soil science (Netterberg, 1970) as NITRR Method CA21-1974 (1980) which was subsequently adopted in TMH1:1979 and 1986.

4.8 KLEINZEE-KOINGNAAS ROAD (APPENDIX A.8)

4.8.1 Overview

This 65 km-length of private road completed in 1980 as a link between the Kleinzee and Koingnaas diamond mines (Spottiswoode and Graham, 1982 a,b) represents the first full-scale application of the author’s research on the use of calcretes (Netterberg, 1971; 1982) and seawater (Netterberg, 1983) in road construction.

Mine tailings with an average EC of 0,25 S/m and natural gravel calcrete with 1,1 S/m after mixing and watering with seawater were used as base course under an emulsion double surface treatment with a fog spray.

Such high ECs were far higher than the maximum of 0,15 S/m usually specified and would never have been permitted by any South African road authority.

However, the accelerated construction method was adopted, with the omission of a prime coat and the requirement that the subbase (mostly the existing saline calcrete gravel road) be covered by the base within 14 days and the base sealed within 7 days of compaction.

Although some looseness and edge fretting of the edges of the seal occurred within two months of sealing the emulsion tack coat without a prime was adhering well and no surfacing was lost due to salt crystallisation.

Three long-term problems due to salt were encountered on this road: holing of the seal due to the crushing and disintegration out of a small proportion of inferior calcrete chippings, edge fretting, and shoulder erosion. The classic form of salt damage, i.e. that of blistering and cracking of the surfacing was almost completely absent.

Apart from this only normal maintenance was generally necessary and the general surfacing and pavement condition was always rated as good or very good during subsequent inspections and pavement evaluations up to at least the last one carried out after 28 years by the author in 2008 (Figure 4.5).

Figure 4.5: Typical view towards Kleinzee after 28 years in May 2008 from km 31,5 over km 30-35 segment with mean base EC of 1,35 S/m and subbase EC of 0,79 S/m after compaction with seawater.

4.8.2 Comment

This case history has been included to show what can be done even with inherently extremely saline materials compacted with seawater in the case where the client is willing to accept a higher level of risk in order to contain costs than would normally be acceptable to a state roads authority.

Seawater can be used successfully for the compaction of already saline, coarsely graded (GM ≥ 2,0) natural and crushed gravel bases with OMCs in the 10-25% range in a cold dry desert climate to be used under an impermeable double surface treatment without a prime, with salt contents in the surfaced base course up to an equivalent < 6,7 mm EC of about 2,0 S/m.

In order to limit the upward migration of excess salts from the construction materials by covering each layer as soon as possible after completion the following clauses were added to the Conditions of Contract:

- The base course had to be completed within fourteen (14) days of completion of the subbase.
- The bituminous double seal had to be completed within seven (7) days of completion of the base course.
- The Contractor was required to accept full responsibility for any defects that might have occurred due to his failure to comply with the above.
- A prime coat was to be omitted and the first spray of 60 or 65% bituminous emulsion had to be applied directly onto the completed base.
- The base had to have been well swept of all dust and loose material and had to be either dry or slightly damp, whichever proved to provide the best adhesion to the surfacing.

The Contractor generally had no trouble in meeting these requirements.

The road has been successful in meeting the Owner's requirements of an adequate low volume surfaced road at the lowest possible initial cost and without excessive maintenance costs.

The alternatives of providing a gravel or crushed stone base of normally acceptable salinity and compacting it with fresh water would have been prohibitively expensive

No salt damage was encountered during construction.

However, shallow holing ("potholing") and fretting of the edges of the seal partly due to salt crystallisation began a few months after construction, but the usual form of salt damage, i.e. blistering and. cracking of the surfacing was almost completely absent.

The shallow "potholing" with an average diameter of about 50 mm was largely due to the combined effects of a small but variable proportion of soft calcrete in the sealing aggregate leading to holes in the seal and the high salt content of the base. As such, it was a surfacing rather than a structural failure.

The frequency of "potholing" was largely independent of the salt content of the base within the range of EC of 0,15-1,0 S/m after compaction or 0,4-1,3 S/m at sealing, but salt contents in excess of 1,0 S/m after compaction or 1,3 S/m after sealing were associated with an increased frequency of "potholing".

The edge breaking (fretting) was due largely to the combined effects of salt action in loosening the edges of the seal and the adjacent shoulder material and bus traffic on the narrow 6,0 m seal.

The "potholing" was not regarded by the mine engineering staff as a significant problem, provided that it did not recur on the patch, but maintenance of the shoulders and edge seal was.

The average maintenance costs over the first six years of R245/km/y were 13% higher than the R215/km/y predicted for any road under similar conditions of climate and traffic, but without the high extent of "potholing" and edge breaking experienced.

The road was resealed about two years sooner than would have been expected for a double seal under similar traffic conditions.

The "potholing" and the necessity for an early reseal could largely or entirely have been prevented by the use of better-quality chippings containing no very weak aggregate, less dust, and a better grading.

Fretting of the edges of the seal into the design width can be prevented by grading the shoulders over the edge of the seal, (not recommended due to windscreen damage), by providing a sacrificial extra width of seal, or by providing an impermeable, thickened edge to the seal, which was adopted.

Fretting of the edges of the seal and looseness and erosion of the shoulder can probably be minimised to a maximum of Degree 2 by limiting the salt content of the surfaced base course and the completed shoulder to a maximum < 6,7 min of 0,5 S/m and by utilising material also suitable for the wearing course of an unpaved road.

The use of calcrete with a completed EC of more than 1,0 S/m as a combined base and shoulder, especially if it does not comply with a suitable specification for a calcrete wearing course may lead to severe edge fretting of Degree 4 or 5 in six years as well as to more severe shoulder looseness and erosion.

All areas of the double seal remaining completely undamaged or having only barely discernible cracking of Degree 1 or 2 after 6 years of service were found to be completely impermeable to air at a water pressure head of 6 mm and to have a maximum permeability of 1,5 mℓ/cm^2/minute at a head of 20 mm of water. Exactly the same spots as tested above were also impermeable to water using the California grease ring method.

A single surface treatment under such conditions is not advisable at an EC of more than about 0,4 S/m at sealing.

The salt content tolerable by a good, impermeable double surface treatment on a coarsely graded (GM ≥ 2,0) calcrete base with a PI of SP -15, a LSM of 20-220 and MAASHO OMC of about 14-16% in an arid climate appears to be equivalent to an EC in the vicinity of 1,8-2,0 S/m after surfacing or, conservatively, 1,5 S/m after mixing and watering but just before compaction.

The EC of both the tailings and the calcrete bases rose by an average of 0,27 S/m from just before compaction until sampling four days to two months after sealing (mostly within seven days of compaction).

The reasons for this rise were presumably migration from the subbase during the few days of exposure before sealing and, in a few cases only, the addition of extra seawater during recompaction, and migration from the subbase due to equilibration after sealing

A detailed pavement evaluation in 1986 after about five years of service showed the average rut depth to be about 5 mm, ruts of more than 10 mm to be rare, with a general pavement condition rating of good except for one 5 km segment noted as “poor” due to a few localised shear failures on a micaceous subbase which were not caused by salt. On a best average basis the whole segment could have been rated as good.

The road was structurally sound, with general pavement condition ratings of fair to good after 13 years in 1993 and good to very good after 28 years and qualified as a Category C road still in sound condition in 2008.

With good maintenance the road easily exceeded its design life of 0,05 MESA in 15 years and had probably carried at least 0,1 MESA after 28 years in 2008.

4.9 SUMMARY

Seawater or salt water with a similar salt content of up to about 3,5% (EC of about 5,4 S/m) can successfully be used for the compaction and slushing of a G2 or G3 crushed stone base as well as the underlying layers with only negligible long-term salt damage to the edges of the seal and possible surface loosening of an unsealed gravel shoulder, without detriment to the structural capacity of the pavement.

However, even the use of water with a TDS as low as 0,2% (EC of about 0,31 S/m) can result in surface disintegration of a primed G3 base if left unsealed for long enough (several months).

A crushed G2, G3 or natural gravel G5-G6 calcrete base of inherently low salinity of less than about 0,15 S/m compacted with seawater is likely to have an EC of about 0,5 S/m.

Salt damage to a primed base course can be prevented by priming and sealing as soon as practicable after compaction.

The maximum advisable delay between priming and sealing a G3 base varies between up to 8 months for an EC before priming of 0,15 S/m to about one week for an EC of about 0,5 S/m.

The maximum advisable delay for a G6 calcrete base with at GM of about 1,5 compacted with seawater is seven days or less. Rain falling on such a primed base is likely to result in immediate salt damage on drying.

Should such damage occur it can usually be broomed off and the base sealed as soon as practicable with or without repriming without significant subsequent salt damage to the seal.

If a long delay between compaction and sealing cannot be avoided it is better to leave the base unprimed and then to prime and seal as soon as practicable, omitting the dampening water spray if possible.

Depending upon the season and weather, the allowable delay for an unprimed G2 or G3 base might be at least six weeks for a G2 or G3 base at least three weeks for a G6 calcrete base.

However, in hot, dry weather in summer it would be safer to allow a maximum of three weeks unprimed for a G2 or G3 base, two weeks for a calcrete G6 base followed by sealing within 1-2 weeks thereafter for a G2 or G3 and 7 days for the calcrete.

Restriction of seawater to the subbase and lower layers or even priming a seawater subbase at the normal rate of about 0,7 ℓ/m^2 will not prevent migration of salt into a base course compacted with freshwater.

However, it should delay such migration and permit a longer delay between compaction and sealing.

The deliberate punching of holes in a rich 13 or 19 mm Cape seal on a G2, G3 or a G6 calcrete base compacted with seawater did not lead to potholing; however the development of holes due to a small proportion of weak aggregate in a double seal on a highly saline G5 calcrete base compacted with seawater did.

Seawater can be used for the compaction of an inherently highly saline (EC 1,0 S/m) but otherwise G5, coarsely graded (GM ≥ 2,0) calcrete base with a mean and maximum EC of up to 1,0 and 1,5 S/m respectively provided that the base is completed within 14 days of completion of the subbase and sealed within 7 days of completion of the base. Under such circumstances the prime may have to be omitted and the emulsion tack spray to a rich double seal with fog or rich Cape seal increased. In addition, the sealing aggregate must be high quality with no weak particles and it may be necessary to provide a thickened edge to minimise edge breaking and shoulder erosion.

Seawater or similar salt water can be used for the compaction of a coarsely graded (GM ≥ 2,0) saline, lime-stabilised G5 calcrete base with a raw PI of up to 10, but fresh water should be used for curing. In addition, the layer must be kept continuously moist and not subjected to wet-dry cycles and probably not left for more than about two weeks before priming and sealing within one week thereafter. In short, to minimise the possibility of both salt and carbonation damage it should be sealed as soon as practicable after compaction with an impermeable seal in preference to asphalt.

In addition, the sealing aggregate must be high quality with no weak particles and it may be necessary to provide a thickened edge in order to minimise edge breaking and shoulder erosion.

In all cases the use of seawater in the base requires the maintenance of a good seal with timely rejuvenation or resealing, especially in the case of natural gravel bases.

CHAPTER 5: GUIDELINES ON THE USE OF NON-POTABLE WATER IN ROAD CONSTRUCTION

5.1 QUICK GUIDE TO ACCELERATED CONSTRUCTION WITH SEAWATER

5.1.1 Introduction

The usual (e.g. COLTO, 1998) requirement for a road to be covered with a bituminous surfacing of a maximum electrolytic conductivity (EC) of 0,15 S/m for a completed base and subbase is a conservative limit intended for normal use without requiring any special precautions and in order to allow a layer and – especially a primed base course – to stand for some time with little risk of salt damage.

The current national draft South African specifications for water (COTO Oct. 2020) limits the TDS of compaction water to a maximum 1 200 mg/ℓ (0,12%) for crushed stone base course and stabilised materials and 2 400 mg/ℓ (0,24%) for other layers and materials. However, **consideration** of the use of brackish and seawater is permitted provided that similar limits to those of COLTO (1998) are not exceeded without expert advice and proved in a trial section (see Section 2.2.1 for further details).

Compaction of most pavement materials with an inherently low EC of less than about 0,1 S/m with seawater or other comparable chloride-sulphate water with a salinity of about 3,5% will raise the EC to about 0,5 S/m, which exceeds the maximum of 0,40 S/m specified for selected layers.

The accelerated construction method was developed in order to enable the use of both inherently saline materials and waters with a salinity up to approximately that of seawater for the compaction of unstabilised materials with little risk of salt damage.

It essentially involves covering each layer with the next as soon as practicable – even just by dumping and spreading it – to minimise upward migration during construction and to then prime and seal the base as soon as practicable after compaction in order to keep the salt safely in solution.

5.1.2 Maximum target delays between layers

The following target delays assume an EC after compaction of about 0,5 S/m but are guidelines rather than specifications as the time before damage occurs also depends upon the season, weather and the particular material and should be varied according to the measured EC – the higher the EC the shorter the delay allowable. Finer-grained materials – especially calcretes – are more liable to salt damage than coarser and also require more water for compaction.

5.1.2.1 *Delay between earthworks layers*

Cover each layer as soon as practicable with the next layer and broom off and/or recompact as necessary. In extreme cases cut off the upper part of the layer and thicken the overlaying layer.

5.1.2.2 Delay between subbase and base

The maximum target delay recommended between a G6, G5 or G4 subbase and completion of the base is three weeks for an EC of 0,26-0,50 S/m and two weeks for an EC exceeding 0,50 S/m.

5.1.2.3 Delay between base and priming

The maximum target delay recommended between completion of the base and priming with a cutback or invert emulsion primer is as follows:

Table 5.1: The maximum target delay recommended between completion of the base and priming

EC (S/m)	G4 or G5	G2 or G3
0,26-0,50	7 days	14 days
0,50-1,0	2 days	14 days

5.1.2.4 Delay between priming and sealing

The maximum target delay recommended between priming and surfacing is as follows:

Table 5.2: The maximum target delay recommended between priming and surfacing

EC (S/m)	G4 or G5	G2 or G3
0,31-0,60	24 hours	14 days
0,61-1,0	6 hours	7 days

5.1.3 Additional precautions

Slushing should be avoided on G4 and G5 bases if possible.

The base must be well-swept of excess fines to as a good mosaic as possible before priming and the use of a dampening spray even of fresh water avoided if possible. If necessary the prime should be omitted and the emulsion tack coat applied at a higher rate.

It is better to delay the priming rather than the sealing as rain on a primed base with an EC of more than about 0,5 S/m may cause immediate prime damage on drying, especially on a calcrete.

The surfacing should be a double seal with a fog or a 19 mm Cape seal with two slurries, in preference to asphalt, designed to be impermeable and as rich as possible without excessive bleeding.

The surfacing aggregate must be of high quality with a minimum of dust and soft or weatherable particles, especially on gravel bases.

The surfacing must be well-rolled over its full width with a pneumatic roller and opened to traffic as soon as possible, if necessary continuing the rolling until opening.

Sealing the shoulders with the same seal should be considered at an EC of about 0,5 S/m.

As most of the salt remains within the pavement the success of this technique is reliant upon reasonable maintenance and the preservation of surfacing integrity.

The surfacing of a base with an EC exceeding about 1,0 S/m or the presence of a shallow water table, especially if it is saline, adds additional risk and requires additional precautions, for which see the text and Appendix A.

5.1.4 Repair of salt-damaged primed surfaces

Should damage to a primed surface occur it should be well swept, any hollows patched, reprimed and/or the spray rate of the tack spray increased if necessary, and sealed as soon as possible thereafter.

Such damage does not usually lead to surfacing damage but may result in some loss of smoothness.

5.2 SUGGESTED METHODS FOR EVALUATING WATER FOR ROAD CONSTRUCTION

5.2.1 Introduction

There are no standard specifications or methods for evaluating water for compaction and curing of treated and untreated pavement materials. This note has therefore been prepared as a guide. **All of these tests will seldom be required** and each case must be considered on its own, bearing in mind the time available, the degree of certainty required, and whether testing the combined effect of the material and the water is necessary (recommended) or the effect of the water alone is to be isolated.

Aspects which should be considered are:

(a) Engineering: efflorescence (surface disintegration, loss of density, blistering, staining), corrosion, compactability, strength, density and durability;
(b) Environmental: possible effect on health and safety of workers and all life (not treated here)

No discussion of these problems can be presented here. It is the purpose of this note simply to suggest those tests which should be considered. The chemical tests should be regarded as indicative and the physical engineering tests and road trials as more definitive. However, little research has been carried out except in the case of damage to primed surfaces and bituminous surfacings.

5.2.2 Chemical analysis and tests

The following should be determined according to the origin and proposed use of the water:

Note that water samples must be taken in clean containers, rinsed out with the water to be sampled, filled completely, and tested as soon as possible. If possible also measure the pH at source. Samples for bacteriological analysis must be taken in sterile containers and tested within 24h. SANS or DWAF requirements should be followed for rigorous work and in cases of dispute.

Borehole and other apparently non-polluted or effluent waters: as for the proposed use.

Effluent and other waste or apparently polluted waters:

(a) for compliance with the General or Special effluent standard as appropriate;
(b) as for the proposed use.

For compaction of untreated materials: EC, TDS and pH (COTO:2020)

For compaction and curing of cement or lime-treated materials: EC, TDS, pH and sulfate (COTO: 2020).

For concrete: COTO:2020 only refers to SANS 51008:2006.

For diluting emulsions (see also COTO:2020 and also consult the manufacturer):

(a) Cationic: EC, pH, total alkalinity, titration with 0,1M HCl to pH4.
(b) Anionic: EC, pH, total hardness.

Use the TMH1 A21T Saturated paste EC and **saturated paste pH (NITRR CA21-1974 (1980) test methods** until the new SANS GR32 method is available. Do **not** use the old SANS methods.

Test the proposed material with (a) the proposed water and (b) distilled or other good, non-saline potable water.

A more accurate simulation should be to determine the EC and pH with distilled water after adding the estimated amount for compaction and also slushing or curing if appropriate and allowing for every extra that might be added to allow for evaporation, etc., to the whole grading before air-drying and screening out the <6,7 mm fraction for the test.

For salt roads use a hydrometer graduated in degrees heavy Baumé (see Section 2.4).

Organic matter
As above but use SANS AG40 test for organic impurities in fine aggregate and record actual colour obtained. On site, the test can be carried out on the water itself by filling the bottle to the 125 m**ℓ** mark with the water to be tested instead of the fine aggregate. This is in addition to the standard COD test usually recommended, when the pH is <5.

As above but use SANS AG42 test for sugar. On site, the test can be carried out on the water itself by using 1 m**ℓ** of it instead of the water extract of the fine aggregate.

Sulphates
As in above but from TRH 13:1986 analyse for:

(a) Acid-soluble sulphate according to BS 1377:**1975**, Test 9, on the whole grading crushed <2 mm, **without** using the correction factor for the percent passing 2 mm.
(b) Water-soluble sulphate according to BS 1377:**1975**, Test 10, on the fraction passing 2 mm.

Use these methods until the new SANS GR33 methods are available. Do **not** use the old SANS methods.

5.2.3 Compactability

Setting time of cement: compare the initial (IS) and final set (FS) of the proposed cement with both (a) the proposed water and (b) distilled or other good, non-saline, potable water.

Rate of early strength development: compare the unsoaked CBR of the proposed mix with both (a) the proposed water and (b) distilled or other good, non-saline, potable water, compacted at MAASHO OMC and the appropriate effort for the layer (say MAASHO for base, NRB for subbase and Proctor for selected) and tested after different curing times, say 0, 4 and 6h.

CBR and density vs time: as above, however only compact the 0h specimen at once. Cover the other two to cure uncompacted, mixing thoroughly with a trowel every half-hour and compact at say 4 and 6h. Determine the unsoaked CBR of **all** three as soon as possible after 6h and (b) the soaked CBR or UCS after the standard 7d or whatever curing time is specified for the works.

This method is intended to simulate the effect of normal construction on both strength and density.

If practicable, the UCS test can be used in preference to the CBR.

The purpose of the above testing is to determine beforehand whether the water or the water-material combination is likely to accelerate the setting or hardening of the mix and thus cause compaction problems (e.g. lower density and/or strength, and degradation and/or shearing of the upper part of the layer). If preferred, or in addition, they can simply be substituted by the normal or modified compaction trials which should in any event always be carried out at the start of the work.

5.2.4 Strength and density

UCS and/or ITS dry density: after 7, 28 and preferably 56 days of curing with (a) the proposed water and (b) distilled or other good, non-saline, potable water.

If compactability is a problem: UCS and/or ITS and dry density as above but after say 0, 4 and 6h waiting and mixing times as for CBR and density above.

5.2.5 Durability

Wet-dry brushing test (SANS GR 55) with both (a) the proposed water and (b) distilled on other good, non-saline, table water. If it is also desired to use the proposed water for curing, then it should also be used for the wetting cycles. If not, then the good water should rather be used for wetting.

UCS and density after extended soaking: If excess sulphates or clay is suspected, compare a 14d cured, 4h-soaked UCS with a 7d cured plus 7d-soaked UCS. Record also the dry density and any signs of cracking or disintegration.

UCS, density and PI after accelerated carbonation: This durability test, which should **always** be carried out when marginal materials are stabilized for strength, should preferably be carried out with both (a) the proposed water and (b) distilled or other good, non-saline, potable water.

ICL/ICC: This test, which should **always** be carried out where stabilization is carried out for strength, should preferably be carried out with both (a) the proposed water and (b) distilled or other good, non-saline, potable water. Use the SANS GR 57 gravel method preferably doing 24h, 7d and 28d measurements on the same 1h mixes as well as the 1h measurements and determining the PI at the end.

The ICL or ICC of the water itself can be determined by essentially the same test if desired.

5.2.6 Efflorescence

Efflorescence, surface disintegration, superficial loss of density and strength, and blistering of primed surfacings are probably best evaluated by means of model experiments in CBR moulds as outlined by

Netterberg and Loudon (1980) and Jones and Netterberg (1999). **Prevention** of this kind of damage is best achieved by applying construction time constraints (essentially covering up each layer as soon as practicable).

CBR mould tests: Compact one mould of the soil at MAASHO OMC with the proposed water at the appropriate effort for the proposed layer. Take an initial swell reading. Partially dry the specimen in an oven at about 50°C or in the sun to less than about half OMC and inspect daily. The perforated soaking base plates should be on, but not the stem plates and weights. If the effect of priming is to be evaluated, lightly dampen the surface of an additional specimen and apply the proposed primer at the proposed rate.

Place both specimens in individual capillary soaking baths with the water level at about 50 mm above the top of the base, plate and allow to dry in an oven at 50°C or in the sun. The **baths** should be **almost** completely sealed to minimise water loss. Top up the water level once after the first day only. Inspect daily until moisture content is less than about one quarter of OMC. If the effect of curing is to be evaluated, rather sprinkle the unprimed mould with the proposed water the proposed number of times a day for the proposed period of time and then allow to dry as above. Inspect daily.

Replace the stem plates and weights, take a swell reading, soak for 4 days and determine CBR and swell. Calculate expansion and dry density at the various stages. Repeat with control specimens of the same mix but using distilled or other good non-saline potable water.

It is advisable to carry out this test with duplicate specimens. The number of specimens and the treatments applied should be varied so as to closely simulate the construction process.

UCS specimen tests: The efflorescence test described in ASTM 67 for brick and structural clay tiles and suggested by Suprenant (1992) for concrete can probably also be applied: Place one specimen made with the proposed water in a partially sealed water bath filled with the proposed water to a depth of about 25 mm. Dry in an oven at not more than 50°C or in the sun for 7 days and inspect daily. Repeat the whole procedure at the same time with distilled or other good, non-saline potable water.

Field trials: If conditions permit, the laboratory tests can be omitted and the usual compaction trials (modified as necessary) used instead – or in addition to the laboratory tests.

5.2.7 Emulsion

Dilution "can" test: "The emulsion is diluted to specification, or in the case of emulsion for slurry, diluted to the ratio of one part emulsion to ten parts of the suspect water, in a clean container such as a can. The water is added to the emulsion (not emulsion to water) to prevent premature breaking. The "can" is then heated to about 60°C and left to stand for 20-30 minutes. The diluted emulsion is then passed through a fine sieve (0,600 mm) to determine if any premature breaking has taken place" (COLAS, 1997) and see also COTO,:2020.

The dilution water for cationic emulsion must not be alkaline. In the case of a cationic emulsion, the addition of hydrochloric acid to the water should be considered. For anionic emulsion, lime ($Ca(OH)_2$) or caustic soda (NaOH) can be added to the dilution water if necessary.

Further chemical testing can be done if the "can" test shows premature breaking. See Section C.2 above and, e.g., COLAS Coltech Technical Note 8/96 and consult the manufacturer.

5.3 WATER FOR CONCRETE

The water requirements for portland cement concrete on a road project are usually small other than for concrete pavements and are not considered here in any detail, for which see Roxburgh (2021).

Potable water is almost always suitable and used, although brackish, salt and even seawater can be used in plain concrete without reinforcement or other embedded metal, and waste waters may also be suitable in all forms of concrete and can be assessed using SANS 51008:2006/EN 1008:200 (Goodman, 2009). However, conservative restrictions are usually placed even on the TDS for plain mass concrete in order to prevent any possible errors or problems such as efflorescence, and the COLTO:1998 and now COTO:2020 (which simply refers to SANS 51008) requirements are relevant for road projects.

Only potable water should be used for curing. The author's only experience of the use of seawater in concrete was on some experimental plain concrete paving slabs laid at the old Walvis Bay airport in 1964. There was no significant difference in strength, performance or visual appearance between them and adjacent slabs made with freshwater for over 30 years until all were removed when the airport was upgraded.

5.4 WATER FOR SALT ROADS

So-called 'salt roads' have been used along the desert coast of Namibia mostly in the Swakopmund area for many years (Bravenboer, 2011) as a dust-free economic alternative to bituminising. With time they acquire a dark grey colour and are often mistaken for bitumenised roads by visitors. They are essentially unsealed gravel roads with a wearing course of plastic, saline, gypcrete gravel compacted with salt water and yield excellent performance except when it rains, which is seldom.

Seawater is insufficiently saline for this purpose and waste brine from the coastal salt works or salt water from a salt pan is used. The use of such waters results in severe corrosion of roadmaking equipment, especially tankers. Whilst this no doubt also affects vehicles using these roads no details thereof have been obtained, nor have any health and safety or environmental implications been considered.

A minimum TDS of 18% in the compaction water is specified for the upper 50 mm of the wearing course and 8% for the lower 100 mm (Roads Authority, 2014). Research by the author together with the local maintenance unit indicated that water with 11% salt was the worst usually used, 22% gave good results and 24-29% very good results. However, the very best water was obtained by excavating a hole on the edge of a salt pan and waiting for the water to exhibit crystallisation of salt. The salt content of such a water would have been around 36%. The salinity of the water on site was determined using a hydrometer graduated in degrees heavy Baumé, which is approximately equal to the salt content.

No attempt should be made to apply any kind of bituminous surfacing to a salt road without expert advice – see Figures 1.2-1.4 and 2.1 for the probable results.

CHAPTER 6: CONCLUSIONS AND RECOMMENDATIONS

6.1 CONCLUSIONS

Factors that appear to be common to practically all cases of surface salt damage to a primed base globally, whether the salt was intrinsic to the material or added by the compaction water, are that it had been exposed for some time (often months) whereas it had remained undamaged for some time beforehand, that it often occurred during drying out after a shower of rain, and that it occurred more often and was worse on finely graded materials, especially calcretes.

The common method therefore adopted is use a coarse material where possible, to omit or limit slushing, to remove excess fines from the surface of the base, to prime and seal with an impermeable surfacing as soon as practicable, to roll it thoroughly over the full width, and to open it to traffic immediately.

This is now simply known as the accelerated construction method.

Damage to a primed base course is repaired by brooming and subsequent damage to the seal seldom results in the absence of a shallow saline water table.

Seawater and salt water of similar composition and salinity can be used for the compaction of all layers of a flexible pavement including a lime-stabilised layer but should not be used for curing such a layer.

Such use should not lead to any significant salt damage during or after construction or to a reduction in the structural capacity provided that certain precautions are taken in the design, construction and maintenance of the facility.

These precautions centre around the widely used accelerated construction method, which essentially involves covering each layer with the next and surfacing the base with an impermeable seal as soon as practicable as described in the Namibia Guide (Roads Authority, 2014), outlined in Section 2.4.5 and summarised as a quick guide in Chapter 5, together with suggested methods of evaluating water for road construction.

6.2 RECOMENDATIONS

The confusion over the test methods to be used for the Botswana guide should be cleared up.

The extent to which the Namibia and Botswana and similar informal guides used by the author have been used and the degree of success achieved should be ascertained.

The long-term performance of the Springbok-Komaggas road which used pumped copper mine water for compaction should be followed up.

Research on the use of pumped mine water, treated acid mine water and treated effluent should be undertaken.

Extracts from this report and any further work should be published in a local civil engineering journal or at a local roads conference if they are to have any impact on local road practice.

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NOTE: Contact details for the older personal communications are no longer available and are probably also now invalid.

APPENDIX A: SELECTED CASE HISTORIES

A.1 STUART HIGHWAY

A.1.1 INTRODUCTION

The 925 km-long Stuart Highway in South Australia completed in 1986 links Port Augusta in the south with Kulgera on the Northern Territory border. Parts of this road in the general vicinity of Woomera appear to have the dual distinction of having what may be both the highest finished salt contents in the world under a thin surfacing and having used the most saline compaction water. It has been described by Foord (1984) and Januszke and Booth (1984, 1992). The traffic on this road at that time was light.

A.1.2 CLIMATE

The climate is BSh (hot dry steppe) for about the first 50 km north of Port Augusta and BWh (hot dry desert) thereafter. The mean annual rainfall is mostly less than 200 mm, together with low humidity and extremely high daytime temperatures (of up to 50°C) and evaporation rates.

A.1.3 PAVEMENT

The type of seal finally adopted over the high salt content bases was a double seal on a primer seal on a "primer" of bitumen emulsion.

Surfacing: Double surface treatment on primerseal:
Class 170 bitumen at 0,9 ℓ/m^2 and crusher sand on
Class 170 bitumen at 1,1 ℓ/m^2 and 14 mm chippings on
Class 170 bitumen at 1,0 ℓ/m^2 and 4 mm chippings.

"Prime": ARS/170 bitumen emulsion at 1,1 ℓ/m^2 (0,7 ℓ/m^2 net bitumen).

Base: 150 mm natural gravel (mostly sandstone or calcrete with sandy loam) compacted to 95% MAASHO. OMC 8-13%, PI 0-9, < 6,7 mm 27-55%, < 2,36 mm 17-34%, < 0,425 mm 8-16%, < 0,075 mm 2-6%, minimum CBR 80

Subbase: (Called selected fill): 150 mm natural gravel, minimum CBR 30.

Width: Total seal width 8 m, edge lined at 0,6 m to give 3,4 m wide carriageways.
Gravel shoulders, each 1,8 m wide. Crossfall 3%.

Note: Australia uses viscosity-graded bitumens: In terms of viscosity a Class 320 bitumen would be 60/70 pen. and Class 170 an 80/100. SP primers are proprietary Mobil cutbacks based on Class 320 bitumen. In terms of viscosity SP60 would be equivalent to MC-30, SP1000 to MC-800 and SP1000V to MC-1800, but in terms of curing properties all appear to be intermediate between MC and SC grades. An ARS/170 emulsion is an anionic, rapid-setting, spray-grade emulsion based on Class 170 bitumen. A primerseal is usually an SP1000 with 2-4 mm grit used with or without a primer as a temporary seal with an expected life of up to one year.

A.1.4 SALT CONTENTS

Base material: 0,3 to 5,3% Total salts (mostly NaCl and gypsum) as determined gravimetrically on a 2:1 water : soil extract on the < 2,00 mm fraction. Equivalent < 6,7 mm EC probably about 0,08-2,8 S/m.

Compaction water: 0,4 to 23% TDS: 0,1 to 7,6% Na+, 0,1 to 13,5% Cl^-, 0,1 to 1,3% SO_4^{2-}; pH 5,9 to 7,6.

The waters were all of the chloride-sulphate type consisting almost entirely of NaCl. Borehole waters were generally < 4% and water obtained from soaks in salt-pans up to 23% TDS. Up to 500 m^3 of water was used per 10 h shift to work an 800 m continuous length of road at a time.

Completed base: Estimated 2 to 3% total salts as determined on a < 2,36 mm saturation extract (estimated < 6,7 mm paste EC 1,5-3,0 S/m). As the water table was mostly deep this was not an additional source of salt to the completed pavement.

A.1.5 PROBLEMS

The problems and experiments on this road have been described in detail by Januszke and Booth (1984, 1992) and only a brief outline and conclusions will be given here.

It was expected that the salt could be contained using the primersealing technique developed on other roads in South Australia where seawater had been used for compaction. This proved not to be the case and a 6,1 km section of primerseal (SP1000 at 0,8 ℓ/m^2 with 4 mm grit) from Bookaloo to Woocalla blistered (up to 10 mm high and 100 mm in diameter), cracked, lifted along the edges and even disintegrated in places, starting after rain 1 to 2 months after sealing in early 1980. A layer of unbound fines containing up to 10% salt was found under the seal and the lower surface of the bitumen appeared “dead” due to absorption by the fines. The worst areas were those which had rapidly absorbed the prime either due to an open surface or an excess of fines. After much experimentation these problems were overcome at least for the duration of the observations reported. The problems reported are typical of those of salt damage and were seen by the author when he visited the Woomera area at the request of the State Highways Department in 1982.

Notable findings included that the salt content in the completed base varied greatly, that the application of a cutback primer increased the salt content in the upper base due to its permeable nature and black-body effect, but that it decreased greatly with time over the nine years of monitoring, especially in the upper 20 mm, apparently due to leaching by rainwater from the shoulders. Salt migration and hence blistering ceased after the moisture contents became very low.

A.1.6 SOLUTIONS

Measures adopted to **prevent** the problem included controlling the salt contents of the materials and waters so as to make use of the best available, reducing the water required during summer by adding and mixing it in at night, not slurrying the surface of the base but exposing as much stone as possible by trafficking if necessary (and accepting a rougher finish), priming and/or primersealing as soon as possible (within 48 h), opening to traffic immediately or rolling with a 10 to 15 ton pneumatic roller until opening, routeing the construction traffic over the full width of the seal, and applying the double seal as soon as practicable (within three months). The primerseal was trafficked or rolled until the double surface treatment was applied.

Damage to the primed base or to the primerseal was generally successfully **repaired** by brooming off all loose material including any visible salt (taking care not to roughen it excessively), repriming with emulsion, primersealing with Class 170 bitumen and surfacing as soon as possible with a double seal. Light damage to the seal was repaired by rolling and resealing where necessary and heavy damage by removal of the surfacing, any visible salt, and the base to the full depth of the powdering, boxing and tacking all potholes and the worst sections, filling with chippings, penetrating with Class 170 bitumen and resealing the full width of road. Resealing with bitumen-rubber may also have to be considered in extreme cases.

Minimisation or prevention of edge damage was achieved by encouraging traffic to use the full width of the seal.

The method of determining the salt content of the base material and finished base course adopted was by drying the saturation extract of the < 2 mm fraction in a microwave oven. This minimizes the effect of gypsum and is more accurate than simply measuring the EC of the saturated paste or the extract but takes much longer.

A.1.7 CONCLUSIONS

With the above precautions the following guidelines can be given as to the lives of different treatments and the allowable construction time constraints (mostly the conclusions of this author)**:**

- **Estimated salt content of 1,7 to 3,3% in the completed base (estimated < 6,7 mm paste EC of 1,5-2,5 S/m)** from 0,3 to 0,5% in material and 6 to 14% in compaction water:

  - A primer of 100:60:40 v/v Class 170 bitumen : diesoline : kerosene (equivalent in viscosity to MC-30 but slower curing) lasted only < 13 h before being damaged. However, by reducing the content of diesoline flux to 100:20:40 (equivalent in viscosity to about an MC-70 but slower curing) it could be made to last the 48 h desired for surfacing. A still later development was apparently to omit the diesoline entirely and to use a primer of 100 Class 170 bitumen : 80 kerosene (equivalent to about an MC-30).

  - An SP1000 (equivalent in viscosity to an MC-800 but slower curing) primerseal, whether single or double, with or without a primer probably only lasted less than one month if untrafficked and one to six months if rolled and trafficked.

  - The best performance by a primerseal (under a double seal) in the 2,5 years of observations was without a conventional primer: all others suffered edge damage within six months.

- **Average salt content of about 2,7% in the completed base (estimated < 6,7 mm paste EC of about 2,0-2,5 S/m)** from 1% average in material and 17 to 23% in compaction water:

  - TO.2 tar (equivalent in viscosity to a 12/15° evt tar) and SP60 (equivalent in viscosity to MC-30 but slower curing) primers were of little value and had to be primersealed in less than one month. SP1000V (equivalent in viscosity to MC-1800) was better, but still lasted for only nine days before damage if not rolled. The blistering could be repaired and prevented by rolling. A TO.2 tar or SP60 followed by an SP1000V primerseal was better still, but could only be made to last six months if trafficked or rolled.

  - The best performance was given by an emulsion "primer" at 1,0 ℓ/m^2 followed by an SP1000 (1,0 ℓ/m^2) primerseal seven days later. This lasted, apparently untrafficked, for the 2,5 months before the double seal was added. The emulsion was not trafficked. It was found that the bitumen distributor could travel on the emulsion in the cool of the morning but later in the day grit was spread sparingly in the wheeltracks. Apart from some bleeding which was regarded as inevitable and acceptable this solution gave entirely satisfactory results in the 18 months service reported.

  - Various modified binder seals on an emulsion "primer" also gave satisfactory performance and it was concluded that 1,5-3% of SBS in the bottom coat and 0-1,5% in the top coat appeared to be sufficient (Januszke and Booth, 1992). However, they were regarded as two expensive for general use.

- In general it would appear that at a probable equivalent < 6,7 mm paste EC of about 1,5-2,5 S/m in a completed gravel base (e.g. calcrete or sandstone) with a fairly coarse finish, an SC/MC-30 or 0/1°

evt tar primer can be expected to last less than 12 hours, an MC-30, SC/MC-70 or 12/15° evt tar primer a few days, and SC/MC-800 or SC/MC-1800 primerseals with or without primers about a week if not continuously rolled and/or trafficked. If continuously rolled and/or trafficked over their full widths they can be made to last a month or more. (In contrast, the **expected** life of a primerseal on this road was about one year.) An anionic spray-grade emulsion at 1,0 ℓ/m^2 used without a primer can be expected to last at least a week and an SC/MC-800 or heavier cutback or hot bitumen primerseal applied on this for at least three months, apparently without extra rolling or trafficking and probably for longer if rolled or trafficked. A double bitumen or bitumen-rubber seal placed on this could be expected to last for at least 18 months and, probably, indefinitely until it becomes permeable due to weathering, cracking or other damage. A primer made by cutting back 80/100 pen. bitumen with kerosene only (i.e. no diesoline) in the ratio 100 bitumen : 80 kerosene (about an MC-30) can apparently also be used successfully in place of the emulsion and reduces the problem of obtaining a good stone mosaic finish on the base without damaging it (Foord, 1984).

On the basis of experience over the previous 25 years on a number of roads affected by salt (mostly sodium chloride) the following treatments were recommended by Januszke and Booth (1992) according to the salt content of the finished base course:

Salt level	Treatment
0-1,5%	As for normal pavement
1,5-2,5%	Primer seal followed as soon as possible by final seal
2,5-3,0%	Sacrificial primed surface swept and followed by a light holding seal and final seal or, if not satisfactory, as for > 3,0%
>3,0%	Emulsion “prime” followed by a C170 light holding seal and double seal

In addition, surface slurrying of fines must be avoided, the “prime” and light holding seal must be applied as soon as possible (but within 48 hours) after completion of the base course, immediately rolled with a 10-15t pneumatic-tyred roller and exposed to as much traffic as possible, the double seal applied as soon as possible (but no longer than three months) with construction traffic routed over its full width. In short, the key factors were a coarse-textured base surface low in fines followed as soon as possible by an impermeable seal and immediate trafficking. Further details are available in a Department of Road Transport (1992) report.

A.1.8 ACKNOWLEDGEMENTS

The writer inspected and advised on sections of this road around Woomera during 1982 in the company of Mr J Potter, Asphalt Engineer, at the request and expense of the South Australian Highways Department and also had discussions with Messrs L Chester, Materials Engineer, R Januszke and E Booth of the Department.

A.1.9 REFERENCES

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A.2 GQEBERHA (PORT ELIZABETH) AREA ROADS

A.2.1 INTRODUCTION

Negative experience with the use of seawater and even more saline water in this area and near Cape Town during 1959 to 1961 led to the then Cape Provincial Roads Department (CPRD) restricting the use of seawater to the subbase and lower layers and the TDS of water for base compaction to 1,2% (D Ackerman, CPRD, 1968, pers. comm.).

The main roads in question all formed part of the old Port Elizabeth-Grahamstown road and employed the same pavement design:

<u>NR2/11 : Swartkops River (Settlers Bridge)-St George's Interchange</u>
Seawater from the Swartkops River was used in the base for this road, apparently only in the Makhanda (Grahamstown)-bound carriageway. Municipal drinking water was used in the other, i.e. apparently the Gqeberha-bound carriageway. This road was completed in 1961 and overlaid in 1973.

<u>NR2/11 : St George's Interchange-Hougham Park</u>
Coega River water with a higher salt content than seawater was used over this section. It was completed in 1957 and resealed in 1966/67.

<u>TR2/16 : Hougham Park-Sundays River (old Mackay Bridge)</u>
Tidal estuarine seawater from the Sundays River at the Mackay Bridge was used for this road. It was completed in 1959 and resealed in 1967.

<u>TR2/16 : Sundays River (old Mackay Bridge)-Colchester</u>
Tidal estuarine seawater from the Mackay Bridge area of the Sundays River was used in this road, which stretches from km 34,1 to 36,4 (km 0 is at the Cadle Street underpass bridge in Gqeberha and the Sundays River Bridge is at km 33,83). This road was completed in 1957, resealed in 1961 with an emulsion and 6 mm chippings, a slurry added in 1962, and resealed again in 1968 with an emulsion tack, 13 mm chippings, an emulsion grout spray and sand.

The traffic when these roads were opened in around 1960 would have been in the range of 500 to 1 000 vehicles per day.

Seawater was also used for the compaction of the calcrete base for the access roads and main streets in the nearby villages of **Colchester** and **Cannonville** constructed in August 1976 (G.H. Webster, Port Elizabeth Divisional Council, 1977, pers. comm.). In all cases the base was sealed with a double seal within two days of completion. When a primer (gas tar) was used it was applied the day after completing the base. The traffic on these roads and streets is light, and limited largely to residential and delivery traffic.

A.2.2 CLIMATE

The area receives a normal annual rainfall of about 400 mm, and classifies as hot dry steppe (BSh), the Moisture Index varies between about -20 at Port Elizabeth to -30 in the area around Colchester and the N-value between 3 and 4. The macroclimate for pavement design purposes is moderate.

A.2.3 PAVEMENT

The pavement design for all sections of the **main roads** was as follows:

Surfacing: 19 mm Old-type Cape seal: crusher dust asphalt instead of slurries.

Primer: Dundee coke oven tar (3/12° evt) at 0,6-0,7 ℓ/m^2, or Gqeberha gasworks tar at similar rates.

Base: Crushed quartzitic sandstone without added soil binder, slushed with seawater. OMC about 6%. (Although it is normal to add binder in the Cape, these sandstones are relatively soft and probably crushed to a grading not requiring binder.)

Subbase: Calcrete.

Width: Probably 6,7 or 7,4 m seal, on a 600 mm wider base course with gravel shoulders. The sections between the Swartkops River and Hougham Park were to dual carriageway standards.

The pavement design for the **Colchester** and **Cannonville** access roads and village streets was as follows:

Surfacing (double surface treatment): 7 mm chippings precoated with tar / cationic emulsion / 13 mm chippings / 65% cationic emulsion at 1,2 l/m^2.

Primer (when used): Gqeberha gasworks tar.

Base: Calcrete, with a MAASHO OMC of 10 to 15%. This would not have been slushed.

Width: Probably 6,7 or 7,4 m seal on a base extending the full width of the shoulders.

A.2.4 SALT CONTENTS

The salt contents are unknown. The salinity of the estuarine water probably varied according to the season, river flow and the tide. However, if it is assumed that the seawater contained a total of 3,5% salt the addition of 6% seawater would have added 0,2% sea salt to what was probably a non-saline crushed stone base material. Depending on the weather approximately up to 10 to 15% seawater could have been added in total, allowing for evaporation and slushing, and more to any section that had to be recompacted for any reason. The total salt content of most of the completed base probably therefore ranged between about 0,35 and 0,5%. If it is assumed that about 50% of the crushed stone base material was finer than 6,7 mm and that all of the salt ended up in this fraction when tested, the average < 6,7 mm EC of the full thickness of completed base must have been between 0,5 and 1,0 S/m. The salt content of the upper 25 mm of base could have been considerably higher due to the slushing process.

The section between St George's Interchange and Hougham Park, which used Coega River water, probably had still higher salt contents.

The salt contents of the calcrete base would have been similar if 10 to 15% seawater was added to a relatively non-saline material. (Unlike the crushed stone it was probably damp or moist on dumping on the road.) However, the calcrete probably had about 80% passing the 6,7 mm sieve and an initial EC of about 0,1-0,2 S/m. The addition of 0,35 to 0,5% sea salt to this material would be expected to have raised the EC after compaction by an amount equivalent to 100/80, i.e. 1,25 times these amounts, i.e. by about 0,5-0,6 S/m to about 0,6-0,8 S/m.

The EC of both the calcrete and crushed stone bases after completion must therefore have been within the range 0,5-1,0 S/m.

A.2.5 PERFORMANCE

The **Sundays River-Colchester** section became loose **before** priming and would not take the tar primer which broke up within eight days or perhaps longer. The ground around about also fluffed up. The loose material was broomed off and surfaced in 1957 with an old-type Cape seal without repriming. However, the surfacing was said “not to have lasted” and in 1975 the maintenance records showed it to have been resealed in 1961, 1962 and 1968.

The northern carriageway of the **Swartkops River-St George's Interchange** section that was compacted with Sundays River seawater, was primed in November 1958 and was fluffed by February 1959. This was broomed off and the section reprimed and surfaced. No subsequent problems were experienced with the surfacing. Why this should be the case seems clear. All those concerned stated that the way to avoid salt damage was to prime and seal as soon as possible. It is therefore probable that the lessons learned on the two earlier sections were successfully applied to these two later sections.

The other sections of this road, i.e. St George's Interchange-Hougham Park-Sundays river gave no problem, even though the latter section was compacted with the Coega River water which was stated to be more saline than seawater.

The experience reported here is consistent with the recorded maintenance history of these roads. The average life expected from an old-type Cape seal was ten years, with twelve years between reseals in the Port Elizabeth Division. Only the Sundays River-Colchester road has therefore required a significantly greater frequency of resealing than normal. When inspected during August 1977 by the writer no signs of salt damage could be found on any of these roads, not even on a one-km stretch from about km 34,1 to 35,0 on the Sundays River-Colchester road, which had been abandoned and lain untrafficked since the realignment of the N2 in about 1970. At least the final double reseal must therefore have been successful in arresting the salt damage.

No significant problems were experienced in the case of the Colchester and Cannonville access roads and streets. In a very few cases less than one percent of the primed area became loose and no problems had been experienced with the seal. No signs of salt damage were evident when the Cannonville road and streets were inspected by the writer in August 1977 and none were subsequently been experienced (G H Webster, 1986, Pers. comm.). The access road (which had received a primer) showed very little (1-2 mm) primer penetration but the seal was nevertheless adhering well. No salt damage to the seal was found even at the end of the dead-end Ruby Street.

A.2.6 CONCLUSIONS

- Seawater and water of apparently greater but unknown salinity was successfully used for the compaction and slushing of graded crushed stone bases without added binder in three out of the five sections of base considered. Both cases of seawater compaction using calcrete as base course were successful.

- In both of the earlier roads the use of seawater led to the fluffing of the tar primer after it stood for some time. In both cases the loose material was broomed off and the roads surfaced with an old-type Cape seal. In the case that was reprimed no damage to the seal took place. In the other case the road was not reprimed and the old-type Cape seal suffered damage which apparently required three reseals in eleven years to cure.

- The later construction both with crushed stone and with calcrete bases, was all entirely successful. It is believed that this was due to priming and sealing as soon as possible, although the season of the year and the weather during construction would also have played a role. In the case of the calcrete it is known that it was sealed within two or three days of completing the base.

- The use of seawater probably added approximately 0,3 to 0,5% sea salts to both the crushed stone and the calcrete base materials and in both cases the < 6,7 mm EC of the completed base was probably between 0,5 and 1,0 S/m.

- The Sundays River-Colchester road appears to be the only known case where an **unprimed** graded crushed stone base (possibly equivalent to a modern G2) became loose due to salt in the base. No soil binder was added in this case.

- The Sundays River-Colchester road, together with a section of the Oshivello-Ondangua road and an airport in Namibia are the only three pavements known to the author in southern Africa in which removal of the loose surface of the primed base was not sufficient to prevent damage to the surfacing.

A.2.7 ACKNOWLEDGEMENTS

Discussions with several of the staff of the then Cape Provincial Roads Department, in particular Messrs KW Puchert, W Lane and RA Appelgryn and a search by the author through the records made it possible to reconstruct this case history. Mr GH Webster of the then Diaz Divisional Council kindly showed the writer over these roads and provided further details.

A.3 MILNERTON-YSTERFONTYN-LANGEBAAN ROAD

A.3.1 INTRODUCTION

This 100 km length of Trunk Road 77 road was completed during 1975 to 1977 for the then Cape Provincial Roads Department and forms the 'west coast road' between Cape Town and Saldanha Bay. Priming problems were experienced on this road even with waters of relatively low salinity (0,15 to 0,8%) and this case history is therefore included since it provides valuable insight into the problem from which a construction methodology has been developed.

Whilst most of the remarks apply specifically to the southern 60 km of the road, similar problems were experienced and solutions adopted for the northern contract.

The road was opened to traffic during 1976 to 1977 and was medium in volume and mass (1 000-10 000 vehicles per day with 10% heavies).

A.3.2 CLIMATE

The normal annual rainfall varies from about 440 mm (falling mostly in winter) at the Milnerton end near Cape Town to 400 mm north at about km 15 north of Milnerton to 270 mm at Langebaan near Saldanha Bay. The climatic classification changes similarly from Csb (warm temperate Mediterranean with a summer dry season) in the south to BSk (dry cold steppe) in the north and the Moisture Index from about -5 to -20 and about -35 near Langebaan. However, with an N-value of about 4 the macroclimate of the whole road is classified as moderate for pavement design purposes.

A.3.3 PAVEMENT (CARRIAGEWAY AND SHOULDERS)

Surfacing: 19 mm Cape seal

Prime: Cape Town gasworks tar or MC-30 prime at 0,7-0,8 ℓ/m^2

Base: 200 mm G3 quality crushed stone base with soil binder in two 100 mm layers compacted to 98% MAASHO. MAASHO OMC 5 to 7%, PI NP-6

Subbase: 150 mm Crushed stone-soil, natural gravel or stabilised natural gravel subbase

Subgrade: 250 mm Gravel/sand

A.3.4 SALT CONTENTS (SOUTHERN CONTRACT)

Base material: Crushed granite and hornfels < 6,7 mm paste EC 0,02-0,18 and binder 0,01-0,52 S/m (0 to 16% added)

Water: 0,15 to 0,8% TDS (10 to 15% added to base for compaction plus 2 to 5% to upper base for slushing)

Completed base before priming: 0,02-0,53 S/m. Salt believed to be mostly NaCl

A.3.5 CONSTRUCTION AND TESTING

Three to fourteen days after compaction the upper base was slush-rolled (slurried) and typically primed the following day.

Salt contents of the materials and the completed layers were monitored throughout construction using the < 6,7 mm paste EC test on all samples taken for grading and Atterberg limit testing.

A.3.6 PROBLEMS

Sections of the primed base between km 9,1 and 11,34 and 15,7 and 17,78 exhibited blistering, powdering and/or general looseness of the upper base to a depth of about 10 mm three to eight weeks after priming with MC-30 during December 1975 to March 1976 (the summer dry season). Over the first section the damage was generally confined to the untrafficked shoulders, particularly the eastern shoulder. Over the second section the damage was generally more severe and more widespread, occurring also in the carriageways. The worst section was that between km 15,7 and 16,78, the portion between km 16,3 and 16,78 being particularly bad. The damage was worst where the primer was oldest and where excess fines from the slushing process had not been removed from the surface of the base. The general finish on the base was inadequate with a good, uniform mosaic not always being present.

Puddles of primer about 4 mm deep had also formed in hollows left by the brooming and slushing. No salt damage had occurred in such places.

Between km 18,72 and 19,500 no salt damage was evident, but salt stains were observed on the tar primer before it was sealed three weeks after priming.

The depth of primer penetration was generally small, possibly due to priming so soon after slushing. (However, priming as soon as possible after completing the base later became recommended practice if salt is present.)

All undamaged areas with conductivities in excess of 0,25 S/m had been primed with Cape gas tar and had been surfaced within two to four weeks, whereas areas with similar salt contents primed with MC-30 failed after two to four weeks. Salt contents of the upper 100 mm of base were higher than expected due to more water being used for compaction and slushing than that expected from the OMC and due to migration from the lower base and subbase, which had similar salt contents. Winter rains reduced upward migration and leached out salts in some cases.

A.3.7 PREVENTATIVE MEASURES

Preventative measures immediately adopted for the rest of the contract were to:

- Stop using the 0,8% salt water from the borehole at km 15,3 (Salt River) and the source of calcareous binder with the high EC;
- Attempt keeping the fraction passing 0,075 mm of the completed base at the lower limit of the envelope of 5% by mass;
- Use only a tar primer if available;
- Omit the primer if the EC of the top 100 mm was above 0,15 S/m or above 0,20 in the top 50 mm after slushing and before priming;

- Tack and chip all completed base and primed base immediately, and
- Complete the surfacing up to and including the first slurry within two weeks if practical.

As an experiment, the primer was omitted on the section between km 19,480 and 19,880. However, the 150/200 pen. hot bitumen appeared to 'ball' due to the dust on the base which could not be removed without damaging the base and the rest of the contract was primed with a tar primer. Sampling at km 19,680 just before tacking and chipping on 12 May 1976 showed the EC of the upper 100 mm base at this point to be 0,70 S/m and that in the upper 50 mm 1,0 S/m. The base at this point had not become loose even though it had stood for seven weeks since compaction and five weeks since slushing. However, this was not during the height of summer.

The EC of the upper 100 mm base was successfully kept below 0,15 S/m over nearly all of the rest of the contract and no further salt damage was experienced. However, four sites were recorded with values of 0,19-0,25 before priming with tar. Binder was used in all cases. The lack of damage can probably be ascribed to the relatively rapid construction, in particular to the sealing of the primed base within about 15 days.

A.3.8 REMEDIAL MEASURES

Slightly damaged sections were simply dry-broomed. The severely damaged section from km 15,7 to 18,7 was slushed with water, rotary-broomed, rolled with a pneumatic roller and opened to traffic for about one month during which time rains leached out the salt from 0,4-0,5 S/m to 0,1-0,2 S/m over the full 200 mm depth of the base. No repriming was carried out, but the tack spray rate of 150/200 pen. bitumen was increased by 0,05 ℓ/m^2. One section with an EC of 0,7 S/m was left unprimed as an experiment, but the tack spray increased by 0,15 ℓ/m^2. The sections were then surfaced.

A.3.9 LONG-TERM PERFORMANCE

The Consultant reported in December 1978 that no salt damage had occurred to the completed surfacing after 21 months under traffic. When inspected by the author in more detail in November 1977 one year after surfacing, occasional slight looseness and blistering of the slurry overspread (the chamfering off of the slurry at the edge of the chippings) was the only damage that could be found anywhere on this contract. Adhesion of the surfacing to the base over the section where the primer had been omitted was good. The whole road was resealed in 1985 and in November 1986 and also in 1992 both the maintenance staff and the author were in agreement that no significant salt damage was present.

A.3.10 CONCLUSIONS

- The Consultant concluded that it was more economic to accept and repair the slight salt damage to the primed base encountered than to avoid the use of the saline materials.
- The Consultant also concluded that most of the damage could have been avoided if, in terms of the specifications, the Contractor could have been required to cover the completed subbase with loose basecourse material, to prime the completed base as soon as possible after compaction and to tack and chip as soon as the primer was dry enough to receive it. In cases where the base could not be surfaced soon after completion it should be possible in terms of the specifications to require the Contractor to omit the prime or to delay it until he is able to start surfacing.
- A relatively low salt content in the slushing water, perhaps as low as 0,15% is sufficient to add enough salt to the upper few mm of the base to cause salt damage to the primer if left long enough. However, 0,8% salt water could be successfully used if the primed base was sealed quickly.
- An analysis of some of the records for the southern contract clearly showed the dependence of the primer damage on both the total salt content of the completed base just before priming and the length of exposure of the primed base (Figure A.3.1). This can probably be used as a guide for construction timing in summer or winter in an arid or semi-arid climate in cases where a fluid cutback such as MC-30 or tar is applied at a rate of about 0,7 ℓ/m^2. Gravels containing much fines may have to be sealed

sooner, whereas a higher application rate, a heavier primer or damp weather may delay the onset of damage.

- A histogram of 131 EC results from undamaged areas and 33 from damaged areas on the northern contract (not shown) did not show such a clear distinction as those on the southern contract: five of the results on damaged areas ranged between 0,06 and 0,10 S/m and 17 between 0,10 and 0,15 S/m, and all damage occurred within 4 weeks. The application of an upper limit of 0,15 S/m as for the southern contract would have resulted in acceptance of about 85% of the undamaged areas at the risk of accepting 14% of damaged areas and rejecting only 1% of good areas.

- Application of a more conservative limit of 0,10 S/m would have resulted in acceptance of only 4% of damaged areas but at the risk of rejecting 13% of good areas.

- Data from the northern contract further suggested that a crushed stone base is liable to become damaged if left long enough when the EC exceeds 0,3 S/m in the upper 15 or 25 mm or 0,15 S/m in the upper 70 mm and that the upper part of a base becomes damaged when its EC **increases** by more than about 0,15 S/m. (Only that depth over which this actually occurs became loose and/or blistered).

- Although this unexpected – and relatively new – form of damage caused great concern at the time it was mostly easily repaired by brooming and only resulted in insignificant damage to the edges of the seal in the long term.

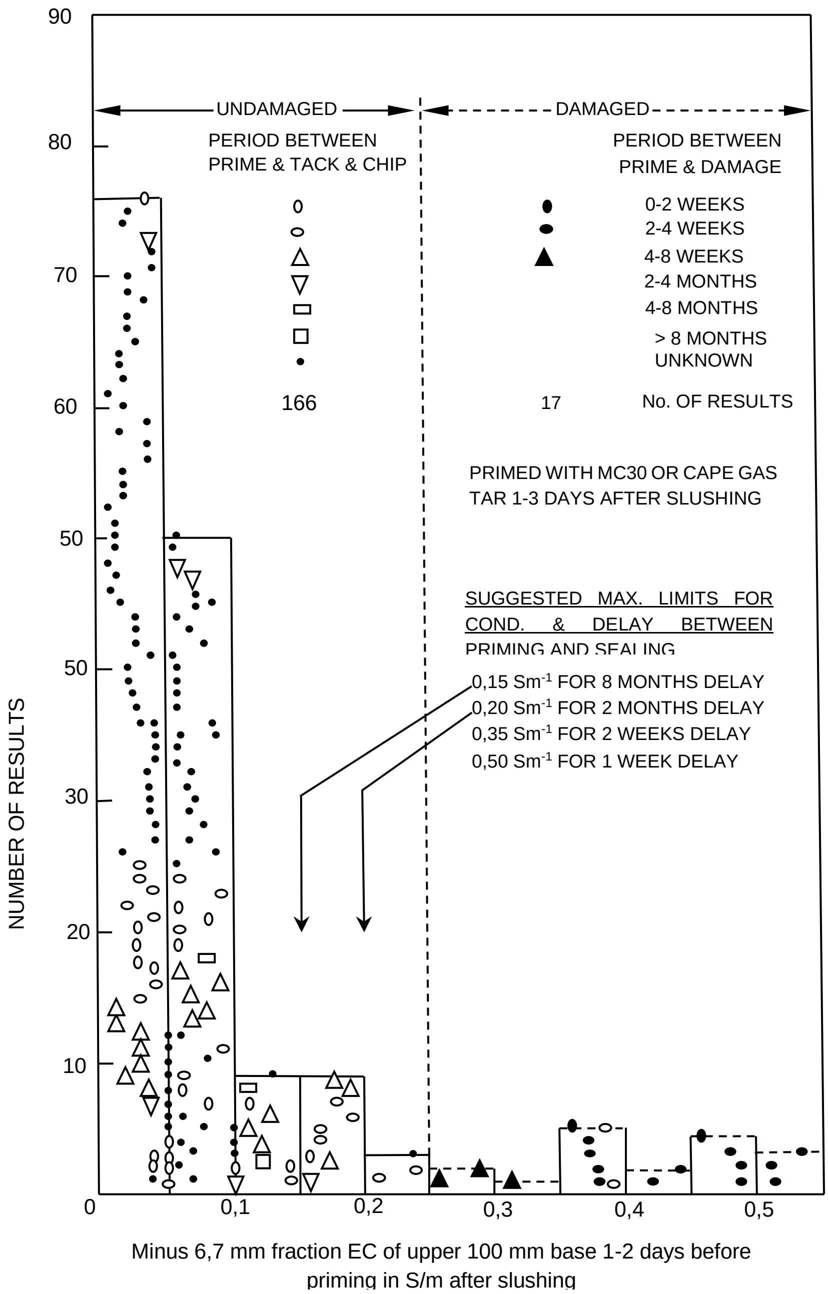

Figure A.3.1. Comparison between paste conductivity of crushed stone base course and prime damage for trunk road 77 Milnerton-Ysterfontein road.

A.3.10 ACKNOWLEDGEMENTS

The very complete records kept by the Consultants, Messrs Jeffares and Green, made it possible to construct a detailed history of this case, of which this is a summary.

A.4 SWARTKLIP EXPERIMENTS

A.4.1 INTRODUCTION

The Swartklip experiments were constructed in 1975 as part of Baden Powell Drive (Main Road 116: Muizenberg-Swartklip-Faure interchange on the N2 (Cape Town-Somerset West) along the northern coast of False Bay in the Western Cape Province. The objectives were to:

- Ascertain whether or not seawater could be used for compaction of (a) graded crushed stone and (b) calcrete bases.
- Ascertain whether or not salt could be prevented from migrating into the base course from the lower layers by priming the subbase.
- Evaluate the structural performance of a finely graded natural gravel calcrete base against that of a graded crushed stone base.
- Study the migration of moisture and salt in road pavements, both during and after construction in order to be able to extrapolate the results of these experiments to other conditions.
- Investigate the importance of surfacing permeability in preventing salt damage.
- Investigate the allowable delay between completing the base and sealing.

As seawater had already been used on several occasions in the subbase and lower layers in the former Cape Province and Namibia without any problems having been reported, its use in these layers was of less concern.

The average traffic carried by the sections as measured with an axle weight analyser is shown in Table A.4.1.

Table A.4.1. Traffic carried by Swartklip experiment

Date	Axles/day		E80/day		% Axles > 80 kN		AADTT (heavy)	
	Left	Right	Left	Right	Left	Right	Both lanes	
Jan/Feb 1977	1 640	1 545	193	310	3	7	1977	250
Nov/Dec 1979	2 124	2 129	427	695	7	9	1979	310
August 1982	2 397 [1]	2 297 [1]	259	228	4	3	1982	420

Note:
[1] 20% Vehicles with GVM ≥ 3 500 kg

An average of 0,15 MESA per year had been carried by the right-hand lane (i.e. Swartklip and Muizenberg-bound) over 10 years, to a cumulative total of about 1,5 MESA up to the end of 1986 since opening to traffic in December 1975

As salt damage usually occurs within the first few years, data for the first 10 years is most relevant. By 1985 the AADTT had increased to 450 and 580 in 1988 and by 2014 the total AADT over the crushed stone section had increased to about 12 000 vpd, with 7% heavy vehicles.

In July 1987 the road was realigned, the SV 200-500 m calcrete and crushed stone control sections being left as an access road to the beach at Monwabisi and carrying only a few, mostly light, vehicles daily. However, this situation was favourable with respect to the long-term evaluation of the performance of a saline base course as salt damage is invariably worst where the traffic is least.

A.4.2 CLIMATE

The normal annual rainfall at the site is between 500 and 600 mm, falling mostly in winter and is “moderate” for the purpose of pavement design (COLTO, 1996). The macroclimate is of the Mediterranean type (Csb), i.e. warm temperate with a summer dry season. Thornthwaite’s Moisture Index is about -5 and Weinert’s (1980) N-value about 2.

A.4.3 PAVEMENT

Surfacing: 13 mm Cape seal: Slurry seal on cationic (65%) emulsion at 0,97-1,06 ℓ/m^2 on 13 mm chippings on cationic (65%) emulsion at 0,94-1,10 ℓ/m^2

Prime: Cape Town gasworks tar (4 mm STV 30-40 secs at 30°C) at 0,7-1,0 ℓ/m^2 or MC-30 primer at 0,7 ℓ/m^2

Base: 150 mm **Crushed hornfels stone** plus binder, or **calcrete**, compacted to 98% MAASHO, 8,0 m wide

Subbase: 150 mm Calcrete compacted to 95%

Selected subgrade: 150 mm Calcrete plus 100 mm calcrete or dune sand compacted to 93%

Subgrade: Dune sand

Shoulders: Calcrete, initially unsurfaced, later sealed with 13 + 7 mm double seal

Surfacing width: 7,4 m wide surfacing on 8,0 m wide base, with 2,4 m wide calcrete gravel shoulders

The sections from SV 0 to 500 were within about 50 m of the sea and their surfacing level about 2 m above high tide level. The other experimental sections were about 3 km from the sea. The possibly brackish, but not particularly saline, water table in this latter area is probably about 2 m below surfacing level in winter and deeper in summer.

A.4.4 MATERIALS

The average laboratory test results on samples removed from the base course after compaction and in-situ DCP CBR measurements are shown in Table A.4.2.

Table A.4.2. Average test results on Swartklip materials after compaction

Stake Value m	Base Course		% Passing by Mass					GM	PI	MAASHO		CBR	
	Material Quality	Water	26,5	6,7	2,0	425	75			Field Comp.	OMC	Lab [1]	Field DCP [2]
			mm	mm	mm	µm	µm		%	%	%	%	%
200-300	Calcrete G6	Sea	96	90	86	70	15	1,29	SP	100	10,6	84	> 100
300-400	Calcrete G6	Fresh	87	80	75	62	13	1,50	SP	99	10,4	120	> 120
400-500	Crushed Stone G2	Fresh	97	45	30	22	8	2,40	5	103	5,8	100	> 100
3300-3600	Crushed Stone G3	Sea	93	50	33	24	9	2,42	5	99	5,5	(120)	(>120)
3700-4000	Crushed Stone G2	Fresh	96	50	34	25	9	2,40	5	101	5,2	(>120)	(> 120)

Notes:

[1] So-called “field CBR”, i.e. equivalent laboratory soaked 2,54 mm CBR at indicated % in-situ compaction. Figures bracketed are estimated

[2] In-situ CBR at in-situ moisture content measured with dynamic cone penetrometer several months after surfacing. Figures bracketed are estimated

The same hornfels graded crushed stone base from Peak Quarries was used over all the sections as well as the rest of the road. A calcified or calcareous sand binder in the ratio 5:1 stone : sand by mass from the calcrete borrow pit was mixed with the stone on the road before adding water. Although the quality of the mix was G2-G3 according to the standards of the time it would probably only be G4 or a marginal G3 according to the later (COLTO, 1998) and the current COTO (2020) standards.

The "calcrete" used for base from SV 200-400 and as subbase for the rest of the road was essentially a calcified sand according to the geotechnical classification of Netterberg and Caiger (1983) and an aeolianite (dunerock) geologically. A lesser amount of pieces of the overlying hardpan calcrete had been ripped, broken down with the bulldozer, grid-rolled and added to the calcified sand in order to coarsen the grading. However, the grading modulus of this material was still only about 1,5 or less and would only have qualified as a G6 according to COLTO (1998) and a G7 according to COTO (2020).

The average < 6,7 mm EC of the calcrete before compaction was 0,08 S/m. The crushed stone had an EC of 0,04-0,08 S/m and the binder 0,04-0,06 S/m. The mixture as used in the fresh water crushed stone base sections **before adding water** had an EC of 0,06-0,08 S/m. For no apparent reason that used in the seawater crushed stone base section SV 3 300-3 600 had an average EC of 0,14 S/m. Some contamination with salt from the underlying subbase compacted with seawater presumably took place even during supposedly dry mixing – it had a mean EC of 0,43 S/m (n = 6) in comparison with the 0,30 of the adjacent SV 3 000-3 300 sections.

The seawater was drawn from the rocks at the beach at SV 0 and the fresh water used for the base of sections 3 000-3 280 and 3 700-4 000 from the Steenbras Dam pipeline. The fresh water used for the base of sections 300-500 was also supposed to have been drawn from this pipeline. The average composition of these waters is shown in Table A.4.3. The marsh water used over most of the rest of the road was not tested but was regarded as fresh and was being used as drinking water for cattle, implying a probable maximum TDS of about 6 000 mg/***ℓ***, i.e. 0,6% (Bond, 1946; DWAF, 1996).

Table A.4.3. Composition of compaction waters used in Swartklip experiment

Components	Units	Seawater	Fresh Water
Na^+	% m/v	1,086-1,087	0,00135-0,00136
Cl^-	% m/v	1,975-1,978	0,0022-0,0023
SO_4^{2-}	% m/v	0,274	0,0023
TDS at 105°C	% m/v	3,48-3,59	0,011
EC at 25°C	S/m	5,00-5,20	0,0185
Salinity (from EC) [1]	%	3,3-3,4% m/m	0,012% m/v
pH		7,9	8,2-8,4

Note:

[1] By NITRR method CA21-74 (1980)

The TDS of 3,5% and composition of the seawater was that of average open ocean seawater and the TDS of 110 mg/ℓ of the fresh water was that of good drinking water. The approximately 10 and 5% of seawater added for compaction on the calcrete and crushed stone sections respectively would have been expected to add about 0,35% and 0,18% salt respectively to the completed base courses. Assuming that all this salt ended up in the < 6,7 mm fraction that was actually tested (i.e. 90% of the calcrete and 50% of the crushed stone), about 0,4 and 0,35% salt would actually have been added to this fraction in the case of the calcrete and the crushed stone, respectively. From Figure 6 of NITRR Method CA21-74 (1980) this would have been expected to **increase** the EC of the calcrete from 0,08 S/m by 0,45 S/m to about 0,5-0,6 S/m. The EC of the crushed stone-binder mixture would similarly be expected to have been increased from about 0,08 S/m by about 0,42 S/m to about 0,5 S/m. In other words the expected EC of both the completed calcrete and crushed stone bases was about the same at about 0,5 S/m.

A.4.5 EXPERIMENTAL LAYOUT, CONSTRUCTION TIMING AND SALT CONTENTS

Details of the layout, construction timing, average salt contents and the performance of the Swartklip experimental sections and most of the rest of the road are shown in Table A.4.4. It was intended that the experimental sections should all be built during the summer dry season as this was when salt damage was most likely to occur. It was also intended that one lane should be primed and sealed as soon as possible and that the other should be left as long as convenient in order to simulate a practical means of preventing salt damage in the first case and normal construction practice in the second. Unfortunately, the construction schedules were such that the seawater crushed stone section was only compacted in autumn near the start of the winter rainy season and that the lanes of the calcrete section could not be treated separately.

This table also shows that the measured ECs of the sections are in good agreement with those estimated. Although fresh water was supposed to have been used for compaction of all the **experimental** sections, the average EC of 0,13 S/m after compaction for the fresh water calcrete base suggests that either brackish marsh water was used or that the fresh water had become contaminated with seawater.

A.4.6 PERFORMANCE DURING AND SOON AFTER CONSTRUCTION

The performance of the experiments was monitored by regular inspections of a panel consisting of representatives of the then Divisional Council Roads Department, the then Cape Provincial Roads Department, and the author.

The primed **calcrete base** was undamaged when primed on November 15, 1975 even though the salt content of the upper 25 mm of the seawater section had increased from 0,5 after compaction to 0,9 S/m at the time of priming 13 days later (Table A.4.4). The average moisture content of the upper 25 mm of the base was 8,0% on the seawater section (n = 4) and 6,1% on the fresh water section (n = 3), i.e. substantially less than the OMC of 10,5%. However, within seven to ten days of priming **both** the seawater **and the fresh water** sections had begun to blister, flake and to “powder” to an average depth of 1-3 mm (Figure A.4.1.).

Table A.4.4. Layout, construction timing, salt contents and performance of the Swartklip experiments and the rest of Main Road 116 (SV 20-4 220 m)

Stake Value	Base Course			Delay between				Season before sealing	Average < 6,7 mm EC of whole base and upper 25 mm / rest of base in S/m (*n* = 3-6)						Prime damage before sealing [5] % Area	Prime [2] overspray [6] damage after sealing % Area
m	Material	Water [1] base/rest	Date completed	Compaction & priming [2] Lane		Priming & sealing Lane			Before watering & compaction	After Compaction [4]	Before priming Lane		After sealing Lane			
				Left	Right	Left	Right				Left	Right	Left	Right		
200-300	Calcrete	Sea/sea	20/10/75	26 d		13 d		Dry	0,08	0,5	0,5		0,5			
										0,5/0,4	0,9/0,4		0,7/0.4		80 in 13d	100 in 2 mo.
300-400	Calcrete	Fresh/sea	20/10/75	26 d		13 d		Dry	0,08	0,13	0,2		0,3			
										0,15/0,12	0,2/0,2		0,4/0,3		40 in 13d	100 in 2 mo.
400-500	Crushed	Fresh/sea	±20/10/75	±26 d		13 d		Dry	(0.08)	(> 0,08)	-		-		0	80 in 2 mo.
500-3000	Crushed	Marsh/ or sea	-	-		-		Dry-wet	-	-	-		-		-	-
3000-3200	Crushed	Fresh/sea	24/6/75	6 w		3 mo.		Wet-dry	(0.08)	0,06	0,10		0,20			
										0,05/0,06	0,9/0,10		0,25/0,20		0 in 2 mo.	100 in 1 y
3200-3280 [7]	Crushed	Fresh/sea	24/6/75	6 w		3 mo.		Wet-dry	(0,08)	(0,08)	-				< 1 in 2 mo.	100 in 1 y
3280-3300	Crushed	Sea/sea	22/4/75 [8]	7 w	3,5 mo.	3 mo.	3 mo.	Dry-wet-dry	(0,14)	(0,5)	-		0,16		1 in 2 mo.	100 in 1 y
													0,18/0,16			
3300-3600	Crushed	Sea/sea	22/4/75	6 w	1 d	15 d	9 d	Dry-wet	0,14	0,5	0,3	0,5	0,3	0,3		
										0,8/0,4	0,4 / 0,3	1,0 / 0,4	0,3 / 0,3	0,4/ 0,4	0	100 in 1 y
3600-3700	Crushed	-	±22/4/75	-	-	15 d	9 d	Dry-wet	-	-	-	-	-	-	0	(30-100 in 1 y)
3700-4000	Crushed	Fresh/fresh	±22/4/75	6 w	1 d	15 d	9 d	Dry-wet	0,08	(0,08)	0,06	0,07	0,05	-		
											0,07 / 0,05	0,15 / 0,06	0,05 / 0,05	-	0	30 in 1 y. 50% in 2 y
4000-4220	Crushed	Marsh/ marsh	-	-	-	-	-	Dry-wet	(0,08)	(>0,08)	-	-	-	-	0	> 50 in 1 y

Notes to Table A.4.4:

[1] Seawater used to top of subbase for all experimental sections (embolded) except for 3 700-4 000 for which fresh water was used in all layers. Transition zones such as the major one at 3 600-3 700 allowed for between all sections. Marsh water drinkable by cattle was used at least in the base over most of the rest of the road

[2] All primer gas tar at 0,74-1,0 ℓ/m² except for 3 000-3 300 (and possibly 500-3 000 and 4 000-4 220) for which MC-30 at 0,74 ℓ/m² was used

[3] Average values are weighted averages of several results for full thickness of base calculated from the results for the top 25 mm and rest of base. Figures in brackets are estimated. Base was sampled within one day of the construction shown, and usually the same day

[4] Subbase after compaction of base (*n* = 5 or 6): SV 200-300: 0,28; SV 300-400: 0,20; SV 400-500: not recorded; SV 3 000-3 200: 0,30; SV 3 200-3 280: 0,30; 3 280-3 300: not recorded; 3 300-3 600: 0,43

[5] Over sections 200-400 blistering and powdering of the primed base to an average depth of 2-3 mm started within seven days and by 13 days had increased to the area shown. The damage on the other sections shown was confined to the outer 100 mm of the 8 m wide primed base. Over the left lane of 3 275-3 300, 100% of this outer strip was damaged, i.e. about 1% of the whole primed area

[6] No damage occurred to those parts of the overspray that had been unintentionally tacked and sealed

[7] Subbase primed with gas tar at 0,74 ℓ/m² on 26/5/75

[8] Left lane 09/6/75

Figure A.4.1. Prime damage on seawater calcrete base

The primer itself was "dry" and had penetrated to a depth of 4-5 mm. Most of the disintegration took place where the base was more finely graded and softish. On the harder areas there was practically no distress. After 13 days some 80% of the seawater section and 40% of the fresh water section were affected. The bases were then broomed (Figure A.4.2.), after which they appeared reasonable and were therefore tacked and chipped without repriming on November, 28 1975, the top spray and the slurry of the Cape seal following within the next five days. During slurrying on December, 03 1975 it was noticed that the edges of the seal had bonded poorly to the base.

After two months under traffic the seal had developed a good bond, but the primer overspray (i.e. the primer on the 300 mm of base exposed at each edge of the seal) had disappeared due to the salt action. The inspection panel considered that this was not important. However, the riding quality had been slightly affected by the salt damage to the primed base and/or the poor bond.

Figure A.4.2. After brooming some of the flaking prime on the seawater calcrete base

The results suggest that **a finely graded calcrete base compacted with seawater** with a < 6,7 mm EC of 0,5 S/m should be **sealed within seven days** in hot dry weather if primer damage is to be prevented. After this, damage is likely. Whether the damage was initiated by the shower of rain that fell just before the damage was noticed or whether it would have occurred anyway is uncertain. As the base had been left exposed in an unprimed state for nearly a month during the dry heat of early summer it must have dried out substantially (as suggested by the ratio of top 25 mm EC to the rest of the base of 0,9/0,4). It seems therefore that damage to an **unprimed** base is not inevitable and that the priming and/or the shower of rain were implicated. It has been observed on a number of roads that salt damage to a primed base occurred soon after rain when this was followed by dry weather. It is suggested that the rain penetrates the primed layer and dissolves the salt in the upper base. When the water evaporates, the salt crystallises and the crystals disrupt the upper base. Why damage did not occur before priming due to the substantial rise in the salt content of the upper 25 mm of base is not clear. Although loosening of the upper part of a gravel base before priming is known (but apparently not a crushed stone base), it is not common and salt damage has repeatedly been observed soon after priming. The priming process may therefore aggravate or precipitate the problem, possibly by means of the dampening water, the reduction of the percentage voids, the increase in temperature due to the black surface, a reduction in solubility of the salt by the organic solvent in the primer, and/or by one or more of these factors promoting the growth of whiskers instead of normal crystals.

Why the **fresh water calcrete** should have suffered damage is also not clear. The EC of 0,13 S/m after compaction (Table A.4.4) suggests that brackish marsh water or fresh water contaminated with seawater might have been used instead of the fresh water specified. Alternatively, the base might have become contaminated with salt from the subbase during construction. However, the mean EC of 0,20 S/m of the subbase was less than that the seawater section (0,28 S/m). The general increase in the average EC of the whole base together with the small EC ratios suggest some further addition of salt to the whole base both during exposure before priming and before sealing. The only way this additional salt could have got into the lower base was by migration from the seawater subbase. In contrast, the average EC of both the whole thickness of base and of the lower base of the seawater calcrete section remained constant at 0,5 and 0,4 S/m respectively, but the EC ratios suggest either upward migration of salt **within** the layer and/or addition of salt as sea spray from the sea only 50 m away. The upper base Ecs of both sections before brooming off the powdery surface were probably considerably higher than those found immediately after surfacing.

The performance of the **crushed stone** sections was quite different and no significant damage occurred to the primed base of the seawater section even with a combined delay between compaction and sealing of up to six months. However, some of this period was during the winter rainy season and the data in Table A.4.4 clearly indicate that salt was lost from unsealed bases during this period. The right (Swartklip-bound) base of some of the sections from SV 3 280-4 000 m was primed and surfaced as soon as possible, while the left (Faure-bound) lane was left unprimed for six weeks and the primed base then left for a further 15 days before sealing. In neither case did any damage to the primed base take place. However, some light rain did fall during this period and it was also misty at times.

The large drop in the upper base EC from 0,8 to 0,3-0,4 S/m between compaction and surfacing both lanes and even between compaction and priming the right lane suggests that this rain leached out some of the salt from the upper base. In contrast, the EC of the rest of the base remained almost constant at 0,3-0,4 S/m but the average value for the whole base did drop slightly from 0,5 to 0,3 S/m. The adjacent short untested section 3 280-3 300 that was left unsealed for three months did suffer slight damage to the outer edge of the primed base within two months.

The data suggest therefore that a **crushed stone base compacted with seawater** with an EC of about 0,5 S/m after compaction should not be left unprimed for more than about six weeks nor the primed base left exposed for more than about 15 days to perhaps a month during warm dry to slightly rainy weather. Furthermore, leaving it unsealed during the rainy season is also apparently a good way of removing salt from a base, whether primed or unprimed, and any salt damage can be expected to be negligible. However, the performance of the calcrete sections (as well as experience on other roads) suggests that salt damage is very likely after light rain (presumably insufficient to **remove** the salt) followed by hot dry weather.

Although no or negligible damage to the primed base occurred on any of the crushed stone sections before sealing, the primed '**overspray**' on the 300 mm of untacked base exposed at each edge of the seal started to blister and powder within a few months and had largely disappeared within a year on all sections, **even on those employing seawater only up to the top of the subbase** (Table A.4.4). This was most rapid in the case of section 400-500, over which 80% of the tar disappeared within two months, possibly due to this being in the height of the dry summer season. The control section 3 700-4 000 for which only fresh drinking water (Table A.4.2) was supposed to have been used in all layers suffered much less damage, losing only 30% of the tar overspray in one year and 50% in two years. Both tar and MC30 suffered similar damage. The inspection panel considered the loss of the overspray acceptable and inevitable in the long run even in the absence of salt. (General South African policy is not to build this type of boxed-in base with exposed edges anymore, but either to seal the full width of base and to provide an edge line 300 mm from the edge or, in the case of higher class roads, to carry the base across the shoulder and to seal it.)

Why the **primer overspray** should suffer damage on the sections that used fresh water in the base is not clear. Possible sources of salt include migration from the subbase and/or the shoulder in the case of those for which seawater was used in these layers, contamination with shoulder material compacted with seawater, and the possible use of brackish water where fresh water was specified. It is not clear what water was used in the shoulders, whereas the loss of some overspray on the section 4 000-4 220 outside the experiment for which marsh water was definitely used suggests that this may also have played a role. The water used in the shoulders, which were built before the base and more or less concurrently with the subbase, was probably the same as that used in the subbase, i.e. seawater in the case of all the **experimental** sections except for 3 700-4 000. In the case of the seawater base sections it therefore is virtually certain that the water used in the shoulders also must have been seawater.

After the **deliberate delays** in order to simulate normal practice on a large job the effect of priming the seawater subbase on Subsection SV 3 200-3 280 was only to reduce the mean EC of the freshwater base when sealed to 0,16 S/m in comparison with the 0,20 S/m of Section 3 000-3 200 with the unprimed seawater subbase. The EC of both the bases was 0,06-0,08 S/m and the subbase 0,30 S/m after compaction.

Although it might be supposed that the reason why the **seawater calcrete base** became damaged while the seawater crushed stone base did not was simply because twice as much seawater had to be added to the calcrete base for compaction, the measured EC of 0,5 S/m (of the minus 6,7 mm fraction) of both was the same (Table A.4.4.). It is therefore likely that the much finer grading of the calcrete (Table A.4.2.) with its much greater capillarity, must also have been a factor – and probably the major one.

A.4.7 LONG-TERM PERFORMANCE

Apart from the damage to the primed base on the seawater and fresh water calcrete sections during construction, and to the primer overspray on all the sections to a greater or lesser degree after sealing, no further damage occurred to any of the sections over the first 20 years that can definitely be ascribed to salt (Figures A.4.3. and 4).

Figure A.4.3. View towards Muizenberg of SV 200-300 seawater calcrete base section after 19 years in 1994

Figure A.4.4. View towards Faure interchange of seawater crushed stone base section after 19 years in 1994. Section SV 3 300 marker plate on centreline placed in1975 shown ringed

Fretting of the edges of the seal on the inside of the superelevated curve on the seawater calcrete section and some erosion of the calcrete shoulder there did take place, but was considered by the panel to be no worse than that which would have been expected had crushed stone and fresh water in all layers been used.

In order to simulate cracks and holes in the surfacing slots were cut and holes 3 mm in diameter punched through the surfacing soon after sealing. However, these closed up within a year due to the very rich surfacing and no salt damage occurred.

A large number of in-situ air permeability measurements in the outer wheelpaths from SV 3 300 to 4 000 made with a Soiltest Asphalt Paving Meter at a 6 mm head of water pressure showed the average surfacing permeability to be about 0,5 ml/cm^2/minute with all except four values being below 0,8 mℓ/cm^2/min. With the exception of these four values, which ranged between 2,5 and 3,0, all these values are far less than the upper limit of 1,7 ml/cm^2/min. for surfacing permeability suggested by the work of Netterberg (1979) on the Witwatersrand as necessary to prevent salt damage to pavements with excess salt in the base course. In-situ air permeability measurements were repeated two years after construction on a number of spots judged visually to be of the greatest permeability. In all cases the surfacing was found to be completely impermeable to air, even at a 20 mm head of water (the highest practical with this instrument).

The performance of the calcrete shoulders was not recorded in detail. However, no unusual looseness was noticed such as had occurred on certain other roads with high salt contents in the shoulders.

A.4.8 MAINTENANCE

The shoulders were later sealed with a 13 + 7 mm double seal due to Divisional Council policy. All the patches present represented sampling holes and minor patching due to bleeding and surfacing picking up under traffic when new (due to the excessively rich seal) and to minor mechanical damage. The carriageway (not shoulders) was resealed with a 9 mm chip and spray (no fog spray) in February 1983 due to bleeding. It is understood that the calcrete bases and their control section next to the beach (SV 0-500 m) were not resealed after having been cut off from the main road in 1987.

A.4.9 PAVEMENT EVALUATIONS AND INSPECTIONS (1977-1992)

A.4.9.1 Calcrete versus crushed stone bases (SV 200-500 m)

Lacroix deflection in Sept. 1981: Average 0,2 mm calcrete (sea and fresh water); 0,3 mm crushed stone (fresh water). No difference between seawater and fresh water calcrete sections

Radius of curvature: Average 600 m seawater calcrete; 700 m fresh water calcrete; 350 m crushed stone

Rut depth in Sept.1981: Average 5 mm, max. 10 mm on all sections (same in April 1985 and 2009).

Riding quality (PSI) in 1979: Seawater calcrete 2,4; freshwater calcrete 2,3; crushed stone 2,5. (Same PSR in April 1985)

The pavement evaluations and visual panel inspections carried out up to 1992 showed that there was no significant deterioration in surfacing condition, deflection, rut depth or riding quality over SV 200-500 m since construction, and also that there was little significant difference between the performance and the predicted residual structural capacity of the seawater and the fresh water calcrete sections. The deflections and especially the radii of curvature showed the calcrete bases to be stronger than the crushed stone. After 17 years in 1992 the overall pavement condition of all three sections was rated as good. These conclusions were confirmed by several subsequent visual inspections carried out by the author up to 2019.

A.4.9.2 Seawater versus freshwater crushed stone bases (SV 3 000-4 000)

Visual inspections by the panel up to 1992 (i.e. up to 17 years) and up to 2019 by the author found that there was no significant salt damage on any section, no significant difference between the performance of the sections on which seawater had been used and those on which fresh water had been used, and that all sections were still in a good condition.

A.4.10 CONCLUSIONS

- It was agreed by the inspection panel that seawater could be used for base compaction under controlled conditions for both calcrete and for crushed stone bases.
- Under controlled conditions the only damage that would be likely would be the loss of the primer overspray within a year and possibly some edge fretting and looseness at the edge of the slurry even if seawater was used only in the subbase, lower layers and the shoulders. The panel considered this to be acceptable.
- Considerable salt migration is possible within a finely graded (GM about 1,5) calcrete base and from the subbase if the base is left exposed in hot dry weather. In 26 days the EC of the upper 25 mm of base compacted with seawater doubled, apparently due to concentration within the layer. Over the same period the EC of the full thickness of base compacted with fresh water increased from 0,13 S/m after compaction to 0,2 S/m. Over a further 13 days, during which time the primer was damaged, this rose further to more than 0,3 S/m.
- The delay allowable during hot dry weather between compacting and priming a finely graded (GM about 1,5 or less) calcrete base with seawater with a compacted EC of up to and including about 0,5 S/m is at least 26 days. During this period no looseness of the upper base would be expected even if the EC of the upper 25 mm should rise to 0,9 S/m.

- The allowable delay during hot dry weather between priming and sealing a similar calcrete base, whether seawater is used in the base or only up to the top of the subbase, with an EC of 0,2-0,5 S/m before priming, is about seven days or less for a gas tar primer.
- Rain falling on such a primed base is likely to lead to immediate salt damage.
- The best procedure for such a base is therefore to rather leave it unprimed and then prime and seal thereafter as soon as possible, omitting the dampening water spray if possible.
- The delay allowable during warm dry to misty or light rainy weather for a G2 or G3 graded crushed stone base between compaction with seawater and priming with gas tar and with an EC after compaction of up to about 0,5 S/m after compaction is six weeks to 3,5 months or more. Under such conditions the EC of the upper 25 mm of base would be expected to decrease substantially and the EC of the full thickness of base to decrease slightly. Under these conditions the base would not become loose.
- The delay allowable during hot dry weather for a similar base cannot be established from the results of the experiment, but can be expected to be less.
- The allowable delay in warm dry to slightly rainy weather between priming with gas tar and sealing a graded crushed stone base with binder with an EC of about 0,5 S/m after compaction and before priming (and/or an EC of the upper 25 mm of base of up to 1,0 S/m) is nine days or more. For one with an EC of 0,3 S/m before priming (and/or an upper 25 mm EC of up to perhaps 0,5 S/m) this can be extended to 15 days to perhaps two months. Any primer damage at two months is likely to be slight during the wet winter. The delay allowable during hot dry weather between priming and sealing a similar base cannot be established from the results of this experiment, but can be expected to be less, i.e. possibly less than nine days and less than 15 days to two months respectively.
- A longer delay is permissible between compaction and priming than between priming and sealing for both calcrete and crushed stone.
- These conclusions may be somewhat optimistic owing to the cool misty and drizzly weather experienced during part of the construction period.
- In order to minimise upward migration during construction the base course should be primed and sealed as soon as practicable.
- Priming a seawater subbase at the normal rate of about 0,74 l/m^2 will have little effect on migration of salt into a freshwater base.
- The surfacing used on this road was very impermeable with nearly all values being less than 0,8 $m\ell/cm^2$/minute soon after construction and dropping to zero within two years. These values were nearly all well below the limit of 1,7 $m\ell/cm^2$/min. above which salt damage was likely.

A.4.11 ACKNOWLEDGEMENTS

These experiments were constructed to the author's design by the Divisional Council of the Cape on behalf of the Cape Provincial Roads Department. Messrs B van Dalen of the Council and Messrs R A Appelgryn, J P Nothnagel and H W Dempers of the Province and the author were involved in the construction and initial monitoring. Subsequent members of the inspection panel included Messrs B van Dalen, WJ Biesenbach, S Streicher, E J Kitley, AP Mouton, G Pike, H Prodgers, L Wessels and the author.

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A.5 LAMBERT'S BAY EXPERIMENT

A.5.1 INTRODUCTION

The seawater crushed stone sections of the Swartklip experiment were constructed during autumn, i.e. towards the start of the Cape rainy season, and not under the more severe conditions of the hot dry summer or in a desert climate. The opportunity was therefore taken to accept as an experiment a 300 m-long section of crushed stone base compacted accidentally with seawater on the eastern approach embankment ramp to the road over rail bridge No. 22 near Lambert's Bay, at km 59,6 on Trunk Road 55/1 : Clanwilliam-Lambert's Bay. The bridge was opened to road traffic on 15 September 1975.

The traffic on this section in 1978 was about 350 vehicles per day with 10% heavy vehicles, increasing to about 500 vehicles per day in 1985.

A.5.2 CLIMATE

The macroclimate is classified as dry for pavement design purposes and as cold dry desert (BWk) with frequent fog, receives a normal annual rainfall of 123 mm, falling mostly in winter, has an estimated Moisture Index of about -20 and an N-value of about 7,5.

A.5.3 PAVEMENT (CARRIAGEWAY AND SHOULDERS)

Surfacing: 19 mm New-type Cape seal with two slurries, tacked and chipped 28 July 1975, second spray 31 July 1975, first slurry 14 August 1975, second slurry 22 November 1975.

Prime: MSP1 (invert cationic emulsion) on 22 July 1975.

Base: 150 mm graded crushed hornfels or granite without added soil binder of almost G2 quality. Maximum size 38 mm, 75 to 90% < 19 mm, 65 to 81% < 13,2 mm, 43 to 51% < 4,75 mm, 29 to 35% < 2,00 mm, 15 to 21% < 0,425 mm, 6 to 9% < 0,075 mm, PI 2-4, MAASHO OMC about 6%, compacted on 25 March 1975.

Subbase: Gravel, 66 to 82% < 4,75 mm, PI 10-12, completed 3 March 1975.

Subgrade: Embankment ramp, completed 31 January 1975.

Width: 6,7 m wide seal plus 1,5 m wide sealed shoulders.

A.5.4 SALT CONTENTS

The salt contents of the completed base course of the east ramp are shown in Table A.5.1. No salt tests were carried out on the materials or the seawater used. However, the salinity of the seawater along this coast is reported to be normal, i.e. about 3,5%. Qualitative testing of the samples for the EC tests showed all of them to contain only chloride (assumed to be NaCl) without even a trace of sulfate and the pHs did not indicate the presence of any unusual salts such as Na_2CO_3.

The data show that the salt content of the upper base increased during exposure primarily due to migration within the layer, until it had reached a value a few days before priming of about double that of the rest of the base when primed. By the time of surfacing it may have dropped somewhat presumably due to brooming and/or rain. However, the upper base results after surfacing may also be low due to material adhering to the bottom of the surfacing not having been added to the sample of upper base. The rest of the base and the whole base remained roughly constant at about 0,5 S/m suggesting that there was little migration from the lower layers, but that some salt may have been lost.

Table A.5.1 Salt contents (EC) and water contents (*n* = 3) of eastern ramp of base course

Depth	1975-04-28		1975-07-17		1975-10-27	
	4 Weeks after compaction		5 Days before priming		3 Months after tacking	
	< 6,7 mm EC	Water content	< 6,7 mm EC	Water content	< 6,7 mm EC	Water content
mm	S/m	**%**	S/m	**%**	S/m	**%**
0-25	0,5-0,8	1,7-2,5	0,71-0,99	2,4-3,5	0,16-0,66	2,5-3,2
25-160	0,5	1,6-3,4	0,460,50	2,5-2,9	0,14-0,46	2,5-3,2
0-160	0,55 [1]	-	0,50-0,61 [1]	-	0,14-0,49 [1]	-

Note:

[1] Weighted mean calculated from above results

Assuming the base to have been initially non-saline, the addition of 8% seawater for compaction, evaporation and slushing would have added about 0,28% salt to the whole base or twice this to the < 6,7 mm fraction if it is assumed that it all ends up there. This theoretical EC of about 0,6 S/m agrees well with the average values of 0,5-0,6 found.

For comparison, the ECs of the western ramp after compaction and slushing with fresh water on July 21, 1975 were 0,15-0,19 S/m in the upper 25 mm and 0,13-0,24 S/m in the lower base (n = 3), with water contents of 2,7-3,2% and pHs of 8,2-8,6 (n = 6).

Samples taken in January 1976 from the eastern ramp six months after tacking and chipping yielded upper base ECs of 0,43-0,70 S/m, all approximately 0,1 S/m higher than the 0,34-0,53 S/m in the rest of the base (n = 3), suggesting that significant upward migration of salt had taken place even under a supposedly impermeable seal.

However, this hypothesis was not supported by the final round of sampling on May 19, 1976, i.e. about 14 months after compaction on March 25, 1975 and 10 months after tacking and chipping on July 28, 1975. The ECs of the upper 25 mm of base were found to be 0,42-0,59 with 0,40-0,57 in the rest of the base (n = 3), with water contents of 2,2-3,0% and pHs of 7,1-7,6 (n = 6). The salt and water contents about one year after construction were therefore still approximately the same as those when constructed.

A.5.5 PROBLEMS

The base course of both approach ramps was originally completed on 25 March 1975 and the west (Lambert's Bay side) ramp primed with MSP1 two days later. Within one month the primed base became loose and powdery in places to a depth of about 10 mm. During this period a total of 16 mm of rain fell, spread out over a period of four days. On 28 April 1975 the EC of the upper 25 mm of the base of the primed western ramp varied between 0,4 and 0,6 S/m and that of the unprimed eastern ramp 0,6-0,8 S/m. The base of the unprimed ramp had not become significantly loose.

The base of the western ramp was rejected because of the above as well as poor mixing, inadequate thickness, and an inadequate subbase, and both layers were reconstructed.

The base of the eastern ramp was accepted as an experiment. It was later broomed, primed on 22 July 1975 and surfaced six days later. It was not rerolled or reslushed and did not become loose. The primer did not blister or powder during the six days between priming and tacking.

A.5.6 LONG-TERM PERFORMANCE

Within six months some slight blistering and looseness of areas where the primer had been oversprayed and where the slurry had been overspread outside the seal had taken place. (The slurry overspread is that part of the slurry, up to 100 mm in width, which chamfers off the edge of the seal.)

At a few places elsewhere on this road (built in 1970), minor blistering and cracking of the slurry seal on the shoulders was noticed in 1975. Sampling of two sites at km 44 showed the base under the undamaged slurry to have an EC of 0,16 S/m and that under a cracked blister 0,28 S/m. It is therefore clear that a primer alone or a slurry seal alone is insufficient for a base compacted with seawater, and probably for any base with an EC of more than 0,15-0,20 S/m.

Some bleeding of the seal was, however, noticed, especially on the eastern ramp. Whether this was due to punching of the chips into a soft base possibly due to loosening by salt was never investigated. However, many Cape seals constructed at this time were giving bleeding and fattiness problems due to the design spray rates being somewhat high.

This experiment was inspected visually by the author either alone or together with a member of the Roads Department in 1975, 1976, 1977 and November 1986 and no salt damage to the seal ever took place. This was in spite of the chippings used being contaminated with about 1% of shale. These inferior chips weathered out within a year to form 20 mm-diameter holes either empty or filled with clay. The clay did not taste salty. It was not established whether the holes penetrated the bottom spray, but it was assumed that they did.

The whole road (the rest of it had been completed in 1970) received a 7 mm reseal in 1982.

A.5.7 CONCLUSIONS

- Seawater can be used in a cold desert climate for the compaction and slushing of a G2 crushed stone base course without added soil binder with an EC at any time between compaction and sealing of up to at least about 0,6 S/m, to be used under a new-type Cape seal.

- The completed base course can be left for at least a month unprimed during warm dry weather in autumn and for at least an additional three months during winter without experiencing any significant looseness.

- Such a base primed with an invert emulsion primer is likely to become powdery and loose to a depth of about 10 mm within six to 30 days during warm dry or slightly rainy weather. It should probably therefore be sealed within about two weeks if damage is to be avoided.

- In the long term the only damage likely is looseness and/or blistering and powdering along the slurry overspread and primer overspray. Whilst undesirable this was regarded by the inspection panel consisting of Mr JP Nothnagel of the Roads Department and the author as acceptable if seawater had to be used.

- However, the possibility of punching of the chippings and subsequent bleeding and possibly even slight rutting in the wheelpaths under traffic due to the salt having softened the base during its long exposure cannot be excluded.

- If a saline base is to be left standing unprimed for more than a few weeks the degree of compaction of the upper base should probably be checked before priming at least visually, and preferably tested. Suitable test methods probably include a nuclear gauge, shallow sand density, Clegg hammer and ball penetration as well as engineering judgment with the aid of a hammer or pick handle. (Looseness of a primed base is usually obvious.)

A primer alone or a double slurry seal alone is inadequate as a seal for a base compacted with seawater or, probably, for any base with an EC at the time of sealing of much more than about 0,15-0,20 S/m.

The development of holes in a 19 mm Cape seal (due for example to the weathering out of some of the chippings) will not lead to salt damage if it is used on a primed, crushed stone base without soil binder with an EC at sealing of up to at least 0,6 S/m.

A.5.8 ACKNOWLEDGEMENTS

Messrs R A Appelgryn and later J P Nothnagel of the then Cape Provincial Roads Department were also involved in the problems experienced during construction and took part in some of the subsequent monitoring with the author.

A.6 LÜDERITZ EXPERIMENT

A.6.1. INTRODUCTION

The Lüderitz experiment, at the entrance to Lüderitz town on trunk road 4/2 to Aus and Keetmanshoop in Namibia constructed in1976 represents an evaluation of the use of seawater for base compaction under the most severe coastal environmental conditions occurring in southern Africa.

The objectives were similar to those of the Swartklip experiment (Appendix A.4), but were more comprehensive. They included an evaluation of the following factors believed to possibly affect the use of saline materials:

- Type of base (crushed stone, with and without added soil binder, and calcrete)

- Content of common salt halite (NaCl) in the base

- Content of gypsum ($CaSO_4.2H_2O$) in the base

- Use of seawater for compaction to the top of the base course, both with and without gypsum in the base

- Use of seawater for compaction to the top of the subbase

- Use of seawater for shoulder compaction
- Slushing the base and subbase
- Delay between compaction and priming
- Delay between priming and sealing.
- Type and application rate of primer, and the effect of omitting the primer completely
- Surfacing permeability
- Salt, gypsum and moisture migration during and after construction
- Climate (inside and outside the mist belt)

One set of 14 experimental sections was constructed at Lüderitz (inside the mist belt) and another set of 21 sections at Haalenberg (outside the mist belt), some 40 km inland from Lüderitz on the same road to Aus.

Because seawater was not used at Haalenberg this report will largely be limited to the four seawater sections and the nearest control section at Lüderitz. As seawater had previously apparently been successfully used in Namibia for up to subbase level its use for this purpose was of less concern.

The traffic carried by this road is light and in the critical first five years was only 50-70 vehicles per day (vpd) with 15% heavies and increased to only 160 (13% heavy) with a cumulative 0,2 MESA after the design life of 20 years in 1996. Even after the full analysis period of 30 years in 2006 the traffic was still only 240 vpd with 24% heavies and the road had carried about 0,5 MESA per lane after the final inspection in 2012. After 35 years it had carried abut 0,75 MESA per lane.

With respect to the experimental design the light traffic was favourable, as salt damage is invariably the worst on pavements and those parts of pavements carrying the least traffic.

A.6.2. CLIMATE AND WEATHER

The climate at Lüderitz is cold dry desert (BWk) with frequent fogs and that at Haalenberg probably hot dry desert (without fog). The normal annual rainfall at Lüderitz and Haalenberg is about 20 and 70 mm, respectively (Meteorological Services, Windhoek 2012, pers. comm.). Lüderitz has fog or mist on average about 100 days per year, mostly at night. This usually clears during the morning. The Moisture Index is less than -50, the N-value exceeds 50, and the macroclimate for pavement design purposes is ‘dry’.

The experiment is at an altitude of about 20 m and about 1,0 km due east of the nearest seafront.

No measurable rain fell during construction, a total of 42 mm was recorded at Lüderitz during 1976 and an average of 13 mm for the period 1976-2000, with a minimum of 0,0 and a maximum of 42 mm (Meteorological Services, Windhoek 2012, pers. comm.).

Shade air temperatures taken by the author at Lüderitz during construction varied between 14 and 34°C and relative humidities between the more usual 18% during the day and a maximum of 100% during the night or morning mist. The mean annual temperature over the period 1976-2000 was 15,9°C with a minimum of 12,7 and a maximum of 19,0°C (Meteorological Services, Windhoek 2012, pers. comm.).

A.6.3. PAVEMENT

Surfacing: 19 mm new-type Cape seal with two coats of slurry and 2,0% crossfalls

Primer: MC-30 at the nominal rate of 0,7 ℓ/m
Base: 150 mm of nominal G3 quality graded crushed stone, mostly with 8% added soil binder by mass, compacted to 98% MAASHO. Completed on 20-21 March 1976

Subbase: 100 mm Natural gravel (mostly calcrete) of nominal G5-G4 quality and an EC of 0,1-0,2 S/m compacted to 95% MAASHO

Selected subgrade: 250 mm G5 calcrete natural gravel with an EC of 0,1-0,2 S/m compacted to 93% MAASHO

Fill: Approximately 1 m of rock and soil (latter compacted to 90% MAASHO

Roadbed: Saline soil

Width: 6,8-6,9 m Wide seal on 7,4-7,6 m wide base, with 2,5 m wide gravel shoulders of nominal G5 quality

Alignment: Straight with 1,4% fall towards Lüderitz town

Drainage: Good, finished road level on centreline over 2,0 m above natural gravel level; water table deep

A.6.4. MATERIALS AND TEST METHODS

A.6.4.1 Materials

The seawater used was drawn from a hydrant at the power station in the dock area and the fresh water from the main Lüderitz supply pipeline from Koichab Pan. Chemical analyses of these waters shown in Table A.6.1 show that the salinity of the seawater was normal at about 3,5%, but that the content of sulphate was slightly low. This is not significant.

Table A.6.1. Composition of seawater and fresh water used in the Lüderitz experiment

Component	Units	Seawater	Fresh water
Na^+	% m/v	1,00-1,06	0,010
$C\ell^-$	% m/v	1,88-1,95	0,011
SO_4^{2-}	% m/v	2,12-2,49	0,006
TDS at 180°C	% m/v	3,39-3,53	0,041
EC at 25°C	S/m	5,25-5,50	0,069
Salinity from EC [1]	%	3,5-3,6 (m/m)	0,045 (m/v)
pH	-	7,8-7,9	8,2

Note:
[1] By NITRR Method CA21-74 (1980)

Typical material properties after compaction are shown in Table A.6.2. The EC of the graded crushed stone (a mixture of granite and dolerite) was 0,05 S/m and that of the two soil binders 0,03 and 0,47 S/m respectively. Four percent by mass of each binder was added to the crushed stone. The measured EC of this mixture as used for most of the road and for the control sections before adding water was about 0,12 S/m. After compaction with fresh water it was 0,09-0,15 S/m and 0,3-0,6 S/m with seawater. The salt added was a coarse dairy salt containing about 97% NaCl as halite and the gypsum a "wet" neutralised, industrial gypsum containing about 93% $CaSO_4.2H_20$.

TABLE A. 6.2. Mean as-built base materials test results on Lüderitz seawater and control sections [1]

Section	Additive				EC	Gyp sum	**Percent passing by mass**								**GM**	**PI**	**COMP.**	CBR					Field water content [7]	
																		Lab. & Field @ 98% [5]	Field DCP [6]					
																			LHS		RHS		LHS	RHS
	Binder	NaCl	Gypsum	Water Base/Rest [2]	[3]	[4]	37,5 mm	26,5 mm	19,0 mm	13,2 mm	4,75 mm	2,00 mm	425 µm	75 µm			**OMC**		DN	CBR	DN	CBR	0-25 25-150 mm	0-25 25-150 mm
No.	+ %	+ %	+ %	Type	S/m	%										%	%	%	mm / bl	%	mm / bl	%	%	%
4 [8]	8	0,5	-	Fresh /fresh	0,51	0,2	100	90	76	66	43	32	18	7	2,43	NP- SP	99 7,5	75	-	-	-	-	1,2 1,6	1,7 1,0
10	5	-	5	Sea /fresh	0,43	4,4	100	86	67	57	37	28	18	9	2,45	SP -4	101 5,4	200 400	2,7	150	3,2	120	2,9 3,1	2,5 2,4
11	8	-	-	Sea /sea	0,40	0,5	100	82	66	56	35	25	15	6	2,54	SP	101 5,8	300 400	2,9	140	3,4	110	2,1 2,5	1,6 1,4
12	8	-	-	Fresh /sea	0,15	0,3	100	83	64	52	35	25	14	7	2,54	SP	101 5,3	- 320	-	-	-	-	- -	- -
14 [9]	8	-	-	Fresh /fresh	0,09	-	100	88	67	55	34	24	14	6	2,56	SP	100 5,1	200 270	3,5	105	2,1	210	- -	- -

NOTES to Table A 6.2:

[1] Sampled on 20-21 March 1976 after compaction, but before slushing for indicators and CBR; gradings are means and soil constants min. and max. of three Roads Branch central lab. results at 20L, 30CL and 40 m R points

[2] Seawater was used in the shoulders of Sections 10-13 only (mean shoulder GM 1,79; PI 6). Except for Section 10 'REST' means all other layers including shoulders

[3] EC after slushing with sea or fresh water as appropriate (means of 4-7 Roads Branch central lab. results)

[4] Gypsum as $CaSO_4.2H_2O$. Means of 2-6 Roads Branch central lab. results on same samples for EC after slushing with sea or fresh water as appropriate, using what became TMH1:1986 Method A22

[5] Laboratory soaked CBR at 98% and field compaction. Roads Branch central lab results on one sample from the middle of each section

[6] In-situ DN in mm/blow and CBR estimated from Fig. 10 average in Kleyn (1975) using CSIR 30° cone at in-situ water content after surfacing (first slurry LHS 24 March, RHS 10 April); three per lane near middle of sections

[7] Just before priming : LHS 21 March, on day after bases completed on 20-21 March**; RHS 08 April 1976 after 18 days of exposure;** from near middle of section; all 0-25 mm and 25-150 mm; Sections 4 and 11 midlane (1,7 m L or R) only; Section 10 means of 0,5 and 2,5 m from edge of future seal. All after drying at 110°C. Results for Section 10 with 5% gypsum added all about 1% point lower after drying at 50°C

[8] Control section with 0,5% added NaCl but with fresh water in all layers. All sections were 60 m in length

[9] Control section of normal construction with fresh water in all layers. Seawater was used in the shoulders only of the adjacent Section 13

A.6.4.2 Test methods

The standard indicator and test methods used were those of the South African Department of Transport (1970). The wet A1(a) soil preparation method was used for the sieve gradings and the soil constants. All CBRs were determined at a penetration depth of 2,54 mm after four days of soaking.

Samples for compaction and CBR testing were taken from the dumps on the road before compaction and for indicators from the layer after compaction.

The methods used during construction for the saturated paste electrolytic conductivity (EC), pH and qualitative testing for chloride, sulfate and carbonate were those of the National Institute for Transport and Road Research (NITRR) Method CA 21-74. Their current equivalents which are similar and are considered to yield the same results are theTMH1:1986 (NITRR1986a) EC Method A21T (for EC only) and NITRR CA 21-74 (1980). The simplified routine method A21T was used for all the later testing, including that done in 2012.

What is now TMH1:1986 Method A22T was used for the determination of gypsum. This method is actually for the determination of all acid-soluble sulfates and is therefore not specific for gypsum. The whole grading is crushed and pulverised to pass 0,425 mm for analysis. The results reported here are as gypsum ($CaSO_4.2H_20$).

The saturated paste EC method is carried out on the air-dried, dry-screened fraction passing 6,7 mm without any crushing in of the plus 6,7 mm fraction. It is intended to provide a rapid indication of only the very soluble salt content such as due to NaCl and Na and Mg sulfates and to minimise the effect of the only slightly salt gypsum.

All water (moisture) contents were determined at the usual 105-110°C without any allowance being made for any possible loss of bound water, such as that in gypsum.

A.6.4.3 Effect of seawater on engineering properties

Limited testing of the effect of seawater indicated that it had little effect but may even have aided compaction slightly.

A.6.5. EXPERIMENTAL LAYOUT, CONSTRUCTION TIMING AND SALT CONTENTS AFTER COMPLETION

The Lüderitz experiment consists of twelve sections each 60 m in length comprising sections with and without added binder, additions of 0,2, 0,5, 1 and 2% deliberately added NaCl, 2, 10 and 20% deliberately added gypsum, as well as three seawater sections. Crushed stone with binder compacted with fresh water as used over the rest of the road served as control sections at each end of the experiment. Details of the seawater section and the nearest control sections are shown in Table A.6.1. The two seawater bases received a normal slush with seawater and the other two a normal slush with fresh water. Most of the other sections only received a light slush (water roll) with fresh water. Seawater was also used in the shoulders of the two seawater bases and the two seawater subbase sections, i.e. from SV 123+980 to 124+220.

Although the stone and additives were not always well mixed, the base was thoroughly broomed and very little fines were present on the base after brooming. A normal light spray of water was usually used before priming.

The Haalenberg experiment was largely similar to the Lüderitz experiment except that seawater was not used and that both crushed stone and calcrete were tested as base course.

It was intended that one lane should be primed and sealed as soon as possible and the other when convenient a few weeks later. The actual timing achieved is shown in Table A.6.3. The effect of a longer delay of up to 18 days between priming and sealing and the effect of different primers and application rates was studied by means of small-scale (1 m x 2 m) priming experiments with MC-30, MC-70, 3/12 evt tar and invert emulsion (MSP1) primes and a spray grade emulsion tack (i.e. with no prime) in the otherwise unprimed north lane. Short sections of base were also left completely unprimed.

Table A.6.3 Construction timing of the Lüderitz experiment

Delay between	Left Lane [1]	Right Lane [1]
Base [2] and priming	1 day	18 days
Priming and tacking and chipping	3 days	1 day
Tack and top sprays	0 days	0 days
Top spray and first slurry [4]	1 day	1 day
First and second slurry	9 weeks	7 weeks

Notes:

[1] Stake value (SV) zero was at Aus; left lane is Lüderitz-bound, right is Aus-bound

[2] Base completed on 21 March 1976

[4] Slurry rolled (2 passes), construction trafficked and opened to general traffic within the following few days.

The Contractor estimated that 6% water was added for compaction plus a 2% allowance for evaporation plus another 2% for slushing, i.e. 10% by mass in all. This should have added about 0,35% sea salt to the base. As the base possessed about 40% passing the 6,7 mm sieve this should have added about 0,9% salt to this fraction and increased the EC by about 0,9 S/m to about 1,0 S/m. However, Tables A.6.1 and A.6.4 show that the measured values are only about half this. The reason for this is not clear. Either the Contractor added half the water that he claimed he added, or the samples were crushed to pass the 6,7 mm sieve instead of simply screened, or the assumption that nearly all the salt ends up in the < 6,7 mm fraction is in error.

Although all EC values quoted here were reported as having been determined on the fraction passing 6,7 mm and not crushed to pass this sieve, there was some doubt about this which was never cleared up. However, ECs on crushed material will be lower than and probably about half of those obtained by simply screening out the < 6,7 mm fraction. The assumption that all or nearly all the salt is in the minus 6,7 mm fraction tested appears reasonable both on theoretical grounds and on the basis of limited testing, but was insufficiently investigated. However, conclusions drawn from the results reported here should at least be conservative rather than the reverse. Moreover, the measured EC of about 0,5 S/m for the seawater bases is about the same as those at Swartklip (Appendix A.4) and Lambert's Bay (Appendix A.5).

In the case of the seawater bases the EC of the full thickness of base was not found to change significantly between completion and surfacing nor was there much difference between the lanes. However, in the case of the fresh water base with a seawater subbase the average EC of the exposed lane increased from 0,15 after completion to 0,25 S/m after two weeks of exposure, indicating migration from the seawater subbase and shoulders. The salt contents in the sealed lane did not change significantly during this period. The ratio of the EC of the upper 25 mm of base to the EC of the rest of the base increased during exposure, indicating migration within the layer.

The measured salt contents of the Lüderitz bases and lower layers after compaction and with time are shown in Table A.6.4. Once again, the ECs of the added salt sections are lower than expected. Although a large variation in the ECs is evident and was expected, the mean for each section agreed well with the nominal salt content added. As the solubility of NaCl is about 36% m/v, some 2,0% salt could be held in solution at the MAASHO OMC of 5,5%. At about half this, which is a likely equilibrium moisture content for a base in an arid area (Emery, 1984) only about 1,0% could be held in solution. Any excess salt built into the base will thus tend to crystallise, with resultant probable damage, as the base tends towards equilibrium.

It may therefore be risky to seal a crushed stone base in an arid or semiarid climate with a content of NaCl greater than about 1,0%, equivalent to an EC of 1,1 S/m with NaCl or 1,05 S/m with sea salt according to Method CA-21. According to this concept the amount of salt tolerable should be proportional to the equilibrium moisture content, which in turn is proportional to the OMC. If the upper base dried out, less would be tolerable.

In order to allow for overlap and contamination between sections all inspections and sampling were limited to the central 20-40 m of these short 60 m sections.

In the case of the Section 10 seawater base the EC of the full thickness of base was not found to change significantly between completion and surfacing nor was there much difference between the lanes over the 14-month period of testing and even after 36 years. However, in the case of Section 12 with a fresh water base and a seawater subbase and lower layers, the average EC of the exposed (left) lane increased from 0,16 after completion to 0,25 S/m after two weeks of exposure, indicating migration from the seawater subbase and possibly shoulders. The salt contents in the sealed lane did not change during this period. The ratio of the EC of the upper 25 mm of base to the EC of the rest of the base (not shown) increased during exposure, indicating migration within the layer.

As halite (NaCl) does not form hydrates at the temperatures involved, and as its solubility in water is almost independent of temperature, ranging only between 35,7 g/100 g water at 0°C and 37,8 g/100 g water at 70°C (Perry and Green, 1984, with similar data in Lide, 1992) the main factor controlling the amount of salt tolerable in a pavement layer is the amount of free water it contains. (Free water is water present as liquid and not part of a mineral such as gypsum, $CaSO_4.2H_20$.)

Table A.6.4 As-built salt (EC) and gypsum contents and changes in EC with time

Section no.				**10**	**11**	12	**13**	14
Base [1]			Units					
		Binder	+%	5	8	8	-	8
		Gypsum	+%	5	-	-	-	-
		NaCl	+%	-	-	-	-	-
		Water	Type	**Sea**	**Sea**	Fresh	Fresh	Fresh
GYPSUM AND EC AT :	Completion	**Gypsum** [9]						
		Mean	%	4,4	0,50	0,30	0,19	-
		Min.	%	3,1	0,44	0,25	0,17	-
		Max.	%	5,9	0,67	0,34	0,20	-
		EC [10]						
		Min.	S/m	0,37	0,21	0,13	0,06	0,09
		Max.	S/m	0,47	0,54	0,23	0,09	0,10
		Mean	**S/m**	**0,43**	**0,40**	**0,15**	**0,07**	**0,09**
		LHS Mean	S/m	0,40	(0,51)	(0,13)	0,07	0,09
		RHS Mean	S/m	0,47	0,35	0,18	0,08	0,09
	2 Weeks [6]	LHS	S/m	0,34	-	0,16	0,15	-
		RHS	S/m	-	-	0,25	0,11	-
	2 Months [7]	LHS	S/m	0,51	-	0,16	0,08	-
		RHS	S/m	0,40	-	0,29	0,07	-
	7 Months [7]	LHS	S/m	0,40	-	0,15	0,07	-
		RHS	S/m	0,36	-	0,27	0,08	-
	14 Months [7]	LHS	S/m	0,33	-	0,24	0,08	-
		RHS	S/m	0,42	-	0,33	0,11	-
	36 Years [8]	LHS	S/m	0,45 0,39	1,77	-	-	0,49
		RHS	S/m	0,39	1,24	-	-	-
Subbase [2]		Water	Type	Fresh	**Sea**	**Sea**	Fresh	Fresh
EC AT:		Completion	S/m	0,09	0,37	0,40	0,08	0,09
		2 Weeks [6]	S/m	0,20	-	0,38	0,09	-
		14 Months [7]	S/m	0,22	-	0,37	0,11	-
Upper Selected [3]		Water	Type	Fresh	**Sea**	**Sea**	Fresh	Fresh
EC AT:		Completion	S/m	0,09	0,54	0,59	0,14	0,08
		2 Weeks [6]	S/m	0,21	-	0,39	0,15	-
		14 Months [7]	S/m	0,28	-	0,69	0,26	-
Lower Selected [4]		Water	Type	**Sea**	**Sea**	**Sea**	**Sea**	Fresh?
EC AT:		Completion	S/m	**-**	**-**	**-**	**-**	-
		2 Weeks [6]	S/m	0,49	-	0,63	0,49	-
		14 Months [7]	S/m	0,41	-	0,53	0,48	-
Shoulders [5]		Water	Type	**Sea ?**	**Sea**	**Sea**	**Sea ?**	Fresh
EC AT:		Completion	S/m	0,15	0,31	0,25	-	-
		2 Weeks [6]	S/m	0,34	-	0,23	0,17	-
		14 Months [7]	S/m	0,27	-	1,15	0,24	-

Notes to Table A.6.4:

[1] Base completed on 20-21 March; LHS primed 21 March; RHS 08 April 1976. Small priming experiments in right lane on Sections 4, 10, 11 and 13 **(embolded)** on 22 March 1976

[2] Subbase completed on 4-5 March (Sections 1-13), 13 March 1976 (Section 14). ECs means of mostly 2-3 samples

[3] Upper selected subgrade completed on 28 Feb. (Sections 1-13), 04 March 1976 (Section 4). ECs mostly on single samples after completion and means of 3-4 subsequently

[4] Lower selected subgrade completed on 06 Feb. (Sections 1-13), 03 March 1976 (Section 14). ECs mostly on single samples after completion and means of 3-4 subsequently.

Notes to Table A.4.1 continued

Final fill completed 14-24 Jan. 1976, with EC of 0,2-0,3 S/m (not tested subsequently). Figures bracketed estimated from results on passing 0,425 mm fractions

[5] Shoulders tested for compaction on 16 March 1976

[6] Two **weeks,** apparently after surfacing (to first slurry). (LHS primed; RHS still unprimed)

[7] Time periods in **months** are after first **slurry** on LHS 24 March, RHS 19 April 1976

[8] Sampled on 29-30 March 2012. All VKE Namibia results except for 1,24 S/m on Section 11 by ANALAB, Windhoek; pH according to TMH 1 Method A20 but on minus 2,00 mm fraction 7,0-8,1.

[9] Base gypsum (as $CaSO_4.2H_2O$) means of mostly 2-4 samples. Neat gypsum content of G3 base mostly 0,1-0,2%; calcrete subbase 0,3-0,4%; calcrete shoulders 0,3-0,5%.

[10] EC means of 5-7 samples. Neat EC mostly about 0,10 S/m. Min., max. and mean for section (i.e. both lanes) **include** centreline results. LHS and RHS lane results **exclude** them. Results bracketed in lanes at completion include one or more estimated from results on minus 0,425 mm fraction.

[11] **Not in table** : Water contents March 2012 : Upper 25 mm : mean 2,5% (0,8-8,6%, n = 5); Rest of base: mean 1,5% (0,7-2,2%, n =14); whole base from Sections 9 and 10 **at 60°C** : mean 1,9% (1,4-2,5%, n = 9), at 107°C : mean 3,0% (2,1-4,9%, n = 9).

The water contents of the density samples of the Lüderitz base courses after compaction ranged between 2,0 and 5,0%. If the result is corrected for the gypsum content, then the average **free** water content was 2,0-3,6%, with an average of 2,8%, thus supporting the suggested maximum of 1,0% NaCl or an EC of about 1,7 S/m, during construction, at least.
As, in contrast to NaCl, the solubility of natural gypsum is only 0,22 g $CaSO_4.2H_2O$ / 100 g pure water at 0°C and 0,25 g / 100 g at 70°C (calculated by author from data as $CaSO_4$ in Perry and Green, 1984) very little can be dissolved in pure water and its migration and crystallisation in significant amounts is therefore unlikely under most conditions. Like NaCl, its solubility is little affected by temperature. Although its solubility is increased in the presence of NaCl, this effect is not great and the added amounts of up to 20% were not found to be detrimental in the G3 bases at Lüderitz and Haalenberg (Netterberg, 2021). Water content determinations on Sections 4, 10 and 11 at Lüderitz, two months after the first slurry seal ranged between 1,2 and 3,1% or 1,3-2,3% with a mean of 1,8% **free** water when corrected for the gypsum content. Such an average water content could only hold about 0,6% NaCl in solution in the base as a whole. As the water contents of the upper 25 mm of the base were mostly 0,1-0,4% lower, a maximum of about 0,5% NaCl would be indicated for this critical upper part of the base.

Water contents of five samples of the upper 25 mm of base at Lüderitz after 36 years ranged between 0,8 and 8,6% (both extremes on Section 14, others 2,3-6,8%), averaging about 2,5% and 0,7-2,2% (average 1,5% for 14 samples of the rest (i.e. 25-150 mm) of the whole of the 150 mm base were similar to the 1,4-3,1% found after two months. At a solubility of 36% and round figures of water content of 1,0 and 1,5%, about 0,35 and 0,5% NaCl could be held in solution, respectively. Checks on Sections 9 and 10 to which gypsum had been added yielded water contents of 1,4-2,6% with a mean of 1,9% after drying to constant mass at 60°C and 2,1-4,9% (mean 3,0%) after the usual overnight drying at 107°C, differences of 0,4-2,4 percentage points.

From the water content determinations and solubility considerations it is **concluded** that an upper limit of about 0,5% NaCl in the whole base course (equivalent to a < 6,7 mm EC of about 1,0 S/m) should generally be safe under conditions similar to those at Lüderitz.

After about one year (tested at 14 months after the first slurry) the average EC of the gypsum-seawater base (Section 10, initially at 0,43 S/m) and the fresh water base with the seawater lower layers (Section 12, initially at 0,16 S/m) was found to be about the same at about 0,3-0,4 S/m in both lanes, the left and right lanes being 0,33 and 0,42, and 0,24 and 0,33 S/m of the two sections, respectively. The EC ratios, i.e. the ratios of the EC of the upper 25 mm to the EC of the rest of the layer (not shown), had dropped to about unity, indicating that the salt was evenly distributed vertically within the base. From this it is **concluded** that there is little long-term value in restricting the seawater to the top of the subbase, although it should help to prevent any salt damage to the primed base if it has to stand for long before sealing. The plain seawater base (Section 11) was not tested, but probably behaved similarly.

The EC of the base of Section 2 without added binder on a saline dorbank subbase with an EC of 0,52 S/m approximately doubled from an initial 0,08 S/m to about 0,18 S/m within less than a year (not shown), whilst that of the subbase had decreased and those of the selected layers had increased. This, together with the findings from the seawater subbase Section 12 and the decrease with time of the EC of the seawater base of Section 10 together with the increase in EC of the selected layers, lead to the **conclusion** that significant salt migration can take place both upward from the subbase into a sealed base over a distance of about 150 mm, as well as downward from the base or subbase to a distance of at least 300 mm. In the case of Section 10 one cannot of course say whether the increase of EC in the upper selected was due to migration from the base or the lower selected, which latter is more likely.

The average salt content of the seawater **shoulders** on Sections 10-13 at Lüderitz had doubled or more from about 0,15-0,3 S/m after compaction to as much as 1,2 S/m in the case of Section 12 after one year and they also became loose to a depth of about 10 mm. Experience elsewhere on this road was that the surface of similar subbase started to become loose when the EC of the subbase rose above about 0,4 S/m. The source of this salt has not been investigated, but is presumed to have risen from the lower layers and/or been added by windblown salt, which can penetrate up to about 10 km inland (Callaghan, 1984).

A.6.6. PERFORMANCE DURING AND SOON AFTER CONSTRUCTION

The performance of this experiment was monitored by regular inspections of a panel consisting of members of the Namibian Department of Transport and the author, who was also present throughout construction. In general the standard of workmanship was average to poor, with poor mixing, uneven primer and tack spraying and an excessively wide base course.

Salt stains and/or salt glazing were evident both before and after priming on all the sections of base containing 0,5% or more of added salt and also to a much lesser degree on the seawater sections. This did not receive any special attention. The salt and seawater sections retained their moisture longer and did not dry out as rapidly as the control sections. The depth of primer penetration on these sections was also reduced thereby and the primer took longer to dry.

When the north lane was primed 18 days after completion of the base neither the bare base nor any of the small-scale patches of primer had become significantly damaged by salt. However, in general, within the first ten days the upper 1 mm of the base had become loose on the seawater sections and also where the primer had penetrated most on the small-scale priming experiments. This was regarded as negligible and acceptable and did not require any special treatment. The primer overspray (i.e. that part of the primer on the edge of the base not intended to be sealed) along the outer edge of the seal of the south lane had become very slightly loose in a few places within about ten days of spraying. This occurred on most sections except the controls. By 14 April 1976 about 5% of the overspray on the 0,5% salt section was affected but less than 1% on the control sections. Wherever the overspray had gone onto the shoulder gravel the overspray had always become loose.

A.6.7. LONG-TERM PERFORMANCE

An inspection one year after sealing showed no significant damage to any section. However, rare hair cracks, salt stains and/or wet patches were noticed on the seawater base and the 0,5% salt sections, suggesting that the seal was not impermeable. These were limited to the areas of the seal outside the wheeltracks. All signs of the primer overspray had disappeared even on the fresh water control sections. However, most of the full 300 to 500 mm width of the normally exposed base at the edge of the seal had received a slurry seal (here called slurry overspread) that would have covered most of the overspray. This slurry had become loose wherever it had been spread onto the shoulder.

In 1979, three years after sealing, similar, hardly noticeable, hair cracks were observed on all sections including the fresh water controls. About 10% of the area of the slurry overspread on the seawater sections had become loose. The gravel shoulders of the seawater sections had become loose to a depth of about 25 mm. All of this was considered acceptable by the panel in this environment.

The panel provisionally concluded that – with precautions – seawater could be used for the compaction of all layers including the shoulders and crushed stone base course. (It was known that salt damage normally appeared within about one year of surfacing.) However, blanket permission for the use of seawater should **not** be given.

No significant further deterioration had occurred at Lüderitz or Haalenberg when inspected by the author in 1981, i.e. after five years of service and all sections were regarded as acceptable. In 1986 it was reported that no further deterioration had taken place, that none of the sections had been patched or resealed, and that the road was not on the programme for resealing, and this was confirmed by panel inspections carried out in 1991 (Figures A.6.1. and 2) and 2012 (Figures A.6.3-5)

A.6.8. PAVEMENT EVALUATION

A pavement evaluation carried out in 1979 showed little difference between any of the sections, in either condition or expected structural capacity.

The in-situ permeability of the surfacing to air as measured with a Soiltest Asphalt Paving meter essentially according to ASTM D3637-84 but without the weight at a pressure head of 6,4 mm of water varied between 0,9 and 2,6 mℓ/cm^2/minute on the control sections and 1,1 and 6,2 mℓ/cm^2/minute on the gypsum-seawater base section. All except two readings on the latter section were below 1,9 mℓ/cm^2/minute. One measurement on an uncracked area of the 2% salt section gave a reading of 1,6 and one on a hair-cracked area 3,8 m **ℓ**/cm^2/min. It is not known whether the higher (in places) permeabilities on the seawater section and having used seawater are causally related. However, this seems otherwise, since the hardly noticeable hair cracking was actually slightly worse on the nearby section with seawater only up to the top of subbase level. The cracking was probably due to crazing of the slurry due to weathering or excess water or insufficient bitumen in the mixture. The three spots on the seawater section with permeabilities in excess of the 1,7 mℓ/cm^2/min. suggested by the work of Netterberg (1979) as necessary to prevent salt damage to asphalt-surfaced pavements with an excess of salt in the base course, were not damaged, however, the one on the 2% salt section was.

Figure A.6.1: (03 Feb. 1991): Section 10 (5% gypsum + seawater), view towards Lüderitz. No salt blisters, but Degree 1/Extent 5 map to star surface cracking, especially in right lane.

This suggests that the salt contents (EC of about 0,5 S/m) of the seawater section were insufficiently high to cause damage to a new-type Cape seal even when relatively permeable. However, an upper limit for permeability of about 2,5 with an average of 2,0 mℓ/cm^2/minute is probably wise.

Radius of curvature measurements of the deflection bowl under an 80 kN wheel load using a Dehlen curvature meter showed values averaging about 180 m for the seawater sections and 150 m for the control sections. In general, those sections containing salt and/or gypsum showed higher radii of curvature than the control sections, indicating that the former base courses were stiffer. The lowest reading anywhere on any section was 102 m. These measurements showed that the structural condition of the upper pavement (especially the upper 300 mm) was good and that the salt, gypsum and seawater had **not** reduced the expected structural capacity life of the pavement. All radii of curvature were also well above the 25 to 30 m below which fatigue cracking of the surface would be expected, supporting the view of the inspection panel that the surfacing cracking seen on all of the sections was due to shrinkage cracking (crazing) of the slurry and not flexure-related fatigue cracking.

There was no significant difference between rut depth (about 5 mm), smoothness (about 5 mm) and riding quality (about 3,3 PSR) between any of the sections.

Figure A.6.2 (03 Feb. 1991). Section 11 (Seawater in all layers), view towards Lüderitz. No salt blisters, but Degree 1/Extent 5 map to star surface cracking. Similar cracking occurred on all Lüderitz sections including the controls.

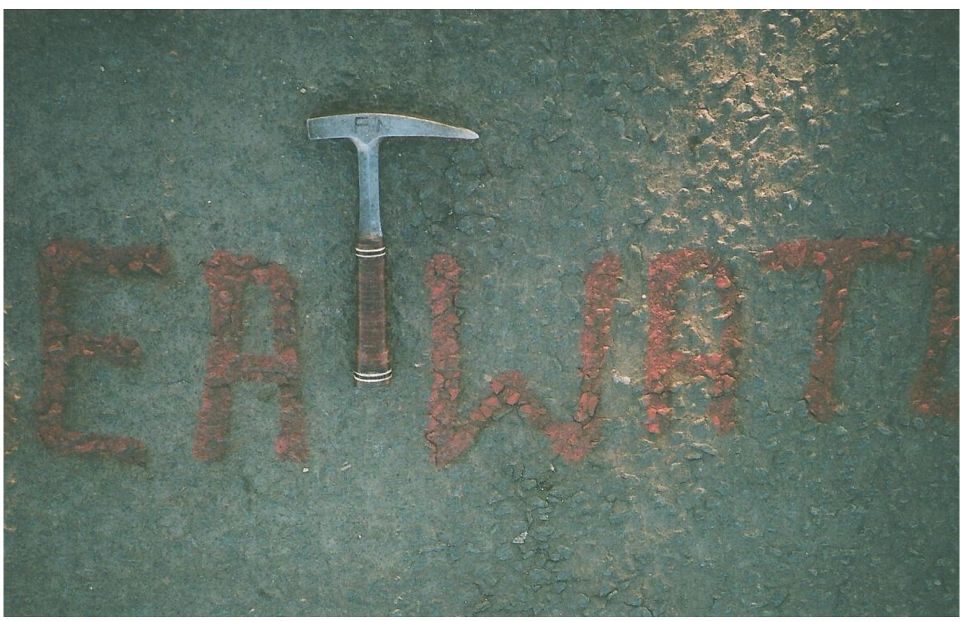

Figure A.6.3 (29 March 2012: Section 11 (Seawater G3 at SV 124 + 070, Right midlane). Close-up view of worst cracking on section (Degree 1/Extent 5 random to star surface, averaging 50 mm length at 100 mm spacing).

Figure A.6.4. (02 April 2012 after 36 years) : Section 14 (West control) at SV 124+280, left lane, view towards Aus. Best section on experiment. Degree 1/Extent 5 map to star surface cracking all over section. Full width slurry from approx. SV 124 + 261 to + 264. Rejuvenation or reseal at C priority recommended. Distinct wetness between wheelpaths on this and other sections after rain indicating permeable seal.

Figure A.6.5 29 March 2012 : Section 11 (Seawater in all layers), Left lane, view towards Aus. No current salt blisters, but scattered (Extent 3) small holes 50 mm diam. possibly representing broken blisters along centreline. Degree 1/Extent 5 map to star surface cracking over whole section. Similar cracking occurred over all the Lüderitz sections to a greater or lesser degree and extent including the control sections. West Control Section 14 (reseal at C priority) only marginally better than Section11 (reseal at B priority).

A.6.10 CONCLUSIONS

- Seawater can be successfully used for the compaction and slushing of a crushed stone base with added soil binder of G3 quality, whether containing gypsum or not, as well as for the underlying layers and for the shoulders, in a climate such as at Lüderitz (BWk: cold dry desert with frequent fog).

- The performance of the seawater bases was similar to that of sections of similar EC (about 0,5 S/m) to which 0,5% salt (NaCl) had been deliberately added.

- The performance of the crushed stone base sections containing added salt at Haalenberg was similar to that at Lüderitz (not shown here).

- Seawater could therefore also have been used for compaction of all layers at Haalenberg (BWh: hot dry desert) and therefore for the whole road as far as would have been economic, and probably under any climatic conditions anywhere.

- At any time of the year under conditions similar to those at Lüderitz a crushed stone base with soil binder compacted with seawater can be left unprimed for at least 18 days (at least 15 days at Haalenberg) without suffering significant loss of density. The presence of seawater and/or other salt will cause the base to dry out more slowly than normal.

- Also under Lüderitz conditions a primed crushed stone base with soil binder compacted with seawater and/or with a similar EC can be left for at least 17 days (14 days at Haalenberg) without suffering significant primer damage. However, very slight and acceptable looseness may occur within ten days when the EC is more than about 0,5 S/m. This should not require any special treatment. [However, experience on the other roads indicates that provision for sealing within one week is advisable at an EC of more than 0,5 S/m, especially (and possibly only) if excess fines are present on the top of the base.] The presence of seawater and/or salt may cause the depth of primer penetration to be less than normal, the primer to take longer to dry and to a reduced application rate of primer being desirable. Any reduction in the primer rate should probably be compensated for by increasing the tack rate in order not to compromise surfacing permeability, durability and bond.

- Under Lüderitz conditions a highly saline crushed stone base can be left for a total of at least 19 days after compaction before sealing without suffering damage, whether primed or unprimed. Under Haalenberg conditions a delay of at least 15 days is permissible.

- Little difference was found between the performance of the different types and application rates of the primers tried. However, none became significantly damaged on the crushed stone bases.

- Under these conditions a primer coat can be omitted. However, the tack spray should then be increased by 0,15 ℓ/m^2 (NITRR, 1986b).

- In the longer term, salt damage to such bases is likely to be limited to looseness or loss of all or most of the primer overspray within one year and, within three years, some looseness, cracking or loss (10%) of the slurry overspread on the base, general looseness of the gravel shoulders to a depth of about 25 mm and, at salt contents of more than about 1% NaCl, some hair cracking (possibly representing incipient blistering). No additional maintenance of the seal should be necessary in ten years.

- In order to avoid any risk of damage within five years to a 19 mm new-type Cape seal used with or without a primer on a 150 mm G2 or G3 quality crushed stone base the EC of the completed base should be limited to a maximum of about 1,5 S/m when sealed. In addition, EC tests should be carried out on all samples taken for gradings and indicator tests and the average value for each lot of base (say 6 000 to 8 000 m^2) should not exceed about 1,2 S/m. In addition, the air permeability of the surfacing, as measured with a Soiltest Asphalt Paving meter at a pressure head of 6,4 mm of water, should probably not exceed an average value of 2,0 and a maximum value of 2,5 m ℓ/cm^2/min.

- The foregoing conclusions do not apply to more finely graded or natural gravels such as calcrete, which are likely to suffer severe primer (and possibly some surfacing) damage under these conditions.

- A primer alone or a slurry seal on a primer are inadequate surfacings even on a crushed stone base under these conditions and will be rapidly destroyed on a gravel base or shoulder.

- Gypsum was not harmful in base courses under the conditions of the experiment and may actually exert a weak cementing effect (Netterberg, 2021).

- The use of seawater and/or high salt contents and/or high gypsum contents does not reduce the structural life of a pavement under these conditions.

- The salt content of a base of initially low salinity will increase both during exposure and after sealing if placed on a subbase of higher salinity.

- Seawater restricted to the subbase will migrate into the base course and within one year the salt content of the base course will be nearly as high as that which it would have been if it had been compacted with seawater in the first place.

- There is therefore little long-term value in restricting the use of seawater or excessively saline materials to the subbase unless an intervening impermeable membrane is utilised. However, in the short term (i.e. during construction) such restrictions may prevent or minimise damage to the primed base if it is left exposed for more than two to four weeks before sealing.

- Under a relatively impermeable surfacing the salt in a base becomes evenly distributed vertically within the base within one year.

- Under the conditions of the experiment at Lüderitz any thermal salt pumping action, diffusion or any other process was acting to even out and not to concentrate the salt in the upper base.

- Calcrete natural gravel shoulders compacted with seawater on lower layers also compacted with seawater became loose to a depth of 10 mm within one year and to 25 mm within three years.

- The salt content of these shoulders increased from an initial EC of 0,3 S/m after compaction to 0,6 S/m after one year. (Experience elsewhere on this road was that the surface of similar subbase started to become loose when the EC of the full thickness of subbase rose above about 0,4 S/m.)

- Seawater should not be used for the compaction of gravel shoulders if such looseness is not tolerable.

A.6.11. ACKNOWLEDGEMENTS

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A.7 OSHIVELLO-ONDANGWA ROAD

A.7.1 INTRODUCTION

This 150 km-long trunk road TR1/11 in the north of Namibia runs from Oshivello (near Namutoni) 80 km northwest of Tsumeb to Oshakati near the Angola border. The road skirts the eastern side of the saline Etosha pan and practically all ground water over the route was saline, often more saline than seawater.

The traffic on this road when the last section was opened to traffic in early 1968 was about 100 vehicles per day with 50% heavies but later increased to one of the most heavily trafficked roads in Namibia.

A.7.2 CLIMATE

The climate is hot dry steppe (BSh) with a normal annual rainfall of 400-500 mm, falling mostly in summer, a Moisture Index of about -50 and an N-value of 5-6. For pavement design purposes the macroclimate is 'dry'.

A.7.3 PAVEMENT

Surfacing: 25 mm continuously graded medium asphalt

Tack: Colas spray at 3,3 ℓ/m^2

Primer: Mostly Dundee or Iscor coke oven tar, some Sasol Lurgi tar, all 3/12 evt, at 0,7 ℓ/m^2

Base: 150 mm 4% lime-stabilised calcrete G5 natural gravel. Unstabilised 95% CBR about 50-70 at 95% MAASHO, GM about 2,0,% passing 6,7 mm 55-65 and PI of SP-13. Minimum laboratory soaked CBR after seven days curing and four days soaking 160 at 97% MAASHO compaction. The PI after stabilisation was generally NP or SP. Mostly water-cured for four days and primed one month later.
Subbase: 100 mm natural gravel calcrete similar to that used for base, but not stabilized.

Subgrade: Natural gravel calcrete.

Width: 7,4 m surfacing with calcrete gravel shoulders.

A.7.4 SALT CONTENTS

Base material: < 6,7 mm EC 0,04-0,74 S/m, mostly NaCl, but possibly some Na_2SO_4.

Compaction water: About 10% of chloride-sulphate water used for compaction with a TDS of 0,5 to 5,0%.

Curing water: Base cured two to three times per day with above water at rate of 5,6-8,4 ℓ/m^2/day for four days.

Completed base: Estimated < 6,7 mm EC of 0,5-0,8 S/m in many places. Curing water estimated to have added up to 3,6% salt to upper 25 mm of base in places.

A.7.5 PROBLEMS

The problems on this road during construction were observed by the author during a visit in January 1968 and during his comprehensive investigation of the almost completed road in February/March 1968 and followed up by several later inspections.

The upper 7 mm of base became loose and dusty in places soon after priming, even when not tasting particularly saline. Such areas were thoroughly broomed, some watered and rerolled, some tacked twice and one 0,7 km section from ch. 125 to 150 ch. rebuilt.

A few months after surfacing it was noticed that a 1,6 km untrafficked section from ch. 181 to 233 was showing distress in the form of blisters about 13 mm high on an average spacing of 0,5 m with a crack on a starburst pattern radiating from the apex of each blister. The star cracks were 150-300 mm long and less than 1 mm wide. On removing the blistered surfacing a layer of “dust” up to 20 mm thick, but averaging 7 mm was found in each case (Figure A.7.1.). Such “dust” was found to have a total salt content of up to 2,2% with1,8% calculated NaCl and the salt was confirmed as halite (NaCl) by X-ray diffraction analysis.

Figure A.7.1. Loose interlayer between asphalt and lime stabilised calcrete base course on intact, undisturbed sample of base and subbase from Oshivello-Oshakati road

Using the sea salt curve of Figure 7 of NITRR Method CA21-1974 (1980) it was estimated that the < 6,7 mm EC would have been about 1,8 S/m assuming that it all passed 6,7 mm. Similar samples from the top 25 mm of the base from two unblistered areas had total salt contents of 1,3 and 1,8%. The < 6,7 mm EC of such material would have been 1,2 and 1,5 S/m respectively if it was all finer than 6,7 mm. If, as is more likely, only about 60 to 80% passed 6,7 mm, the< 6,7 mm EC would have been about 1,4 times (i.e. 100/70) these amounts, i.e. 1,7 and 2,1 S/m. This is about the same as the disintegrated material under the blisters and implies that a coarser material can tolerate more salt. The actual star cracking was limited to shorter lengths within the section from ch. 181 to 233. For the section almost continuously affected (and also the reconstructed section) water with a TDS of about 5% m/v had been used for both compaction and curing, as well as base material with an EC of 0,2 S/m. The estimated EC of this compacted base was probably about 0,8 S/m before curing. Where water with only 1,8% TDS was used with material from the same borrow pit the damage was only sporadic (estimated EC of compacted base before curing 0,6 S/m). No problems were encountered over a section where water with about 3,5% TDS was used with a material with an EC of only 0,04 S/m (estimated EC of compacted base 0,5 S/m).

The available data suggest therefore that possibly both the TDS of the curing water must be limited to less than 1,8% and the EC of the compacted, uncured base limited to a maximum of 0,5 S/m if damage is to be avoided. Alternatively, it would appear that water with up to 3,5% TDS could be used for both compaction and curing for four days provided that the EC of the base material before the addition of lime was not more than about 0,05 S/m.

Further investigation of the whole completed road by means of trial pits and radius of curvature measurements using a Dehlen curvature meter under an 80 kN wheel load showed, however, that the upper 25 mm of the base was significantly weaker than the rest of the base (which was hard and well-cemented) over much of the road, even where water spray curing had apparently not been carried out. Unfortunately, the samples taken from such areas were never tested. Although additional factors such as over-compaction and degradation of the upper base may also have been involved this problem was never completely solved. With hindsight it can now be said that both the curing procedures used, i.e. water spraying and leaving for a month before priming, or priming as soon as possible with a light (0,7 **ℓ**/m^2) spray of a light (3/12 evt) tar primer and then leaving for some time before sealing would have led to rapid carbonation of the stabiliser in the upper base (Netterberg and Paige-Green, 1984) as well as inhibition of the stabilisation in the second case. (It is now known that carbonation is almost invariably accompanied by a significant weakening or even complete loss of cementation over the whole exposed surface of the stabilised layer.)

It is further suggested that curing any saline base with any water, saline or fresh, is likely to bring about or aggravate salt damage in a situation in which it is not possible to keep the layer genuinely moist all of the time. In the real situation in hot dry weather such curing invariably leads to the upper part of the layer being subjected to wetting and drying cycles and therefore a form of the well-known wet/dry durability test for cemented materials.

The investigation led to several new findings:

- The necessity for a rapid site test for soluble salt content.
- The first case in which NaCl was found to have damaged a road pavement (all previous cases had been caused by acidic sulphates).
- That, contrary to expectations, radius of curvature measurements on such a pavement were highly temperature-sensitive, yielding **higher** radii of curvature with increasing temperature (Netterberg and Haupt, 2003).
- Although the initial hypothesis was that the asphalt was being attacked by the sulphate-reducing bacterium Desulfovibrio desulfuricans, and it was indeed found in the base, water and soil, it was concluded that the damage was due to the crystallisation of NaCl as halite in the highly deleterious filamentous habit (Figure A.7.2).

A.7.6 SOLUTIONS

The brooming and even rerolling and extra tacking in places were insufficient to prevent the blistering and star cracking over the cracked mile from ch. 184 to 225, nor the general occurrence of a loose and/or weak layer at the top of the base over much of the road. At least some of the looseness must have been developed after surfacing and could hardly have been caused by the little (or even no) traffic using the road at that stage. Evaporation through the probably permeable asphalt probably led to crystallisation of more salt at the point where the water evaporated and hence the blistering and star cracking.

The section from ch. 181 to 233 showing blistering and/or cracking of one kind or another was overlaid with an extra 25 mm of asphalt during1968 and opened to traffic

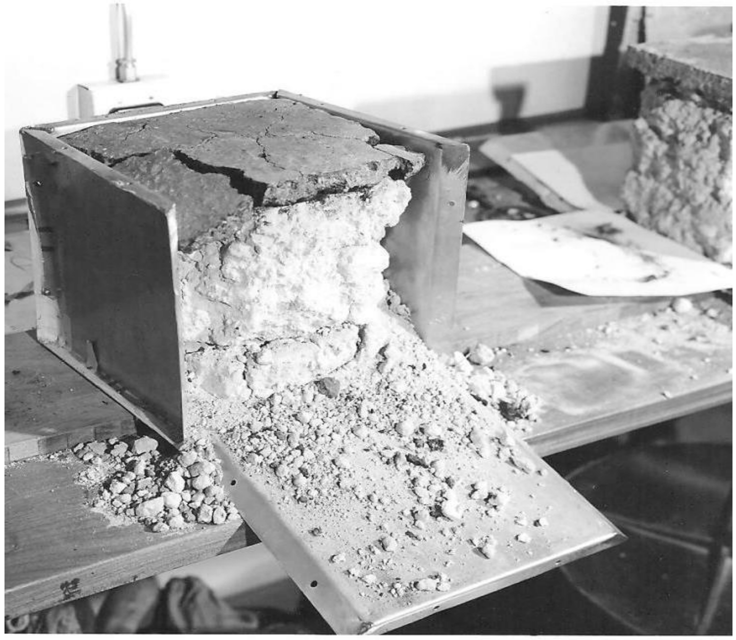

Figure A.7.2. Disintegration due to crystallisation of halite of undisturbed sample of lime stabilised calcrete base course from Oshivello-Oshakati road on standing in the laboratory

The saturated paste electrolytic conductivity test (Netterberg, 1970) later developed into a standard NITRR / CSIR (1980) and national (NITRR, 1986) test method was found to meet the requirement for a rapid and simple but reliable site and laboratory test for soluble salts.

A.7.7 LONG-TERM PERFORMANCE

Although great concern was felt over this problem at the time, especially the general occurrence of a loose or weak layer under the asphalt – and more recent work (Netterberg and De Beer, 2012; De Beer et al., 2012) has shown how deleterious it can be – the road did not give any significant problems until about 1980 when a section failed under the, by then, quite heavy traffic. A superficial investigation found high PIs in the failed section and did not ascribe the failure to salt. The whole road was overlaid with 30 mm of asphalt and continued to give satisfactory service. Various cracks, both the block cracking typical of stabilisation as well as more unusual types and the occasional star pattern penetrated even the extra 25 mm of asphalt between ch. 181 and 233 within the first year, but never caused any problems.

The crucial role of surfacing permeability in preventing salt damage is now much better understood and the current rehabilitation solution of choice for the 'cracked mile' would now be a new-type Cape seal or double surface treatment (Netterberg, 1979a, 1979b). Indeed, such surfacings would now be recommended by the author as the original surfacing in preference to asphalt in practically all cases where a highly saline base is to be used.

A.7.8 CONCLUSIONS

- Chloride-sulphate water with a TDS of up to at least 5% m/v can probably be used for compaction of a lime-stabilised base of initially low plasticity but should not be used for curing.

- Curing water used two or three times a day for four days should probably be limited to a TDS of less than 1,8%.

- The theoretical < 6,7 mm EC of a lime or cement-stabilised base calculated from the initial EC of the material and the amount of salt added during compaction (or measured experimentally but without adding the stabilizer) should possibly be additionally limited to a maximum of 0,5 S/m.

- Other possible factors such as carbonation, inhibition by a penetrating primer, and over-compaction due to the rapid-hardening calcrete when stabilised with lime make it difficult to define the role of salt in the distress on this road more closely. However, the combination of curing an already saline base (estimated EC of 0,8 S/m after compaction) with a 5% saltwater was definitely implicated in the blistering and star cracking of the asphalt.

- Curing any stabilised layer with salt water is probably undesirable and curing any saline base with even fresh water is probably also undesirable unless the upper base can be prevented from drying out between water sprays.

- The surface "powdering" of the primed, lime-stabilised base on this road as well as the blistering and star cracking of the asphalt surfacing were similar to the salt damage experienced by pavements with unstabilised gravel and graded crushed stone bases.

A.7.9 ACKNOWLEDGEMENTS

The investigation of this road was carried out by the author, then at the NIRR, together with Mr E Lowe of the Namibian Department of Transport, Mr D Burger of the Consultants and Mr I L Jamieson of the NIRR.

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A.8 KLEINZEE-KOINGNAAS ROAD

A.8.1 INTRODUCTION

This 65 km-length of the then private road owned by De Beers Consolidated Mines between the Kleinzee and Koingnaas diamond mines along the west coast completed in June 1980 and proclaimed as a public road after the closure of the mines represents the first full-scale application of the preliminary findings of the Swartklip (Appendix A.6), Lamberts Bay (Appendix A.5) and Lüderitz and Haalenberg (Appendix A.4) experiments and other of the author's researches on the utilisation of plastic calcretes (Netterberg, 1971, 1982) and highly saline materials (unpublished). The road largely followed the alignment of an existing gravel road and involved the addition of a base (and in many cases a subbase), a seal, and gravel shoulders, all at the lowest possible cost, i.e. approximately R30 000/km – about one-quarter of the cost of a rural road to provincial standards – all in only 15 weeks, with a design life of 15 years and 0,05 MESA.

This was achieved by, among other innovations:

- The use of base course materials with a maximum PI of 20 instead of the then recommended maximum of 10 for a G5 gravel in TRH 4:1978, but with a minimum CBR of 60 at 98% MAASHO instead of the minimum of 45 at 95% for a G5.
- The use of seawater for the compaction of already saline mine tailings and calcretes, instead of the usual requirement of fresh water;
- resulting in total electrolytic conductivities (ECs) of up to 1,6 S/m – ten times the recommended maximum of 0,15 S/m for base course (Netterberg, 1979) and subsequently specified for state roads (CSRA, 1987, COLTO, 1998);
- but with accelerated construction to minimise upward salt migration and damage during construction,
- the omission of a prime coat; but using an emulsion tack coat applied at an increased rate, and
- the use of a rich, impermeable double seal to prevent upward salt migration and damage in service.

Although provision for a pavement with a G5 gravel base for a Category C road for less than 150 vehicles per day (vpd) in both directions and a structural capacity of 0,05 MESA was made in TRH 4:1978, such high PIs and salinities would never have been permitted by any South African road authority.

Moreover, the existing gravel road to be used as subbase and/or selected subgrade had been constructed of saline calcretes and maintained with seawater and with ECs of 0,6-1,4 S/m therefore contained a large reservoir of salt mostly in the form of NaCl.

As the design, construction and initial performance have been described by Spottiswoode and Graham (1982 a,b) only a brief summary thereof and the author's additional and long-term observations with respect to the use of seawater will be given here.

The traffic on this road since its opening in June 1980 has been light, averaging about 100 vpd in both directions with about 10% heavy vehicles (i.e. about 4 E80/day) over most of its length, rising to about 150 vpd by 1993. The first 8,5 km from Kleinzee received additional bus traffic averaging an additional 23 E80/day in both directions. The total traffic carried over the first 8,5 km over the first 13 years up to 1993 is estimated at about 0,1 MESA in both directions and about 15 000 E80 over the rest of the road. There were no axle load restrictions on this road and 100 kN axle loads were not rare. Whilst no further traffic information has been obtainable, it is estimated that the whole road carried at least 0,1 MESA in each direction over the 28 years up to the author's last inspection in 2008.

This extremely light traffic is a severe test for salt damage since such damage is invariably worst on and in areas of pavements carrying the least traffic.

km Zero was at the intersection with 8th St and 3rd Ave. at the then Kleinzee traffic circle, km 1,1 at the Kleinzee control gate, km 8,5 at the intersection with the Kommagas road and km 62,5 at the Koingnaas control gate.

A.8.2 CLIMATE

The climate of this area is cold dry desert (BWk) with frequent fog. The mean annual rainfall is approximately 75 mm, but is extremely variable and falls mostly between May and August. Weinert's (1980) N-value is about10, Thornthwaite's Moisture Index about minus 55, and the macroclimate is Dry for pavement design purposes.

A.8.3 PAVEMENT, MATERIALS, DRAINAGE, GEOMETRICS, AND RESEALS

Surfacing: Double surface treatment: Fog spray on nominally 6,7 mm chippings on penetration spray on nominally 19 mm chippings on tack spray. A total of 2,2 **ℓ**/m² residual bitumen was aimed at and not less than 2,0 **ℓ/**m² accepted. All bitumen was 60% anionic emulsion. Crossfall of 2% on seal.

Prime: None

Base: 125 mm Crushed Kleinzee diamond mine tailings consisting of rounded quartzite, granite, quartz and calcrete from km 0 to 18,5, thereafter natural gravel calcrete, all compacted to 98% MAASHO. Min. soaked 2,54 mm 98% MAASHO CBR 60, min. GM 2,0, max. PI 20, max. LSM 220.

Subbase: 125 mm Natural gravel calcrete compacted to 95%. Min. soaked 2,54 mm 95% CBR 20, min. GM 1,0, max. PI 20 (imported or in-situ).

Subgrade: 150 mm In-situ existing calcrete gravel road, min. CBR 7; existing EC 0,6-1,4 S/m, compacted to 93%.

Roadbed: Free-draining sand.

Width: 6,0 m Seal on 6,0 m base with 0,5-1,0 m shoulders compacted to 95% as part of the base out of the same material; max. batter of sideslopes 1:2

Drainage: Owing to the low rainfall no additional drainage was provided.

Geometrics: The vertical alignment of the existing gravel road was gently undulating and only a few improvements were made for a minimum stopping distance of 115 m for a design speed of 80 km/h. The horizontal curves were improved to a minimum radius of 400 m with the necessary superelevation.

Reseals: July-September 1986 4 mm slurry seal: well-rolled and, in places, thickened edge; August 1995: 13 mm double seal using cationic emulsion consisting of a single layer of chippings followed by a sand seal with a latex-modified binder; some base course rehabilitation, edge patching and more thickened edging. Up to at least May 2008 no further reseals and only minor yearly maintenance had been applied.

A.8.4 MATERIALS USED

The tailings used for the base over the first 18,5 km from Kleinzee had the following properties before compaction: GM 1,8-2,2; % < 4,75 mm 64-83, and PI 7-11. After compaction the % < 6,7 mm was 78-83%. The MAASHO OMC was 6,9-10,5% and the MDD 1 966-2 086 kg/m³. The 98% CBRs ranged between 64 and 113, but were mostly 60-70 and nearly all of it would have classified as a COLTO:1998 G5 material before compaction.

The calcrete used over the rest of the road for base course had a GM of 1,5-,7 with only one result less than 2,0% < 4,75 mm 26-81, PI of SP-15 and LSM of 20-270 but nearly all < 200. After compaction the % < 4,75 min was 32-97%, averaging about 70%. The MAASHO OMC was 14,0-27,0% and the MDD 1 459-1 839 kg/m³. Despite such low MDDs the CBRs were always relatively good; 98% MAASHO CBRs of more than 100 for calcretes with MDDs of less than 1 600 kg/m³ were not uncommon and 98% CBRs ranged between 56 (one only) and 170, with nearly all in excess of 80. All of the calcrete would have classified as a COLTO:1998 calcrete G5 or better before compaction.

The intended seal was a fogged single-size 6,7 on 13,2 mm double seal. However, neither the production facility at Kleinzee nor at Koingnaas using crushed boulders was able to meet the requirements for even the lower grade of stone recommended in SABS : 1083 (1979) and in practice the shape and grading of both sizes were poor, with oversize up to 26,5 mm in the case of the 13 mm chippings and, at times, a very high dust content in the 6,7 mm. It was not considered cost-effective to import chippings from the nearest commercial quarry at Springbok.

Clean, washed, 1-12 mm Koingnaas tailings (a mixture of well-rounded quartzite, quartz and granite, and calcrete) were used for the second layer of the seal in places.

A.8.5 SOLUBLE SALT CONTENTS

The soluble salt contents of the materials as represented by the saturated paste electrolytic conductivity (EC) of the particle size fraction passing 6,7 mm are in shown Table A.8.1, tested under the author’s supervision at the NITRR of the CSIR according to Method CA 21-74 (1980) (NITRR, 1980). Up to and EC of about 1,5 S/m the EC determined by this method is approximately equal to 80% of the percentage of NaCl or sea salt present. The initial salt contents of the calcretes, even before adding seawater are probably still the highest of any material so far used for base course for a full length of road in southern Africa. The calcrete subbase was of similar material to that of the base and was also compacted with seawater, yielding comparable ECs to that of the calcrete base.

With minus 6,7 mm ECs of 0,31-0,41 S/m the salinity of the tailings was much lower than the 0,57-1,62 S/m of the calcrete. Calcretes from borrow pits (BP) km 6 and 25 had mostly much higher ECs of 0,8-1,6 S/m than those from BP km 45 of 0,5-0,9 S/m.

Testing of the fraction passing 0,425 mm dried at 105-110°C prepared according to the DoT (1970) wet method A-1(a) but also involving decantation, for Atterberg limit testing of the base material before compaction yielded on average 1,11 times the EC of the minus 6,7 fraction for both tailings and calcretes (r = 0,97; n = 5 tailings, 26 calcretes). The pH of this fraction also tended to be higher than that of the minus 6,7 mm fraction.

Qualitative testing showed that the salts present were largely chlorides and $CaCO_3$ with lesser sulphate. However the pHs in excess of 8,4 and up to 9,9 of some samples indicated the presence of some $NaHCO_3$ and/or Na_2CO_3 in such cases. X-ray diffraction analyses of the saturation extracts of three samples of calcrete and one of tailings dried at 105°C confirmed the presence of halite (NaCl) as the dominant salt present but $NaHCO_3$ and Na_2CO_3 were not found.

Table A.8.1. Salt contents of base and subbase

km	Base Material	< 6,7 mm EC of base			< 6,7 mm EC of subbase after surfacing
		Before watering [1]	After mixing & watering but before compaction [2]	After surfacing [3] [4]	[3] [5]
		S/m	S/m	S/m	S/m
5,0-17,0	Tailings	0,17-0,29	0,13-0,41	0,41-0,63	0,77-1,18
19,0-63,0	Calcrete	1,05-1,14	0,57-1,62	0,76-1,84	0,64-1,64

Notes:

[1] Means : Tailings (km 5,0-15,0): 0,23 S/m (*n* = 3); calcrete (km 19,0-59,0): 1,09 S/m (*n* = 23)

[2] Means : Tailings (km 5,0-17,0): 0,25 S/m (*n* = 7); calcrete (km 19,0-63,0): 0,99 S/m (*n* = 23)
pH: Tailings 8,4-10,0 (*n* = 7, with 6 > 8,4 and 3 > 9,0); calcretes 8,0-8,7 (*n* = 23, with 1 > 8,4
Minus 0,425 mm EC: Tailings 0,21-0,65 S/m; calcrete 0,52-1,96 S/m

[3] Sampled 4 days to 2 months after surfacing

[4] Means : Tailings (km 5,0-17,0): 0,51 S/m: calcrete (km, 21,0-63.0) 1,31 S/m

[5] Means : (km 5,0-17,0): 0,99 S/m (*n* = 4); (km 21,0-61,0): 1,19 S/m (*n* = 11)

Comparison of the partial analyses of the seawater used (Table A.8.2) with normal open ocean seawater indicated it to have been average to more saline in composition. Although the pH found was abnormally low, the samples were only tested some four months after sampling.

Table A.8.2. Composition of seawater used

Component [1]	% m/v
Na^{+}	1,05-1,22
Mg^{2+}	0,13-0,15
K^{+}	0,04-0,05
Ca^{2+}	0,04-0,05
Cl^{-}	Much
SO_4^{2-}	Present
EC at 25°C (S/m)	5,05-6,06
Salinity [1] from EC (% m/m)	3,3-4,1
pH	7,1-7,2

Notes:

[1] Elemental analyses: NIWR / CSIR (*n* = 4)

[2] From Figure 3 of NITRR Method CA 21-74 (1980)

Four days to two months after surfacing a total of 16 samples were taken to check the as-built properties of the base and subbase. In spite of sealing within one week of compaction it was found that the tailings had increased in EC to 0,41-0,63 S/m and the calcretes to 0,76-1,84. representing an average increase of 0,27 S/m (r = 0,96) – Figure A.8.1. Those samples showing the greatest increase all possessed unusually high OMCs (> 20%) and low MDDs (< 1 550 kg/m^3) and/or had the lowest moisture contents after surfacing and/or had the highest subbase ECs. The outlier on Figure A.8.1 may also be due to a section which had to be recompacted; this was omitted from the regression analysis. However, the ECs after surfacing of those sections known to have been rewatered and recompacted are not among those plotting to the right of the regression line on Figure A.8.1.

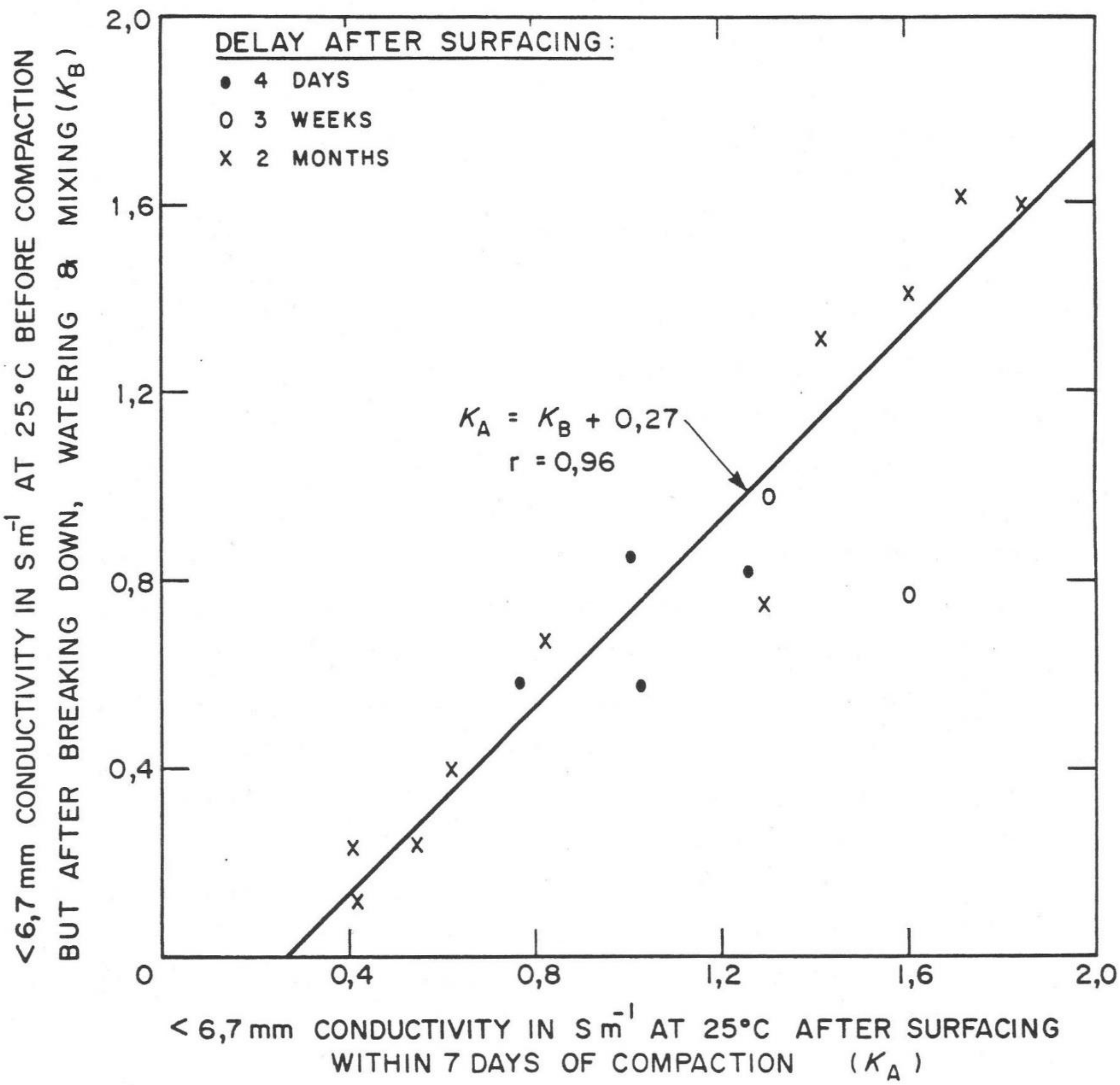

Figure A.8.1: Comparison between salt content of base before compaction and after surfacing on the Kleinzee-Koingnaas Road.

A.8.6 MOISTURE CONTENTS

Moisture (water) contents were also determined 4 days to 2 months after surfacing at the same time as the as-built samples were taken. These were 8,6-18,8% for the tailings and 9,7-21,3% for the calcrete. However, as some of these materials contained gypsum, which may lose 20% of its mass on drying overnight at 105-110°C, the true free water content of some of these bases was lower.

A.8.7 CONSTRUCTION METHODS

In order to prevent the base and subbase becoming loose due to salt migration it was required that the subbase be covered by the base within 14 days and the base sealed within seven days of compaction. This timing was achieved for all except 1 or 2 km of base between about km 20 and 25 and near km 45 which stood over a long weekend and had to be rewatered and recompacted. The subbase and base were compacted in half-width lengths of 500-1 000 m at a time, in order to carry the little traffic (about 50 vpd) through the job. The other half was compacted on the same or the following day. The shoulders were constructed as part of the base. According to the site staff no base or subbase ever became significantly loose even though salt weathering of lumps of calcrete was observed.

No prime was used and the emulsion tack coat at an increased application rate followed by the chippings was applied in half-widths in 5-6 km lengths mostly within seven days of compaction. The seal was designed to be as rich as possible and the possibility of bleeding was accepted.

Construction started at Kleinzee and progressed without any gaps or jumps.

The Contractor (LTA) was reported to have had no difficulty with any of these construction constraints.

A.8.8 PERFORMANCE DURING AND SOON AFTER CONSTRUCTION

This road was inspected frequently by the author, initially usually in the company of engineers from the Owners (De Beers Consolidated Mines) and Mr B Spottiswoode of Keeve Steyn, Consulting Engineers.

Due to the dusty chippings the top layer could not always be fogged immediately and was ridden off in places.

Within two months by September 1980 some looseness and edge breaking (fretting) of the edges of the seal had already been recorded due to salt crystallisation. However, the emulsion tack coat without a prime was adhering well by then and no surfacing was lost solely due to lack of adhesion.

Some longitudinal cracking and minor rutting at km 5,0, 10,3 and 17,65 apparently caused by settlement due to ponding of water after unusually heavy rains was observed in November 1980.

A.8.9 LONG-TERM PERFORMANCE

A.8.9.1 Problems, cost, maintenance, and performance

Three salt-related problems were encountered on this road since construction: “potholing” of the seal, edge

fretting and shoulder erosion. The classic form of salt damage, i.e. that of blistering and cracking of the surfacing in a starburst pattern was almost completely absent.

The mine engineering staff considered the road to be a definite success even solely from the point of view of the reduced vehicle maintenance and accident rate. The patching of the potholes was not regarded as a problem but the increased edge and shoulder maintenance was. Regravelling along and over the edges of the seal was carried out once in 1981 and the first 20 km again in 1982. The total costs of maintenance inclusive of labour, vehicles, fuel and materials up to the end of April 1986 were estimated by the Kleinzee staff as about R85 000. The average figure of R245/km/year is probably not significantly higher than the R215/km/year predicted by the Tarboton (1984) Model 1A for a road anywhere in the then Cape Province with similar climatic and traffic conditions, similar age and in a relatively uncracked condition. This model does not allow for potholing or edge fretting and excludes the cost of resealing. Using this model the minimum cost of maintenance was then estimated to be about R150/km/year even for a new road in an arid environment in its first year of service.

In 1986 after six years of service the road was provided with a thickened edge and a slurry reseal at an estimated total cost of about R600 000. According to TRH 4:1985 this was two years sooner than the typical 8-14 years of life expected from a double seal on a granular base for a Category C road to carry 0,2-3M E80/lane, but about average for such a road with a cemented base

This was very successful and very little maintenance was subsequently carried out until 1995 when a double seal with further thickened edging and some base course rehabilitation were applied.

Pavement evaluations of this road were carried out during February 1986 and in 1993 and visual assessments according to TMH 9:1992 in February 1992, June 1993 and May 2008 (Table A.8.3). However, mostly only those aspects concerning the performance of the road as affected by salt are considered here.

In June 1993 it was concluded that the road was still in a generally satisfactory condition, the thickened edge was very effective in minimising edge breaking, the slurry seal had also worked well, and that the road probably had a remaining life of about 10 years before some of it would require rehabilitation. However, attention to some cracking, shear failures, edge breaks, edge drops and shoulder erosion was recommended.

In July 1997 a visual assessment over 5 km lengths according to TMH 9:1992 was carried out at the author's request by the Department of Transport of the Northern Cape as part of their network evaluation. The general pavement condition was rated as good or very good with no reseal or fogspray being necessary and that either no or only routine maintenance was required over practically the whole length. Only the section between km 45,0 and 50,0 was rated as fair, with some structural maintenance of D4/E2 failures / potholes recommended at a B priority in a few places.

In 2002 and 2003 SEMANE on behalf of the mine concluded that the road was still in an "excellent general condition", the salt damage had abated, only minor routine work was required, that the 1995 reseal had exceeded design expectations, and that a reseal was not required, in 2004, but may be required in 2005.

In March 2007 a visual assessment of the road was carried out by AFRICON on behalf of the Northern Cape. It was concluded that the surfacing and structural conditions were both good, and that only a few short sections required resealing (apparently a total of about 200 m) and/or structural rehabilitation (apparently a total of about 50 m), with no immediate urgency. It was recommended that a fog spray be applied within the following four years, that the road markings be repainted, the signage be improved and that the operating speed be reduced to 80 km/h due to a substandard vertical alignment in places. The remaining useful and expected useful lives of

Table A.8.3. Summary of mean as-built electrolytic conductivity in 1980 and road condition in April 1985, February 1986, 1993 and May 2008 [1]

Segment Length	**EC**		**Both lanes**											**Koingnaas**		**Kleinzee**	
	Base [2]	**Sub-base** [3]	**Surfacing holes & Patches** [4]			**Edge breaking** [5] [6]			**Rutting** [5] [7]	**Sur-facIng width**	**Overall pavement condition**			**Rut depth [8]**	**Edge drop [9]+**	**Rut depth [8]**	**Edge drop [9]**
	1980	1980	1985	1993	2008	1986	1993	2008	2008	2008	1986	1993	2008	2008	2008	2008	2008
				[5]	[5]					[8]	[9]	[9]		[8]	[8]	[8]	[8]
km	S/m	S/m	No.	D/E	D/E	D/E	D/E	D/E	D/E	m	Degree	Degree	Degree	mm	mm	mm	mm
0,0-5,0	-	-	184	1/1	1/5	-	1/5	1/5	1/5	6,5	Good	Good	Good	0	10	3	20
5,0-10,0	0,22	1,00	446	0/5	0/5	2/5	2/5	1/5	0/5	6,3	Good	Good	V.good	0	20	2	10
10,0-15,0	0,27	1,0	420	0/5	0/5	3/5	1/5	1/1	1/5	6,3	Good	Good	Good	4	25	3	25
15,0-20,0	0,57	1,0	903	0/5	0/5	2/5	3/3	2/5	1/5	6,3	Good	Good	Good	5	10	2	15
20,0-25,0	1,43	1,63	1173	0/5	0/5	2/5	2/5	0/5	1/5	6,6	Good	Good	Good	2	15	5	15
25,0-30,0	1,48	0,9	1908	1/5	0/5	3/5	1/5	1/5	1/5	6,5	"Poor"	"Fair"	Good	5	40	0	10
30,0-35,0	1,35	0,8	1204	1/1	0/5	4/5	1/5	0/5	1/5	6,8	Good	Good	Good	8	10	3	10
35,0-40,0	0,83	1,5	84	2/1	0/5	3/5	2/5	2/1	1/5	6,3	Good	"Fair"	Good	4	15	5	25
40,0-45,0	0,88	1,16	70	1/1	0/5	2/5	1/5	2/1	1/5	6,2	Good	Good	V.good	6	15	7	50
45,0-50,0	0,78	1,5	2400	3/3	0/5	3/5	3/3	1/1	2/2	6,2	Good	"Fair"	Good	6	15	5	20
50,0-55,0	0,62	0,9	964	4/3	2/1	2/5	2/3	2/1	2/1	6,1	Good	"Poor"	Good	4	30	7	25
55,0-60,0	0,74	1,3	1802	3/2	0/5	3/5	3/3	2/1	3/1	6,2	Good	"Fair"	Good	6	30	9	25
60,0-62,5	0,69	0,6	402	2/2	0/5	3/5	3/3	2/1	1/5	-	Good	"Fair"	V.good	-	-	-	-
Mean	0,82	1,11	191	1/3	1/5	3/5	2/5	1/3	1/5	6,4	Good	F-G	Good	5	20	4	21

Notes to Table A.8.3:

[1] Assessed by F Netterberg (FN) assisted by Anglo American (AAC) or mine staff (according to TMH 9:1992 (in 1986 according to TRH 6:1985 later converted to TRH 9:1992 format). In 1993 km 0-30 in June by FN and AAC/mine staff and 30-62,5 in November 1993 by AAC staff

[2] Mean EC (*n* = 2 or 3) before compaction, but after breaking down, watering with seawater and mixing. Base: tailings km 0-18,5; calcrete km18,5-62,5. Mean EC for tailings 0,25 S/m (*n* = 7), calcrete 0,98 S/m (*n* = 19)

[3] Mean EC (*n* = 2) to two decimals, single result to one decimal; before compaction, but after breaking down, watering with seawater and mixing. All subbase calcrete. Mean EC 1,11 S/m (*n* =15)

[4] Holes mostly 20-50 mm in diameter to a depth of less than 20 mm (recorded as Degree 1 or 2 surfacing failures/patching and not potholes)

[5] Visually assessed best average according to TMH 9:1992, using five-fold scale of Degree and Extent (D/E), e.g.: Degree 0 = None, 1 = Small, 3 = Warning, 5 = Severe; etc. Extent 1 = Isolated (< 5% of segment length), 3 = Scattered or over limited portion, 5 = Extensive occurrence over entire segment (> 50% of length)

[6] Degree 1 < 50 mm wide, 3 approx. 150 mm, 5 >300 mm (hazardous)

[7] Degree 1 < 5 mm deep, 3 10-15 mm, 5 > 20 mm (hazardous)

[8] Surfacing widths, rut depths and edge dropoffs**,** are **spot values** at the approx. km points at the end of the section evaluated visually. The overall pavement condition is the best average for the whole segment.. Rut depths are the maximum depths measured in the outer wheelpath using a 2,0 m straight edge and a 20 mm-wide wedge. Degree 1 Edge dropoff = < 50 mm

[9] "Poor" or "fair" only due to a few localised (i.e. Extent 1 or 2) structural failures **not** due to salt; most of segment fair or good, respectively

the road were estimated at about 7 and 10 years respectively, and the current replacement cost to be about R25,4 M. It was further recommended that the road should be proclaimed as a provincial main road.

In the May 2008 visual assessment the author noted that the present seal up to about km 21,2 was a 7 mm chip with a 13 mm thereafter, that both were still in good condition but that the 13 mm seal was the better, and that the surfacing width of 6,1-6,8 m was still more than the required design width of 6,0 m. However, stone loss had reached a warning level between km 10 and 20 and a reseal or at least a fog spray should be considered within the following two years.

The occurrence of cracking of all types was negligible, pumping, potholes and shear failures were absent and the degree and extent of rutting were generally negligible with only short sections of Degree 3 (i.e. at a warning level of 10-15 mm between km 55 and 60 (Table A.8.3). The total length of patching in the outer wheelpath of each lane did not exceed about 1 km. The extensive edge patching was due to repairs to the previous edge breaking and the installation of a thickened edge. These had been very successful, with edge drops of generally less than 30 mm. Only the riding quality was rated as fair to good, although acceptable for this category of road, but the shoulders were generally in a warning condition due to being too narrow and/or too steep.

The road had undergone little or no deterioration since 1997, the general pavement condition was rated as good or very good over the whole road and only a fog spray, repainting of the white centre and barrier lines and yellow edge lines and some attention to the shoulders were recommended.

A.8.9.2 "Potholing"

A few patches were necessary for example at around km 18,7 during the Contractor's maintenance period due to the poor and dusty chippings used.

After about two years of service, patching of small, shallow holes in the seal became necessary, peaking after about 3 years (Figure A.8.2). This was mostly due to a small proportion of inferior calcrete chips which

Figure A.8.2. Showing small white surfacing holes ("potholes") and black bleeding spots, white calcrete chips, and a few black patched holes in seal at km 56,8R after 12 years in February 1992 (cumulative total of 342/km holes and patches around this point up to April 1985). Mean EC of km 55-60 segment of calcrete base 0,74 S/m and subbase 1,26 S/m after compaction; Degree/Extent of holes and patches in 1993 3/2; overall pavement condition in 1992/3 fair, (good in 1986 and 2008).

disintegrated out leaving small (20 mm) holes in the seal, but also to a proportion of oversize or dusty chips. Salt crystallisation then loosened the base in and around the hole to a maximum depth of 20 mm, which led to the formation of small "potholes" averaging about 50 mm in diameter, (i.e. Degree 1 according to TMH 9:2016) but which rapidly grew to 150 mm (i.e. the warning level of Degree 3) or more if not patched. As the mine staff were well aware of the importance of maintaining the impermeability of the seal these potholes were patched by a small team consisting of a driver with two labourers and a light delivery truck. Patching took place from December 1982 until March 1985, initially every day, later only once or twice a week and then only once a year after the reseal in 1995.

Such small, shallow holes not significantly affecting the base are regarded as surfacing and not structural failures (TMH 9:1992, 2016) but are referred to here as "potholes" or simply as holes for convenience. The possible effect of even a small proportion of inferior chips in the chippings in a thin seal does not appear to be generally realised: at a typical application rate of about 1 500 of 19 mm chips/m^2, only 0,1% inferior chippings means an average of 1,5 potential holes/m^2 in the seal or about 9 000/km on a 6,0 m-wide seal. If the base is of poor quality and/or contains a large amount of salt this may mean one or two "potholes"/m^2 which in time could progress to real potholes.

A survey carried out by the mine in April 1985, i.e. after 5 years under traffic, showed that there were a total of 11 960 "potholes" and patched "potholes" in a length of 62,5 km, representing an average of 191/km, or approximately 20/100 m. Some stretches of road had as few as 18/km, while the highest recorded was 469 in a 200 m length, i.e. 2 345/km. The actual spacing of the potholes and patches varied tremendously, but in the worst areas in February 1986 a spacing of 1 or 2 m was common, with localised areas of 0,5 m (e.g. at km 25,80 R, 26,4 R, 27,16 R) or even almost continuous patching being noted, such as along the centreline at km 23,0, 33,0 and 35,0. The generally worst areas were probably around km 47,0 R and 57,0 R. At km 47,0 the right lane had edge breaks of Degree 3 (i.e. 50-150 mm in width) and potholes averaging 100 mm in diameter on a spacing of 2 m in general and 1 m in the outer 2 m of the lane. The larger potholes were empty and possessed a hard bottom but the smaller ones were still filled with loose, saline, fluffed base course. Only one single patch could be found over a stretch of 100 m in the left lane. The general count for patching and potholing over this general section of road was 686/km. At km 57,0 which had a general count of 514/km the 200 mm diameter patching in the left lane averaged a 3 m spacing and in the right lane 0,7 m in general and 0,5 m in the outer wheeltrack. The right lane here had lost much of its top layer of wheeltrack chippings.

The incidence of surfacing "potholes" and patches increased sharply from the 184-420 / km over the first 15 km where tailings with an average EC of 0,35 S/m had been used for base to mostly over 1000 / km after km 20 where calcrete with an average EC of 0,98 S/m was used.

In general, the **greatest** incidence of patching and potholing was associated with an obviously leaner than average seal, and/or complete absence of the upper layer of chippings, and/or a higher proportion than average of soft calcrete chippings, an EC before compaction of greater than 1,0 S/m, and a calcrete base. In some cases the damage was evenly spread across the entire lane and at others confined to a strip along the centreline, or in one of the wheeltracks, or between the wheeltracks, or between the outer wheeltrack and the edge of the seal. The right (Kleinzee-bound) lane also had a greater incidence than the left lane. This may be because until 1986 Koingnaas received all its supplies, including all fuel, via Kleinzee so that the traffic using the right lane was more lightly loaded. (The sole export from Koingnaas was diamonds!).

The **lowest** incidence of patching and potholing occurred where the surfacing was richer than average (and often showed slight to moderate (Degree 2 or 3) bleeding in the wheeltracks), and/or where both layers of the seal were intact, and/or where a low proportion of calcrete chippings could be observed in the seal, and/or a lower base course salt content, and/or a tailings base in the same lane patches and potholes were often observed to cease abruptly at a change in surfacing from lean to rich (e.g. km 47,09 L) and to start abruptly where the bottom layer of chippings was missing (at km 58,27 L and 60,4). This stretch of road all had fairly uniformly moderate salt contents of 0,6-0,9 S/m before compaction and high values of 1,0-1,6 after surfacing.

At the same km point they were often confined to or much more common in the leaner lane (e.g. km 45,0; 47,0 R and 49,0 R) or were much common in the lane which had lost most of the upper layer of chippings (e.g. the right lane at km 49,0 and 57,0) or which was on the lower side of a superelevated curve (km 46,290 R). (At least at the same km point with no superelevation the salt contents in both lanes should have been approximately equal.

The quality of the seal was therefore the most important single factor influencing the damage.

The observations at km 58,27 and 60,4 showed even a rich single 6 mm seal to be inadequate at such high salt contents (> 0,5 S/m) if individual chips weather out and more general observations (e.g. at 49,0 and 57,0 that such a single 19 mm seal is also inadequate. On other hand at km 58,27 it was also noticed that the seal was undamaged where only some of the top layer of chippings had weathered out leaving the bottom layer intact. A single seal with a fog or a Cape seal is therefore feasible on such bases.

No evidence for widespread loss of chippings after the fog spray was seen (other than the weathering out of individual calcrete chippings).

Up to km 18,0 the incidence of patching and potholes varied between 43 and 124/km. From about km 18,5 there was a change from the tailings base to a calcrete base from a borrow pit at km 25. From km 18,0 to km 31,8 the incidence of patching and potholes was at its highest, varying between 109 and 2 345/km. The highest incidence occurred in short stretches of a few hundred metres at a time and was often confined to one lane: in one lane there might be 200 mm diameter patches or potholes at a spacing of 1 m, while in the other there might be none at all. This was often interspersed with stretches of several km with a comparatively low incidence of patching and potholing. As the road was surfaced in half widths this strongly suggests that the surfacing is at fault and in particular individual truckloads containing an abnormally large proportion of poor chippings.

A closer scrutiny of the two km points with the highest salt contents (km 21,0 and 29,0) is instructive. Both had ECs before compaction of 1,6 S/m and 1,7-1,8 after surfacing. km 21,0 Fell within the section of road from km 18,8 to 21,4 recorded as having an average of 109 potholes and patches/km and 29,0 within the section from 27,9 to 30,3 with 170/km. Yet within 50 m on either side of km 21,0 itself there were **no** potholes or patches at all in the left lane and only one in the right lane, i.e. 1/100 m. Similarly, around km 29,0 there were none at all in the left lane but patches 200 mm in diameter on a spacing of 3 m in the right lane. In both of these cases the salt contents in both lanes should have been the same, and showed that it is possible to construct a double seal without a prime which will give perfectly satisfactory service for at least six years, even at salt contents of 1,6 S/m or more before compaction and 1,8 S/m or more after sealing.

The relatively poor correlation was between the salt content of the base and the incidence of patching and potholing (Figure A.8.3) shows that salt was not the only factor responsible for the damage. The salt content samples taken before compaction are believed to be representative of the full width of the road including shoulders at a particular km point, while those taken after surfacing were mostly taken more or less along the centreline. Between km 35,0 and 46,1 the incidence of potholing and patching were at their lowest for the whole road at only 14-18/km, whereas the ECs varied between 0,68 and 0,99 S/m before compaction and 0,83-1,31 after surfacing. This section even had a calcrete base. The remaining length of road had similar ECs but more than 100 and mostly several hundred patches and potholes/km. Even the tailings base with an EC of 0,13-0,41 before compaction and 0,4-0,63 after surfacing had 43-124/km. The lowest ECs of all (0,13 S/m before compaction and 0,42 after) occurred with tailings base at km 7,0 (124 patches/km) and 9,0 (119,/km). This is comparable to the EC of the seawater calcrete base at Swartklip (Appendix A.4) and much lower than the high salt calcrete sections at Haalenberg, the Cape seal surfacing of which has given over 30 years of service without requiring a single patch (other than after sampling).

However, whilst it is clear from Figure A.8.3 that the **risk** of damage did increase with salt content and that an EC of more than 1,0 S/m before compaction or 1,3 after surfacing was associated with an increased incidence of damage, approximately 100/km were present even at the COLTO:1998 limit of 0,15 S/m for base course.

The method of patching used was to remove all the loose material from the pothole, to fill with 19 mm chippings, hand compact, penetrate with emulsion, add a layer of 6 mm chippings, penetrate again with emulsion, and to blind with crusher dust. This procedure was 100% successful and yielded rich patches which never gave further trouble.

An unusual type of bleeding mostly of Degree and extent 3-5 in the form of small (20 mm) black spots was noticed over most of the road in 1986 and 1992/3. Such bleeding is usually due to the punching of especially oversize chips into a soft (or wet) base and also occurs on roads with non-saline bases.

Twenty-one air permeability tests essentially according to the ASTM D3637-84 pressure system method, but with the earlier grease ring without the additional weight on the surfacing, were carried out as part of the pavement evaluation using a Soiltest Asphalt Paving Meter. Wherever the surfacing was not potholed and even where narrow longitudinal cracks were present it was found to be completely impermeable to air at a pressure head of 6 mm of water. At all except a few cracked, very rough or very lean surfacings it was also impermeable at a head of 20 mm (the highest practicable). At such sites permeabilities of up to 1,5 mℓ/cm^2/minute were obtained. Within the limitations of such tests it therefore appears that the surfacing on this road more than satisfied the requirement (Netterberg, 1979a) of a maximum permeability of 1,7 mℓ/cm^2/min. at

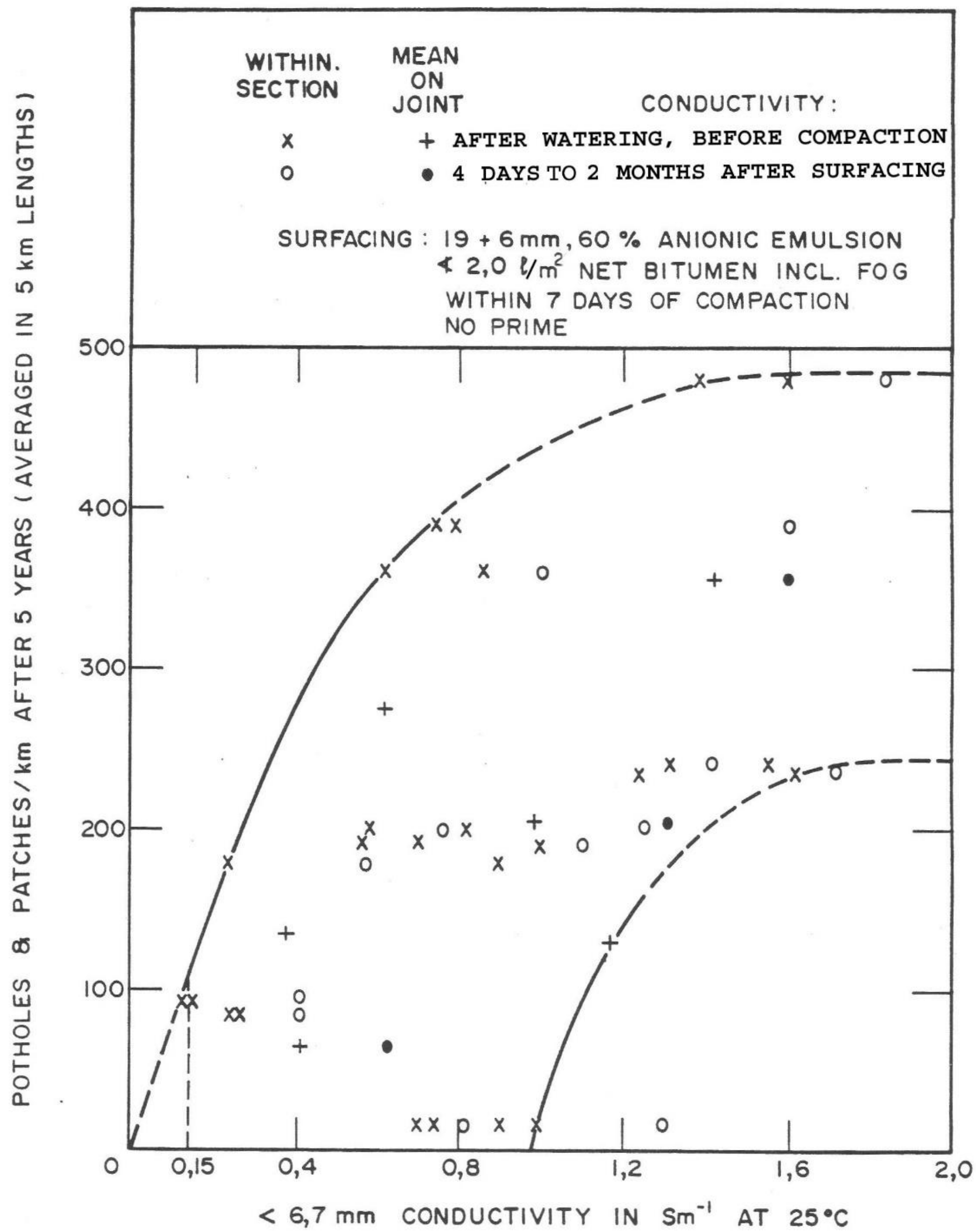

Figure A.8.3.

Comparison between salt damage in the form of “potholing” to seal and salt content of basecourse on the Kleinzee Koingnaas road.

a head of 6 mm of water for resistance to salt damage. As soon as the seal is punctured due to the loss out of a chip the permeability naturally becomes much greater than this. Measurements of water permeability at some of the sites using the Zimbabwe (1977) STP.343 metricated version of the California 341:1978 grease ring method gave readings of 10-19 mℓ/min. wherever a zero air permeability at a head of 6 mm had been measured. In this method, depending upon the texture depth of the surfacing, up to 20 mℓ of water is required to fill the spaces between the protruding chippings and to just cover them with 1 mm of water. A reading of anything less than 20 mℓ/min. is therefore normally regarded as impermeable.

A.8.9.3 Edge breaking (fretting)

Fretting of the edges of the seal (Figure A.8.4) had been expected and it started within about six months of sealing, even on the relatively less saline tailings section notably at km 17,0 L with an EC before compaction of only 0,24 S/m and after surfacing of 0,57 S/m. The edges of the seal were also loose in places (e.g. at km 18,7 R) and could easily be removed. As the seal was generally sprayed wider than the specified 6,0 m it was thought that this might have been aggravated by inadequate brooming of the base under the edges of the seal. (The shoulders were constructed as part of the base with the same material.)

Figure A.8.4. Showing Degree 2-3 edge breaking at km 59 after 12 years in February 1992 and coarseness and tightness of calcrete gravel base and localised strip of partly disintegrated white calcrete chips (cumulative total of 273 holes and patches/km around this point up to April 1985). Mean EC of km 55-60 segment of base 0,74 S/m and subbase 1,26 S/m after compaction; Degree/Extent of edge breaking 3/3 (3/5 in 1986); overall pavement condition in 1992/3 fair (good in 1986 and 2008).

A mixture of crusher dust and clayey soil was applied along the edges and just over the edge of the seal during 1981 in an attempt to protect the edges from fretting. This was quite successful but did not last and it was felt that an annual reinstatement of this kind during the winter would be advisable. Owing to the narrow seal buses tended to drive close to the edge which led to rapid erosion of the crusher dust-soil mixture.

During July-August 1982 the shoulders were reinstated up to km 20 by ripping, regravelling with tailings, watering with fresh water (TDS of 0,15%) and recompaction so that they covered the edge of the seal. This was very successful and no fretting was observed in February 1986 where the edges of the seal were thus still covered. However, this practice was discontinued because of a shortage of labour and breakage of windscreens by the coarser tailings material. Only spot maintenance of shoulders was carried out elsewhere on the road.

In January 1983 it was estimated that the width of the seal was being reduced at an average rate of about 25 mm/year, which was no more than had been expected. At km 62,5 where one edge of the seal was covered by the shoulder gravel no fretting was taking place, whereas on the opposite side a 20 mm width of seal had been lost in the six months since the shoulder had been recompacted.

By February 1986 edge fretting averaging Degree 2 to 3 (i.e. edge breaks of up to 150 mm in width) was general over most of the road. In a few places such as at km 25,0 R, 27,0 R, 31,0 L and 35,0 R it had reached Degree 4 (150-300 mm) and at two places (km 18,4 R and 46,290 R) to Degree 5 (>300 mm). Fretting was generally worse on vertical crest curves and on the inside of horizontal curves, probably due to vehicles tending more towards the edge as well as wind and water erosion at such places and on the edge of the right-hand lane, probably because the prevailing wind is from that direction. Out of 37 measurements of seal width, 14 were less than 6,00 m but only two less than 5,90 m. Actual values ranged between 5,81 and 6,40 m.

The amount and degree of edge fretting and the incidence of patching and potholing were not highly correlated. For example, between km 37,0 and 45,0 which had the lowest incidence of the latter (14-18/km) the Degree of edge fretting varied between 2 and 4 and averaged 3, whilst the edge drops varied between 0 and 25 mm and averaged about 20 mm. The change of base course borrow pit at km 40 had no influence on the frequency of patching nor the degree of edge fretting, but the average edge drop was halved from about 25 mm to about 12 mm. The salt contents over this section were all similar at 0,7-1,0 S/m before compaction and 0,8-1,3 S/m after surfacing.

The salt contents and the Degree of edge fretting and the edge drops also did not appear to be highly correlated. For example, at km 21,0 and 29,0 which had the highest salt contents (1,6 S/m before compaction and 1,7-1,8 after surfacing) the Degree of edge fretting varied between l and 3, averaging 2 (250 mm) and the edge drops between 0 and 10 mm, averaging 5 mm.

The engineering staff at Kleinzee were very strongly of the opinion that it was a combination of the narrow seal, the bus traffic and its wind of passage, the strong wind which caused vehicles to veer towards the edge, and the salt which was the cause of the problem and not just the latter. A wider seal of 7,0 m, but preferably 8,0 m was considered advisable under these conditions (G. Oberholzer, De Beers, 1986 pers. comm.).

However, in general, edge fretting of Degree 3 or more seemed to be associated with salt contents before compaction of more than about 0,5 S/m (all of the calcrete and some of the tailings). All the cases of Degree 4 for which results were available (km 25,0, 27,0, 31,0 and 35,0 had ECs before completion of 1,2-1,6 S/m and came from BP km 25. As a rough guide it therefore seems likely that for this seal, traffic and environment, and for these materials (crushed tailings and natural gravel calcrete), significant (more than Degree 2) edge fretting can be expected at an EC of more than about 0,5 S/m and that Degree 4 edge fretting is likely at an EC of more than about 1,0 S/m.

To what extent the shoulder maintenance affected the reliability of the above observations and conclusions is not clear. However, the edge fretting could only have been worse without such maintenance. Most of the patching effort had been directed at the potholes in the seal and little of the edge fretting had been patched.

Possible solutions to the edge fretting problem considered included:

- Strengthening of the edges and reduction of the moisture and salt migration at the edge of the seal by installing a vertical moisture barrier by means of an old-fashioned thickened edge traditionally used to prevent water infiltration.
- Widening of the seal sufficiently to provide a sacrificial edge.
- Reduction of the salt content of the shoulder material by regravelling with material of low salinity and compaction with fresh water.
- Reduction of the moisture and salt migration at the edge of the seal by means of a 'mulch' of loose material covering the edge.
- A combination of several of these measures.

The measure adopted was the patching of the edge fretting and the provision of a thickened edge comprising a slot approximately 100 mm wide at the top, 50 mm at the bottom and 75 mm deep filled with a premix of 1 : 1 crusher sand and tailings chips mixed with : 150 **ℓ** of emulsion/m^3 and 2% cement and compacted with a vibrating pedestrian roller. All of this was covered by a 4 mm thick rich bitumen slurry which was well-rolled and was more elaborate than the traditional thickened edge made by simply filling the slot with clean 2-7 mm gravel and saturating it with bitumen (Mitchell et al., 1979) but provided a combination of mechanical strength and a vertical moisture and salt barrier and was very effective. (It had previously been shown (Netterberg, 1979b) that a slurry seal alone cannot be relied upon to repair a salt-blistered surfacing.)

A.8.9.4 Shoulder looseness and erosion.

No systematic measurements of shoulder looseness were made. However, the upper part of the shoulders were observed to become loose in places within six months of construction (Figure A.8.5).

Figure A.8.5. Degree 1-3 edge fretting and surface shoulder looseness soon after construction.

This section (from approximately km 20-25) stood over a long weekend and had to be rewatered resulting in the highest salt contents on the whole road before compaction (1,3-1,6 S/m) and some of the highest after surfacing (1,6-1,7).

The shoulder edges were apparently reinstated in 1981 and again (up to km 20) during July-August 1982. Fresh water (and regravelling) was used for compaction in this case and seemed to have led to less looseness and a reduction in edge fretting. In February 1986 significant shoulder looseness was noticed at km 31,0 R (edge drop of 20 mm and shoulder generally loose to 25 mm) 47,0 R (20 mm edge drop) and 59,0 R (15 mm edge drop). The salt contents before compaction at these points were 1,56, 0,79 and 0,63 S/m, respectively. No results were available after surfacing, but from Figure A.8.1 they can be expected to have been about 0,3 S/m higher in each case.

The measured edge drops on this road were generally not excessive and up to 1986 varied between 0 and 25 mm with an average of about 15 mm, and included any looseness of the shoulder at the edge of the seal. In some cases less erosion was evident at the very edge of the seal than about 300 mm from the edge. In such cases the measured edge drops underestimated the thickness of the shoulder which had been removed by erosion. In 2008 the edge drops varied between about 10 and 25 mm with an excessive drop of 50 mm only being recorded at km 45.

Although there did not seem to be a good correlation between salt content and shoulder looseness or erosion, in general it appeared that significant looseness and erosion was likely where the EC before compaction exceeded 0,4-0,6 S/m and perhaps 0,6 S/m after surfacing. It will probably become more quickly apparent and more severe at higher ECs (probably more than 1,0 S/m).

These observations must be tempered by the knowledge that the shoulders were regravelled several times since construction.

Shoulder erosion is of course not a problem confined to saline materials, but one of the general problems to be considered when choosing a shoulder material. Saturated paste pHs in excess of 8,4 (Table A.8.1) also indicate such soils to possess an exchangeable sodium percentage (ESP) of 15 or more and therefore to be potentially dispersive and more erodible if leached (United States Salinity Laboratory Staff, 1954) by rainfall. Such high pHs were mostly confined to the tailings (used up to km 18,5) with pHs of 8,4-10,0 with most over 8,4 and only one calcrete (at km 25,0) with a pH of 8,7 and an EC of 1,42 S/m had a pH in excess of 8,4. (However, the higher salinities of most of the calcretes would also have tended to depress the pH.)

Comparison of the test results available with suggested specifications for calcrete wearing courses for unpaved roads (Netterberg, 1978, 1982) indicates that only the material at km 31,0 was adequate for wearing course. All the other materials lacked sufficient plastic binder and would be expected to have become loose and to erode under traffic. It can therefore only be said that the presence of salt especially with a pH in excess of 8,4 probably aggravated any naturally low erosion-resistance possessed by the material itself, and that even a highly suitable material is likely to be affected by the higher salt contents (probably more than 1,0 S/m).

A.8.9.5 Blistering

Blistering of the seal was almost completely absent and except for one area was confined entirely to minor small blistering of the edges of the seal which were sprayed wider than the 6,0 m specified. Such areas may also not have been cleaned properly before spraying. The only other area blistered during the pavement evaluation in February 1986 was at km 31,0 where small blisters or potholes averaging 50 mm in diameter occurred in the outer 300 mm of the seal in the right lane only. This same area also had edge fretting of Degree 3 (R) and 4 (L), an edge drop of about 20 mm and general shoulder looseness of about 25 mm. The EC recorded at km 31,0 before compaction was 1,56 S/m and the area was in a general length of road with 519 patches and potholes/km. However, at km 31,0 they were confined entirely to the right lane. No EC was measured after compaction, but it was probably nearly 2,0 S/m – probably the highest on the whole road.

This salt content of about 2,0 S/m after surfacing is probably near the limit of salt tolerable by a double seal on a coarse natural gravel calcrete base under these conditions.

A.8.9.6 Cracking

Minor localised longitudinal cracking with or without associated shallow rutting in the outer wheeltrack occurred at km 5,04 L, 25,0 R, and 53,0 R in November 1980 after about six months of service. A limited investigation of these cases indicated that they were due to fill and roadbed settlement of the underlying old gravel road where some ponding of unusually heavy rains had taken place. A few more cases of such cracking were recorded during the pavement evaluation in February 1986.

The only cracking which could possibly have been due to salt was at km 47,0 where block to random cracking of Degree 2 occurred on a spacing of about 0,5 m across both lanes. Longitudinal cracks of Degree 2 also occurred in the outer 0,5 m of both lanes. The incidence of potholes averaging 100 mm in diameter and patches averaging 200 mm in the outer 2 m of the right lane was about the worst seen on the whole road, but only one could be found in the left lane within 50 m on either side of km 47,0. The construction records showed that the base before compaction was exceptionally coarse (GM 2,7) was SP and had an EC of 0,79 S/m. This type of cracking is usually associated with shrinkage of the surfacing or base (Grant and Netterberg, 1983) and the blistering and star cracking typical of salt damage (Netterberg, 1979b) were absent. Several cases of similar cracking on non-saline calcrete bases of low plasticity are known to the author.

Apart from the rare cases of blistering reported in the previous section the very little cracking on this road could not therefore be ascribed to salt.

A.8.9.7 Shear failures

One shear failure in the outer wheelpath at km 27,2 L on the edge of a high fill occurred after exceptionally heavy rains of over 120 mm in 1983. Failure occurred on a horizontal plane on or just into the highly micaceous and slippery subbase which had formed the surface of the old gravel road. Several other similar failures between km 25,7 km and 26,5 were recorded during the pavement evaluation in 1986. None of these could be ascribed to salt.

As at May 2008 no further reseals had been applied and it was said that only localised patching of potholes had been carried out once a year and excessive growth of vegetation on the shoulder removed.

A.8.10 PAVEMENT EVALUATIONS

The detailed results of the pavement evaluations carried out 1986 and 1993 and the visual evaluation in 2008 are not described here. However, apart from a few localised shear failures such as those mentioned above, rutting practically never exceeded 10 mm and averaged less than 5 mm, cracking of any description was minimal and confined to a few longitudinal cracks, very rare blisters and the one case of block cracking already described. Permeability measurements showed the seal to be completely impermeable where not holed.

Riding quality measurements by the NITRR using a vehicle-mounted linear displacement integrator (LDI) carried out after 12 years on May 21, 1992 over 614 100 m intervals showed the whole road to have a good riding quality of a minimum 80 percentile terminal present serviceability index (PSI) of 2,0 for a Category C or D road (Table A.8.4).

Table A.8.4. Riding quality in 1992

Lane	Measured (PSI)						Recommended (TRH 12: 1985)		
	Min.	**Max.**	$\bar{x}$	**80%-*ile***	***s***	***n***	**Severe Max.**	**Warning**	**Sound Min.**
To Koingnaas [1]	1,6	3,9	3,00	3,3	0,38	614			
							1,5	≥1,5-≤ 2,0	2,0
To Kleinzee	2,0	3,8	3,00	3,3	0,35	614			

Note:

[1] Only a total of about 300 m of the actual measurements in the Koingnaas-bound land fell within the “warning” range.

Repeat measurements In April 1996 also gave a mean PSI of 3,0 in both lanes.

The average riding quality was thus the same as the 3,0 recorded after six years in June 1986 using the NITRR PCA Roadmeter in 500 m intervals (*n* = 121), although the minimum PSI recorded then was 2,2 in both lanes.

Apart from the patching, “potholing”, edge fretting and a few localised shear failures the seal and pavement were thus in good condition (Figures A.8. 6 and 7) and the pavement was expected to easily exceed its design life of 0,05 MESA in 15 years. The traffic carried over the first 10 km had already probably reached this in six years with no significant structural distress to the pavement itself.

Figure A.8.6. Typical view of road towards Kleinzee in good condition after 28 years in February 1992 from km 31,0 over km 30-35 segment with mean calcrete base EC of 1,35 S/m and subbase EC of 0,79 S/m after compaction. Small (20 mm) Degree 4 / Extent 5 black bleeding spots. Overall pavement condition good in 1992/3, 1986 and 2008; edge breaking Degree/Extent 1/5 (4/5 in 1986); number of surfacing holes/patches 1 204 (i.e. 241/km) as at April 1985 (1/5 in 1993, none (0/5) in 2008).

A.8.11 CONCLUSIONS

- Seawater can be used successfully for the compaction of already saline, coarsely graded (GM ≥ 2,0) natural and crushed gravel bases with OMCs in the 10-25% range in a cold dry desert climate to be used under an impermeable double surface treatment without a prime, with salt contents in the surfaced base course up to an equivalent < 6,7 mm EC of about 2,0 S/m.
- In order to limit the upward migration of excess salts from the construction materials by covering each layer as soon as possible after completion the following clauses were added to the Conditions of Contract:
  - The base course had to be completed within fourteen (14) days of completion of the subbase.
  - The bituminous double seal had to be completed within seven (7) days of completion of the base course.
  - The Contractor was required to accept full responsibility for any defects that might have occurred due to his failure to comply with the above.
  - A prime coat was to be omitted and the first spray of 60 or 65% bituminous emulsion had to be applied directly onto the completed base.

Figure A.8.7. Typical view of road towards Kleinzee after 28 years in May 2008 from km 31,5 over km 30-35 segment with mean base EC of 1,35 S/m and subbase EC of 0,79 S/m after compaction. General pavement condition good (and in 1986 and 1993); no edge breaking (0/5) (1/5 in 1993; edge drops about 10 mm; surfacing holes and patches 0/5 (1/5 in 1993); rutting about 10 mm (1/5); surfacing width 6,8 m.

- The base had to have been well swept of all dust and loose material and had to be either dry or slightly damp, whichever proved to provide the best adhesion to the surfacing.
- The Contractor generally had no trouble in meeting these requirements.
- The road has been successful in meeting the Owner's requirements of an adequate low volume surfaced road at the lowest possible initial cost and without excessive maintenance costs.
- The alternatives of providing a gravel or crushed stone base of normally acceptable salinity and compacting it with fresh water would have been prohibitively expensive.
- No salt damage was encountered during construction.
- However, “potholing” and fretting of the edges of the seal partly due to salt crystallisation began a few months after construction.

- The usual form of salt damage, i.e. blistering and. cracking of the surfacing was almost completely absent.
- The shallow "potholing" with an average diameter of about 50 mm was largely due to the combined effects of a small but variable proportion of soft calcrete in the sealing aggregate leading to holes in the seal and the high salt content of the base, aggravated in places by a lean seal and/or absence of one of the layers of the seal. As such, it was a surfacing rather than a structural failure.
- The frequency of "potholing" was largely independent of the salt content of the base within the range of EC of 0,15-1,0 S/m after compaction or 0,4-1,3 S/m at sealing.
- However, salt contents in excess of 1,0 S/m after compaction or 1,3 S/m after sealing were associated with an increased frequency of "potholing".
- The edge breaking (fretting) was due largely to the combined effects of salt action in loosening the edges of the seal and the adjacent shoulder material and bus traffic on the narrow 6,0 m seal, probably aggravated by most of the base course being unsuitable also as shoulder material and some of it also possibly being potentially dispersive.
- The "potholing" was not regarded by the mine engineering staff as a significant problem, provided that it did not recur on the patch, but maintenance of the shoulders and edge seal was and they considered that under such circumstances a seal width of 7,0 m but preferably 8,0 m was advisable.
- The average maintenance costs over the first six years of R245/km/y were 13% higher than the R215/km/y predicted for any road under similar conditions of climate and traffic, but without the high extent of "potholing" and edge breaking experienced.
- The road was resealed about two years sooner than would have been expected for a double seal under similar traffic conditions.
- The "potholing" and the necessity for an early reseal could largely or entirely have been prevented by the use of better-quality chippings containing no very weak aggregate, less dust, and a better grading. However, this could not be economically provided. (The standard aggregate crushing tests also do not detect such small quantities of weak aggregate in a mixture with much stronger material.)
- Fretting of the edges of the seal into the design width can be prevented by grading the shoulders over the edge of the seal, (not recommended due to windscreen damage), by providing a sacrificial extra width of seal, or (best) by providing an impermeable thickened edge to the seal.
- Fretting of the edges of the seal and looseness and erosion of the shoulder can probably be minimised to a maximum of Degree 2 by limiting the salt content of the surfaced base course and the completed shoulder to a maximum < 6,7 min of 0,5 S/m and by utilising material also suitable for the wearing course of an unpaved road.
- The use of calcrete with a completed EC of more than 1,0 S/m as a combined base and shoulder, especially if it does not comply with a suitable specification for a calcrete wearing course may lead to severe edge fretting of Degree 4 or 5 in six years as well as to more severe shoulder looseness and erosion.
- All areas of the double seal remaining completely undamaged or having only barely discernible cracking of Degree 1 or 2 after 6 years of service were found to be completely impermeable to air at a water pressure head of 6 mm and to have a maximum permeability of 1,5 mℓ/cm^2/minute at a head of 20 mm of water.
- Exactly the same spots as tested above were also impermeable to water using the California grease ring method.
- A single surface treatment under such conditions is not advisable at an EC of more than about 0,4 S/m at sealing.
- The salt content tolerable by a good, impermeable double surface treatment on a coarsely graded (GM ≥ 2,0) calcrete base with a PI of SP-15, a LSM of 20-220 and MAASHO OMC of about 14-16% in an arid climate appears to be equivalent to an EC in the vicinity of 1,8-2,0 S/m after surfacing or, conservatively, 1,5 S/m after mixing and watering but just before compaction.

- The EC of both the tailings and the calcrete bases rose by an average of 0,27 S/m from just before compaction until sampling four days to two months after sealing (mostly within seven days of compaction).
- The reasons for this rise were presumably migration from the subbase during the few days of exposure before sealing and, in a few case only, the addition of extra seawater during recompaction, and migration from the subbase due to equilibration after sealing
- A detailed pavement evaluation in 1986 after about five years of service showed the average rut depth to be about 5 mm, ruts of more than 10 mm to be rare, with a general pavement condition rating of good except for one 5 km segment noted as "poor" due to a few localised shear failures on a micaceous subbase which were not caused by salt. On a best average basis the whole segment could have been rated as good.
- The road was structurally sound, with general pavement condition ratings of fair to good after 13 years in 1993 and good to very good after 28 years and qualified as a Category C road still in sound condition in 2008.
- With good maintenance the road easily exceeded its design life of 0,05 MESA in 15 years and had probably carried at least 0,1 MESA after 28 years in 2008.

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