

DEALING WITH TUNNEL AGEING

GR Basson • M Visagie • JG Malan

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Dealing with Tunnel Ageing

Report to the
Water Research Commission

by

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In Association with the University of Stellenbosch and
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EXECUTIVE SUMMARY

South Africa has several inter basin water transfer schemes, as indicated in Figure 1-1, with the most recent one the Lesotho Highlands Water project (LHWP) transfer tunnels bringing water under gravity from Lesotho through about 80 km of concrete lined tunnels. South Africa also has the largest irrigation tunnel in the world, the 5.33 m diameter Orange-Fish Tunnel, 81 km in length.



Figure 1-1 Water transfer schemes in South Africa

A concrete lined tunnel can lose as much as 30 percent of its hydraulic capacity over a period of 30 years. Engineers have a sound knowledge of the hydraulic roughness of new tunnels, as has been confirmed during the recent (1998) commissioning tests of the LHWP Transfer and Delivery Tunnels. Furthermore, recently completed research funded by the Water Research Commission (Pegram and Pennington, 1998) helps to assess the hydraulic capacity of new tunnels. There is however a need for a better understanding of the tunnel ageing process in order to plan for the phasing in of future possible new tunnels as water demands increase.

South Africa will have to rely more and more in future on water transfer schemes. These tunnels are extremely expensive, with estimated capital costs of about R30 000 /m length.

The main aim of the project was to provide a reliable hydraulically based methodology with which concrete lined tunnel ageing and the associated decrease in hydraulic capacity can be predicted. Both the ageing mechanisms of sliming (with sediment deposition) and soft water corrosive conditions were investigated. This will ensure the timeous phasing in of new schemes when tunnel ageing decreases the hydraulic capacity.

Furthermore, the aim was to establish whether remedial measures can be implemented to reverse or contain the tunnel ageing process.

Tunnel design and operation guidelines were also developed to limit tunnel ageing.

This study combined laboratory and field test data to obtain a better understanding of the ageing process in large diameter conduits such as water transfer tunnels. Field tests were carried out since 1998 at the Orange-Fish Tunnel, Roode Elsberg Tunnel and Theewaterskloof Tunnel, while laboratory investigations at the Universities of Stellenbosch and of Pretoria were also carried out on pipes. The following are key findings of this research project:

- Hydraulic ageing is caused by sediment deposition consisting of fine cohesive sediment, in some cases also containing some organic material. Manganese deposition by bacterial action also occur which is difficult to flush out at high discharge. In the Theewaterskloof Tunnel, sand at the tunnel invert also deposited due to diversion of Berg River water into the tunnel.
- Soft water corrosion occurred at the Western Cape and LHWP tunnels. The effect is a softening of the upper 3 mm of concrete, but this was protected by a thin layer of sediment at all the tunnels investigated. The soft water corrosion can however lead to deep scour holes and etch marks in the concrete, where a poor quality concrete was used during construction (Theewaterskloof Tunnel). This is however not expected in the high strength LHWP concrete lining.
- A relationship was developed and calibrated to determine the effective hydraulic roughness of a segmentally lined tunnel, including the joint step losses.
- A hydraulic rate of ageing equation was developed and calibrated on UK and South African data, for conditions where sediment deposition and sliming plays a role. The pH of the water is a dominant variable.
- An ultimate hydraulic roughness equation was developed and calibrated. A probable and maximum probable ultimate roughness formulation have been proposed.
- Hydraulic flushing at high flows are effective in reducing the hydraulic roughness by flushing out sediment deposits from the tunnel. Where bacterial action however plays a role, one cannot rely on flushing alone and other measures have to be considered.
- Chlorination has been used at a limited scale elsewhere in the world to reduce roughness, but was found more effective after cleaning of the tunnel. Depending on the system configuration, there are environmental problems associated with chlorination.
- Mechanical cleaning of a tunnel in soft water conditions is not recommended since it damages the tunnel and increase the roughness, no matter how careful you are. In hard water conditions, high pressure water jets damaged the tunnel surface. Literature also indicates rapid increase in roughness after cleaning, within one to two years.
- Tunnel linings such as poly-urethane can be considered in conditions of soft water corrosion, but should preferably be applied during the tunnel construction. These linings are however expensive and their lifespan is unknown. A good quality concrete without a lining, designed and operated correctly, should be sufficient for most conditions.
- Literature reviewed indicates that there are two main factors influencing the deterioration of the concrete due to soft water attack, namely the aggressiveness of the water and the composition of the concrete under attack.

No cement extenders were used in the concrete of the Theewaterskloof Tunnel and it was found that the leaching rate of Ca-ions increase exponentially with an increase in the water operating velocity in the pipe system. The main chemical process taking place in the concrete during soft water attack in the Theewaterskloof Tunnel is the leaching of Ca(OH)_2 . A maximum and minimum theoretical concrete deterioration depth rate was determined for the concrete. These rates also increase exponentially with an increase with water operating velocity in the pipe system. The permeability of this concrete is very high and therefore the leaching rates are also high.

The concrete mixture of the LHWP linings of the delivery tunnels contains fly-ash as a cement extender. Two different Ca-ion leaching rates for a specific water operating velocity were found for the LHWP concrete. It indicates that as a result of the fly-ash content in the mixture, decalcification of the C-S-H gel takes place at a low rate and Ca(OH)_2 leaching at a higher rate. The theoretical concrete deterioration depth rates for the LHWP concrete are much lower than the rates for the Theewaterskloof Tunnel. There are not enough data available at this stage, but it seems that the concrete deterioration rate increases with an increase of water operating velocity.

Although the soft water attack cannot be stopped with a specific concrete mix design it is possible to decrease it significantly with an optimum design. The optimum concrete mixture as designed by Castro & McIntosh (1994) for the LHWP tunnels will still be the best mixture for resistance to soft water attack because the Ca(OH)_2 content is very low in this design mix.

- Design and operational guidelines for the determination of tunnel ageing and selection of tunnel diameter during design, have been proposed. The LHWP Delivery Tunnel North (DTN) has been used as a case study.
- Prediction of future ageing of the LHWP DTN indicates that under current operating conditions, the rate of ageing would be relatively small and the ultimate discharge capacity would be between 80% to 85% of the new tunnel capacity. Previous studies used a higher ultimate roughness (DWA, 1999) of $k_s = 4 \text{ mm}$ which resulted in an aged discharge capacity of about 73% of the original capacity. The current predictions therefore indicate that it might be possible to implement the next phase of the LHWP without doubling the tunnels: Low Mashai Dam, that would require a total tunnel discharge capacity (including phase I flow) of $35,8 \text{ m}^3/\text{s}$ without downtime, or $39,8 \text{ m}^3/\text{s}$ with 10% downtime.

It is recommended that the design guidelines (Chapter 10) are followed during the design of a tunnel. Even more important however, is that the tunnel should be operated according to the design.

- Regular hydraulic tests at say 5 year intervals should be carried out at the LHWP, Theewaterskloof, Orange-Fish and Roode Elsberg Tunnels to determine the hydraulic roughness to extend the current data base.
- The theory developed for this study should be calibrated against field data of pipelines for application in pipe design.

Attention to the following chemical aspects regarding soft water corrosion is necessary:

- More laboratory tests are necessary on the LHWP test section at higher water operating velocities to determine the relationship between the Ca-ions leaching rates and water operating velocities.

- Tests on the microstructure of the concrete of the test sections are necessary to calibrate the rate of the theoretical concrete deterioration depths and to determine the exact influence of the permeability and porosity on soft water attack.
- Tests on the Theewaterskloof Tunnel core have to be completed to calibrate laboratory tests.
- A core from LHWP is required to calibrate laboratory tests.
- The aggressiveness of the water actually flowing through the tunnels should be determined. Laboratory tests with the water from the field will be necessary to confirm the calibration of the accelerated rate of deterioration in the laboratory with the actual field behaviour.
- Further tests on concrete mixtures with silica-fume as cement extender will be valuable to determine a concrete mixture with a higher resistance to soft water attack.
- The effect of soft water in the concrete mix should be studied
- Coatings should be applied in the Delivery Tunnel North to evaluate its long term performance
- Tunnels should not be cleaned by mechanical equipment or water jets, as this will increase the hydraulic roughness, especially in softwater conditions. Controlled hydraulic flushing at high discharge is recommended instead.

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Finally, the Steering Committee members need to be commended for their role in steering this project to its successful completion from 1999 to 2002. The project team would also like to thank Mr David van der Merwe, chair person for the WRC on this project, for the way in which he managed this final project prior to his retirement, and also for the valuable contribution that he made on hydraulic and water resources projects in South Africa.

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Table of Contents

LIST OF FIGURES	3
LIST OF TABLES	7
LIST OF SYMBOLS	9
KEYWORDS	10
1 INTRODUCTION	11
2 OBJECTIVES	13
3 METHODOLOGY	14
4 TYPES OF HYDRAULIC AGEING AND CONCRETE DETERIORATION	16
4.1 Introduction	16
4.2 Physical characteristics of aged tunnels	16
4.3 Concrete deterioration	17
5 HYDRAULIC ROUGHNESS	20
5.1 Boundary layer theory	20
5.2 Friction loss equations	23
5.3 Hydraulic roughness of new conduits	25
5.4 Critical conditions for deposition of sediment	32
5.5 Critical condition for re-entrainment of non-cohesive and cohesive sediment	33
5.6 Sediment transport in closed conduits	36
5.7 The rate of ageing	38
5.8 Ultimate roughness in tunnels / large conduits	50
6 SOFTWATER AGGRESSIVENESS INDEX	60
6.1 Ageing by soft water (chemical) corrosion	60
7 SOFT WATER CORROSION LABORATORY TESTS	63
7.1 Introduction	63
7.2 Background	63
7.3 Laboratory tests	68
7.4 Conclusions on chemical aspects	93
7.5 Recommendations	94
8 REMEDIAL MEASURES	95
8.1 Hydraulic flushing	95
8.2 Mechanical cleaning	127
8.3 Chlorination	135

8.4	Concrete design	136
8.5	Tunnel linings & other materials	137
8.6	Other measures	140
8.7	Tunnel costs	140
9	LHWP TUNNEL RATE OF AGEING AND ULTIMATE ROUGHNESS PREDICTION	143
9.1	Introduction	143
9.2	Ageing prediction (DWAF, 1999)	145
9.3	Transfer Tunnel	147
9.4	Delivery Tunnel	148
9.5	Proposed operation to minimize rate of ageing	149
9.6	LHWP Tunnel roughness prediction based on findings of this WRC study	154
10	TUNNEL DESIGN AND OPERATIONAL GUIDELINES INCORPORATING TUNNEL AGEING	160
11	CONCLUSIONS	162
12	RECOMMENDATIONS	164
13	REFERENCES	165
	APPENDICES	167
	APPENDIX A	167
	SOFTWATER CORROSION LABORATORY DATA	167
	APPENDIX B	181
	BIOLOGICAL ANALYSIS ON LHWP DTN SEDIMENT	181

LIST OF FIGURES

- Figure 1-1 Water transfer schemes in South Africa
- Figure 5.1-1 Variation of λ with Re for artificially roughened pipes
- Figure 5.1-2 Variation in thickness of laminar sub-layer with Reynolds number
- Figure 5.1-3 Laminar sub-layer on smooth boundary
- Figure 5.1-4 The transition zone for artificially roughened pipes
- Figure 5.1-5 Laminar sub-layer formation at Orange-Fish Tunnel
- Figure 5.3.3-1 Segmentally lined Delivery Tunnel North
- Figure 5.3.3-2 Roughness height as a function of step height to step length
- Figure 5.3.3-3 Segmentally lined tunnels new roughness prediction
- Figure 5.5.1-1 Modified Liu diagram (Rooseboom, 1992)
- Figure 5.5.1-2 Re-entrainment of sediment with laminar and turbulent boundary conditions
- Figure 5.5.2-1 Critical shear stress for mass erosion of reservoir sediment in open channel flow
- Figure 5.6-1 Head loss as given by Durand-Condolios relation for sand
- Figure 5.6-2 F_L values (Durand, 1956).
- Figure 5.7.1-1 Observed linear roughness increase with time of Thirlmere pipeline
- Figure 5.7.3-1 Thirlmere pipeline rate of ageing (Colebrook, 1938)
- Figure 5.7.3-2 USA pipeline rate of ageing (Colebrook, 1938)
- Figure 5.7.3-3 UK and USA pipeline rate of ageing (Colebrook, 1938)
- Figure 5.7.3-4 Relationship between pH and hydraulic ageing for pipes (Colebrook, 1938)
- Figure 5.7.3-5 Relationship between pH and hydraulic ageing for tunnels (Colebrook, 1958)
- Figure 5.7.3-6 Relationship between input stream power, pH and rate of ageing
- Figure 5.7.3-7 Observed versus predicted rate of ageing-hydraulic radius ratio values
- Figure 5.7.3-8 Predicted and observed discharges based on ageing equation 5.7.3-6
- Figure 5.7.3-9 Discharge capacity of new tunnel at $k_o = 0.03$ mm
- Figure 5.7.3-10 Typical 30 year aged flow capacity based on rate of ageing equation and pH = 7.5.
- Figure 5.7.3-11 Typical 30 year aged flow capacity based on rate of ageing equation and pH = 5.5
- Figure 5.8.5-1 Relationship between k_s/R and shear velocity
- Figure 5.8.5-2 Relationship between k_s/R and energy slope
- Figure 5.8.5-3 Relationship between k_s/R and pH
- Figure 5.8.5-4 Ultimate roughness tunnel discharge prediction
- Figure 5.8.6-1 Location plan of Roode Elsberg Tunnel
- Figure 5.8.6-2 Sediment deposition in Roode Elsberg Tunnel

- Figure 6.1-1 Soft water zones in ground water in South Africa
- Figure 7.3.2-1 Pipe system at the hydraulic laboratory of the University of Pretoria
- Figure 7.3.2-2 The experimental set up in the laboratory
- Figure 7.3.2-3 90 ° V-notch at the end of pipe system
- Figure 7.3.3-1 Surface of sample after 10 days leaching of Ca (X20)
- Figure 7.3.3-2 Inside of sample after 10 days leaching of Ca (X20)
- Figure 7.3.3-3 Surface of sample after 10 days leaching of Ca (X5000)
- Figure 7.3.3-4 Inside of sample after 10 days leaching of Ca (X5000)
- Figure 7.3.3-5 Ca-ions leached from concrete (pilot test section)
- Figure 7.3.4-1 Cumulative leaching of Ca-ions at different operating velocities (Theewaterskloof test section)
- Figure 7.3.4-2 Leaching rate of Ca-ions from the concrete for different velocities (Theewaterskloof test section)
- Figure 7.3.4-3 Leaching rate of Ca-ions per total volume of water through TWK test section (Theewaterskloof test section)
- Figure 7.3.4-4 Leaching rate of Ca-ions per water volume for different water velocities (Theewaterskloof test section)
- Figure 7.3.4-5 Theoretical deterioration depths of concrete for different water operating velocities (Theewaterskloof test section)
- Figure 7.3.5-1 Cumulative leaching of Ca-ions per day (LHWP test section)
- Figure 7.3.6-1 Laboratory hydraulic roughness changes due to softwater at $v = 1.2$ m/s: Pilot test section
- Figure 7.3.6-2 Laboratory hydraulic roughness changes due to softwater at $v = 1.2$ m/s and following 3 m/s flushing: Pilot test section
- Figure 7.3.6-3 Laboratory hydraulic roughness changes due to softwater: Theewaterskloof Tunnel test section
- Figure 7.3.6-4 Laboratory hydraulic roughness changes due to softwater: LHWP DTN test section
- Figure 8.1.2-1 Longitudinal profile of Orange-Fish Tunnel
- Figure 8.1.2-2 Orange-Fish Tunnel valve
- Figure 8.1.2-3 Orange-Fish Tunnel outlet
- Figure 8.1.2-4 Orange-Fish Tunnel flow measurement gauge
- Figure 8.1.2-5 Observed maximum annual Orange-Fish Tunnel flows
- Figure 8.1.2-6 Localized clay deposition in Orange-Fish Tunnel
- Figure 8.1.2-7 Typical hard smooth concrete surface of Orange-Fish Tunnel
- Figure 8.1.2-8 Depth gauge at tunnel shaft operated by DWAF personnel
- Figure 8.1.2-9 Depth recording at Orange-Fish Tunnel Shaft
- Figure 8.1.2-10 Ott Water level recorder and Thalimedes data logger
- Figure 8.1.2-11 Historically observed hydraulic roughness of Orange-Fish Tunnel
- Figure 8.1.2-12 Orange-Fish discharge capacity change over time
- Figure 8.1.2-13 Orange-Fish Tunnel shaft 0 to 6 roughness (2001)

- Figure 8.1.2-14 Orange-Fish roughness shaft 6 to 7a (2001)
- Figure 8.1.2-15 Orange-Fish Tunnel sediment transport 2001
- Figure 8.1.2-16 Orange-Fish Tunnel cleaned section roughness 2001
- Figure 8.1.3-1 Theewaterskloof Tunnel layout
- Figure 8.1.3-2 Longitudinal profile of the Theewaterskloof to Kleinplaas Dam tunnel
- Figure 8.1.3-3 Lowering of the boat at the Theewaterskloof Dam intake shaft
- Figure 8.1.3-4 Bacteriological growth between the intake and Berg River, Theewaterskloof Tunnel
- Figure 8.1.3-5 Deep scour hole in soft water corroded concrete, intake to Berg River, Theewaterskloof Tunnel
- Figure 8.1.3-6 Deep scour in soft water corroded poor quality concrete, intake to Berg River, Theewaterskloof Tunnel
- Figure 8.1.3-7 Exposed coarse aggregate after sediment removal due to emptying, Banghoek to Jonkershoek, Theewaterskloof Tunnel
- Figure 8.1.3-8 Sand deposition, Wolwekloof to Banghoek, Theewaterskloof Tunnel
- Figure 8.1.3-9 Typical soft water corrosion underneath sediment deposition, Theewaterskloof Tunnel
- Figure 8.1.3-10 Tunnel flows and sediment concentrations during 1999 tests
- Figure 8.1.3-11 Test flows used during 2001 Theewaterskloof Tunnel tests
- Figure 8.1.3-12 Banghoek to Jonkershoek roughness 2001
- Figure 8.1.3-13 Wolwekloof to Banghoek roughness 2001
- Figure 8.1.3-14 Berg River west to Wolwekloof roughness 2001
- Figure 8.1.3-15 Theewaterskloof Dam to Berg East portal roughness 2001
- Figure 8.1.3-16 Sediment concentrations flushed out at Jonkershoek
- Figure 8.1.4-1 Roode Elsberg Dam
- Figure 8.1.4-2 Roode Elsberg Tunnel outfall
- Figure 8.1.4-3 Roode Elsberg Tunnel inspection team
- Figure 8.1.4-4 Roode Elsberg Tunnel at intake
- Figure 8.1.4-5 Lowering of the emergency gate at Roode Elsberg Tunnel
- Figure 8.1.4-6 Sediment deposits in Roode Elsberg Tunnel (1999)
- Figure 8.1.4-7 Construction equipment left in Roode Elsberg Tunnel
- Figure 8.1.4-8 Bypass pipe at Roode Elsberg Tunnel
- Figure 8.1.4-9 Sediment transport through Roode Elsberg Tunnel (1999)
- Figure 8.1.5-1 Dasbos pipeline before tests started
- Figure 8.1.5-2 Dasbos pipe flushing hydraulic roughness test results
- Figure 8.1.5-3 Dasbos pipe after testing and cleaning
- Figure 8.2.1-1 Tunnel cleaning machine before lowered down shaft into tunnel
- Figure 8.2.1-2 Rear view of tunnel cleaner
- Figure 8.2.1-3 Side view of cleaner with steel brushes

- Figure 8.2.1-4 Tunnel cleaning machine inside Colorado MWD tunnel
- Figure 8.2.4-1 Tarraleah pipes ageing after cleaning
- Figure 8.2.5-1 Water jet cleaning of Orange-Fish Tunnel (1999)
- Figure 8.2.5-2 Cleaning team on truck in Orange-Fish Tunnel
- Figure 8.2.5-3 Field measurement at shaft 7a during 2001
- Figure 8.2.6-1 Theewaterskloof Tunnel cleaning during 1999
- Figure 8.2.6-2 Theewaterskloof Tunnel with locally cleaned section next to sediment deposits
- Figure 8.2.6-3 Theewaterskloof Tunnel after cleaning
- Figure 8.2.7-1 Tunnel cleaning at Roode Elsberg Tunnel
- Figure 8.3.2-1 Hydraulic roughness change with chlorination at Tarraleah siphons (Brett, 1980)
- Figure 8.7-1 Unit costs of recently completed tunnels
- Figure 9.1-1 Layout of LHWP tunnels
- Figure 9.1-2 Layout of the LHWP Transfer Tunnel
- Figure 9.1-3 Layout of the LHWP Delivery Tunnel
- Figure 9.1-4 Intake tower of the LHWP Transfer Tunnel
- Figure 9.1-5 Delivery Tunnel North at Ash River
- Figure 9.3-1 Simulated hydraulic capacity loss of the Transfer Tunnel due to ageing (DWAF, 1999).
- Figure 9.4-1 Simulated hydraulic capacity loss of the Delivery Tunnel due to ageing (DWAF, 1999)
- Figure 9.5.3-1 Delivery Tunnel capacity with raised Muela Dam
- Figure 9.6-1 Segmentally lined Delivery Tunnel North
- Figure 9.6-2 Sediment deposition in the DTN (1999)
- Figure 9.6-3 Sediment deposition partially covering the joints in the DTN (1999)
- Figure 9.6-4 Soft water corrosion in the DTN (1999)
- Figure 9.6-5 Predicted ageing of the Delivery Tunnel North (this study)

LIST OF TABLES

Table 5.3.1-1	Recommended new pipe roughness values k_s (mm) (HR, 1990)
Table 5.3.2-1	Observed roughness of new (or near new) cast in-situ concrete tunnels
Table 5.3.3-1	Segmentally lined tunnel data
Table 5.3.3-3	Calibration of segmentally lined tunnel relative roughness (R/k_s)
Table 5.3.3-3	Predicted and observed segmentally lined roughness
Table 5.5.2-1	Tunnel shear stress and stream power
Table 5.7.2-1	Roughness values for different sewer pipe material
Table 5.7.3-1	Rate of ageing calibration data
Table 5.7.3-2	Predicted and observed roughness and flow
Table 5.8.2-1	Recommended aged pipeline hydraulic roughness k_s (mm) (HR, 1990)
Table 5.8.4-1	Maximum ultimate roughness prediction by Colebrook (1958)
Table 5.8.5-1	Ultimate tunnel roughness prediction with new equations
Table 5.8.7-1	Predicted discharge based on probable and maximum probable ultimate roughness
Table 6.1-1	Aggressiveness index interpretation (DWAF, 1996)
Table 6.1-2	Recommended limits for assessing aggressiveness (Basson, 1989)
Table 6.1-3	Aggressiveness index interpretation (Basson, 1990)
Table 7.2.1-1	The main compounds in Portland cement: (Neville, 1987)
Table 7.2.2-1	Swedish classification of natural waters containing carbon dioxide (Lea, 1989)
Table 7.3.3-1	Leaching of Ca-ions from pilot test section
Table 7.3.4-1	Chemical composition of cement used for Theewaterskloof tunnel section
Table 7.3.4-2	Composition and properties of concrete mixture for Theewaterskloof tunnel section
Table 7.3.4-3	Theoretical concrete deterioration depths (Theewaterskloof tunnel section)
Table 7.3.5-1	Chemical composition of cement used for LHWP tunnel section
Table 7.3.5-2	Chemical composition of Lethabo fly-ash
Table 7.3.5-3	Concrete mixture for precast linings of the LHWP Delivery Tunnels from the 1994 records as obtained from DWAF
Table 7.3.5-4	Theoretical depth of deterioration of the concrete for LHWP tunnel section
Table 7.3.7-1	Mix proportion for LHWP tunnel linings as proposed by Castro & McIntosh ((1994)
Table 8.1.4-1	Roode Elsberg Tunnel field test data
Table 8.5.4-1	Coatings for concretes in aggressive water (Basson, 1989)

Table 8.7-1	Historical tunnel costs
Table 9.2-1	Design tunnel roughness for aged condition
Table 9.3-1	Hydraulic capacity of the Transfer Tunnel with ageing (m^3/s) (DWAF, 1999)
Table 9.4-1	Hydraulic capacity of the Delivery Tunnel with ageing (m^3/s) (DWAF, 1999)
Table 9.5.2-1	Transfer Tunnel ageing and related MOL(m) required at Katse Reservoir for various possible yields (Surge shaft MOL = 1950 m) (DWAF, 1999)
Table 9.5.2-2	Transfer Tunnel ageing and related MOL(m) required at Katse Reservoir for various possible yields, with doubled surge shaft (Surge shaft MOL = 1919 m)
Table 9.5.2-3	Hydraulic capacity of the Transfer Tunnel with ageing with reduced surge shaft MOL (m^3/s) (DWAF, 1999)
Table 9.5.3-1	Hydraulic capacity of a doubled Delivery Tunnel

LIST OF SYMBOLS

A	Al_2O_3
AOL	Average operating level
C	CaO
C	Volumetric sediment concentration
C	Chezy roughness coefficient
C_3A	Tricalcium aluminate
C_2S	Dicalcium silicate
C_3S	Tricalcium silicate
C_4AF	Tetracalcium Aluminoferrite
CH	$\text{Ca}(\text{OH})_2$ calcium hydroxide
d	Particle diameter
dv/dy	Velocity distribution
D	Datchet Tunnel
D	Open channel flow depth
D	Pipe diameter
D	Tunnel diameter
DTN	Delivery Tunnel North
DWAF	Department of Water Affairs and Forestry
E	Enfield
EO	Elyouse Tunnel
F	Fe_2O_3
FSL	Full supply level
g	Gravitational acceleration
h	energy head
h/L	Energy slope
H	H_2O
k_o	New tunnel hydraulic roughness
K_{op}	Coefficient of oxygen permeability
k_s	Effective hydraulic roughness
l	Segment length
L	Liquid
L	Pipe length
LHWP	Lesotho Highlands Water Project
M	Mixture
MOL	Minimum operating level
n	Manning hydraulic roughness value
pH	Water quality variable

Q	Discharge
R	Hydraulic radius
Re	Reynolds number
Rv	Roding Valley Tunnel
s	Hydraulic gradient
s	Mean step height between joints
ss	Ratio of density of solids to that of fluid
S	SiO ₂
S _t	Hydraulic gradient
T	Years after commissioning
TBM	Tunnel boring machine
V	mean flow velocity
V _c	Critical velocity
w	Particle settling velocity
W	Walsall Tunnel
WRC	Water Research Commission
x	Calibrated coefficient
y	depth measured from the bottom
yr	year
κ	Von Karman coefficient = 0.4 for clear water
ϕ _D	Dimensionless sediment parameter
ρ _s	Sediment density
τ ₀	Bed shear stress
v	Flow velocity
ν	kinematic viscosity of fluid
v*	Shear velocity
λ	friction coefficient
α	rate of ageing
δ ¹	Laminar layer thickness
μ	Dynamic viscosity
τ	Shear stress

KEYWORDS

Tunnel ageing; Water Transfer Tunnel; concrete corrosion; hydraulic roughness; friction loss; mechanical cleaning; rate of ageing; ultimate ageing; hydraulic flushing; soft water corrosion.

1 INTRODUCTION

South Africa has several inter basin water transfer schemes, as indicated in Figure 1-1, with the most recent one the Lesotho Highlands Water project transfer tunnels bringing water under gravity from Lesotho through about 80 km of concrete lined tunnels.

A concrete lined tunnel can lose as much as 30 percent of its hydraulic capacity over a period of 30 years. This ageing process is mostly due to sediment deposition and sliming, but soft water corrosion can also play a role. Engineers have a sound knowledge of the hydraulic roughness of new tunnels, as has been confirmed during the commissioning tests of the LHWP Transfer and Delivery tunnels. Furthermore, recently completed research by the Water Research Commission (Pegram and Pennington, 1998) helps to assess the hydraulic capacity of new tunnels. There is however a need for a better understanding of the tunnel ageing process in order to plan for phasing in of future possible new tunnels as water demands increase.

Tunnels are extremely expensive infrastructure, with capital costs in the order of about R30 000 /m length. South Africa will have to rely more and more in future on water transfer schemes and this study will provide very important information on long-term tunnel capacities and possible management of these tunnels to limit the ageing rate.

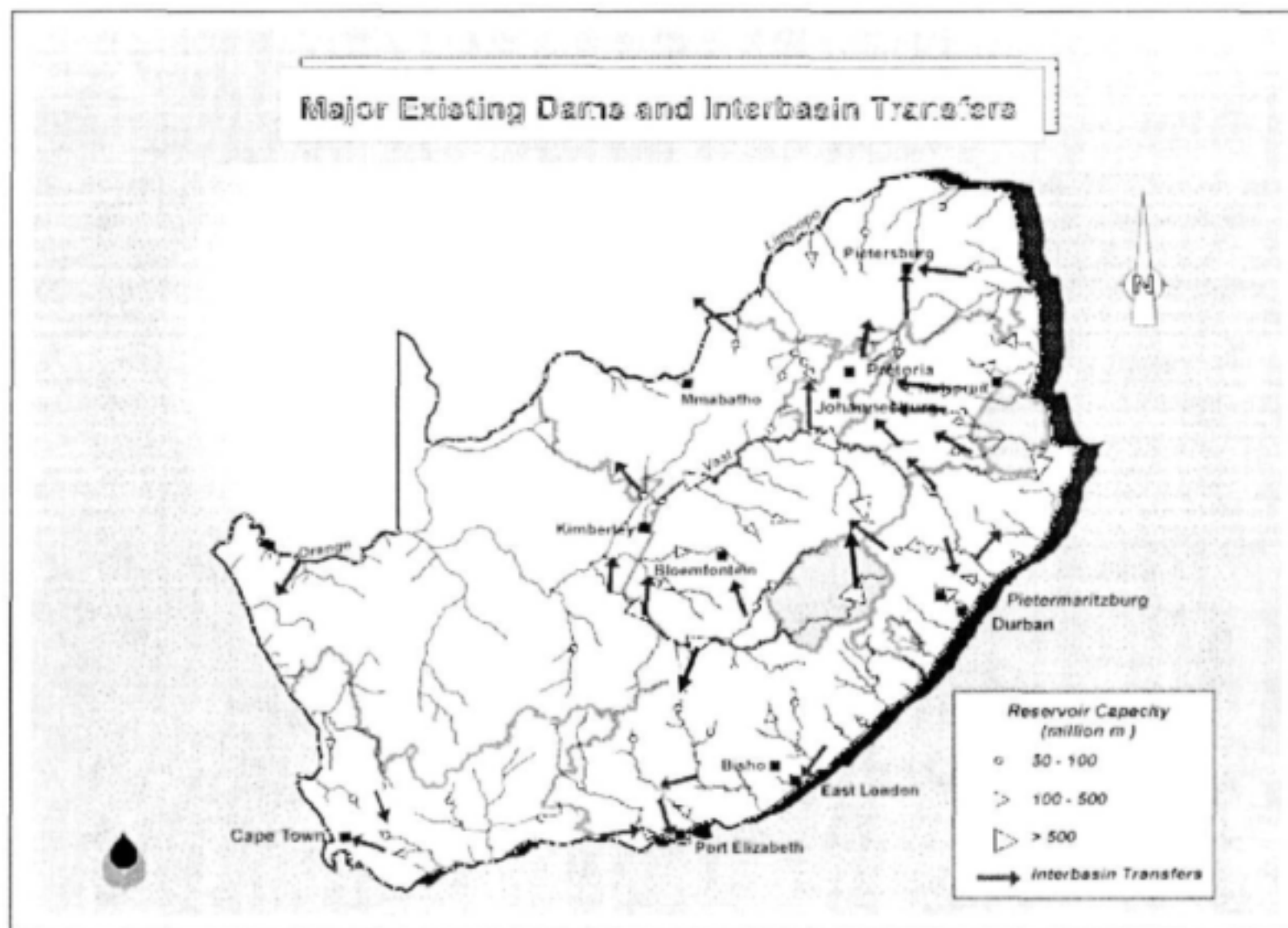


Figure 1-1 Water transfer schemes in South Africa

2 OBJECTIVES

The main aim of this project was to provide a reliable hydraulically based methodology with which concrete lined tunnel ageing and the associated decrease in hydraulic capacity can be predicted. The ageing mechanisms of sliming and soft water corrosive conditions were investigated. This will ensure the timeous phasing in of new schemes when tunnel ageing decreases the hydraulic capacity.

Furthermore, the aim was to establish whether remedial measures can be implemented to reverse or contain the tunnel ageing process. Two methods were considered and tested in the field: hydraulic flushing and mechanical cleaning.

Guidelines were also developed according to which hydraulic flushing of water transfer tunnels can be used with safety as management tool under conditions of soft water concrete corrosion, in order to limit the rate of ageing and ultimate ageing due to sediment deposition, based on theory calibrated on laboratory data and verified against field data.

3 METHODOLOGY

The study involved a combination of theoretical hydraulic assessments of the tunnel ageing process under conditions of sliming or corrosion, as well as laboratory and fieldwork. The following key aspects were identified at the start of the study and some were investigated in greater detail than others as the study progressed:

- **Literature review** of tunnel ageing rates, the processes and variables involved and remedial measures. International information/data on tunnel roughness and rates of ageing for conventional concrete linings and segmentally lined tunnels were obtained. In this regard the Metropolitan Water District of Southern California (MWD) contributed data on the Colorado aqueducts and tunnels.
- **Theoretical analysis** which investigated the interrelationship between the ageing rate and stream power, pH, concrete characteristics, etc., also depending on available data.
- **Laboratory tests** on sediment transport in pipes and the theoretical analysis of the interrelationships between roughness, diameter, sediment concentration, energy dissipation, deposition and critical conditions for re-entrainment. This theory is essential when predictions on the effectiveness of tunnel flushing are made. The theory could be applied to minimize the rate of ageing for specific boundary conditions.
- **Field investigation** of the Orange-Fish Tunnel and LHWP Tunnel, and Theewaterskloof Tunnel and Roode Elsberg Tunnel were carried out. Hydraulic grade lines for different discharges were measured and the tunnel roughness was calculated to obtain long-term ageing information necessary for the theoretical analysis of South African data. A field study at the Orange-Fish Tunnel was carried out in 1998 and indicated improvement of the hydraulic capacity by hydraulic flushing. This test was repeated in 1999, to check whether this was a long-term improvement. Physical inspection of the tunnels was also carried out.

While conditions (hard water) at the Orange-Fish Tunnel are such that slime would form with sediment deposition, it was also considered necessary to study soft water corrosion of concrete at other tunnels. The Theewaterskloof Tunnel, Roode Elsberg and LHWP Tunnel North (DTN) were therefore selected as case studies to provide long-term information in this regard.

- **Hydraulic investigation of remedial measures:**

Hydraulic flushing and the effects on decreasing tunnel roughness and increasing the discharge capacity was carried out. Field tests were carried out at the Orange-Fish, Roode Elsberg and Theewaterskloof Tunnels.

Mechanical cleaning of the tunnel. Investigation of roughness reduction and increased discharge capacity, based on literature and the cleaning of a part of the Orange-Fish, Roode Elsberg and Theewaterskloof Tunnels with measurement of the hydraulic gradient line and roughness reduction were carried out. The USA experience was also incorporated in the study.

Other: Chlorination, UV sterilization and relining alternatives were investigated.

- **Chemical soft water corrosion tests** involved laboratory work, investigating the different concrete mix designs, flow velocities, rate of soft water corrosion (leaching of cement) and measurement of the change over time of hydraulic roughness. Three 5 m test concrete pipe sections were carried out by using distilled water,

running first at low flow velocities. Near the end of the tests, high flushing velocities were used to investigate possible erosion of the soft concrete surface. If hydraulic flushing is used as a management tool, it would be essential to have this knowledge to ensure the flushing discharge is small enough without risk of damaging the concrete surface with associated increased friction losses.

4 TYPES OF HYDRAULIC AGEING AND CONCRETE DETERIORATION

4.1 Introduction

Increased frictional energy losses in tunnels are often caused by sediment deposition. The sediment is typically fine clay with silt fractions, with some organic material, and forms a thin 1-3 mm (LHWP Deliver Tunnel) to a 7 to 10 mm layer (Roode Elsberg Tunnel), covering the whole tunnel. When rivers are diverted directly into the tunnel, sand deposition occurs at the invert of the tunnel (Berg-Jonkershoek Tunnel). Sometimes the sediment has a slimy character which is associated with the water pH. Bed forms develop in the deposited sediment which leads to increased hydraulic roughness and reduced discharge capacity.

Sediment deposition can also be beneficial in lowering secondary losses caused by joints in segmentally lined tunnels. In the LHWP Delivery Tunnel joint openings were smoothly covered by sediment, two years after commissioning.

Hydraulic ageing can also be caused by erosion of the concrete, leading to higher frictional losses. Concrete erosion in hydraulic structures are caused by cavitation, abrasion and/or chemical or biological attack. Of these, chemical attack by mineral free water or soft water corrosion is of most concern in tunnels, not that it affects the structural strength much, but rather because of a possible increase in frictional losses created by an eroded concrete surface. The soft water corrosion effect could however be masked by a thin layer of deposited fine sediment on the concrete surface, which limits concrete erosion.

If sediment deposition occurs with biological growth, the hydraulic roughness could become quite high (Theewaterskloof-Franschhoek Tunnel).

4.2 Physical characteristics of aged tunnels

4.2.1 Cause of ageing: Sediment deposition

(a) International experience

A good description of the characteristics of tunnel ageing as experienced at the MWD-USA Colorado aqueduct is presented by Horowitz and Lee, (1971). Deposition was found to consist of about 80 % CaCO_3 with the remainder being mostly organic matter. It was estimated that about one-third of the CaCO_3 precipitation in the aqueduct water is deposited as scale on the aqueduct walls.

"Visual inspection indicates the thickness and composition of the deposits to be remarkably uniform, suggesting existence of a state of equilibrium between the organic and inorganic components. Biological analysis of the scale indicates that both bacterial and higher life forms such as algae and protozoa live in the deposits." (Horowitz and Lee, 1971). A certain type of worm was also found in the deposits. The worms activity was found to increase the surface roughness of the scale by the worm's burrowing action, but on the other had it also made deposits spongy and therefore easy to remove by mechanical means.

"Age of the conduit surfaces alone does not appear to have a significant influence on carrying capacity of the aqueduct. Up to a certain point, even the accumulation of some deposits on the concrete surfaces will lower the coefficient of roughness, thus increasing flow capacity" (Horowitz and Lee, 1971). (This has also been experienced at the Hartebeespoort Dam siphon, where the initial estimated roughness of 12 mm was decreased over time to less than 5 mm, based on an investigation by Hopkins (1978).)

Hopkins stated: "However as the coating increases in thickness, becomes stringy and spongy, or both, the carrying capacity of the conduit gradually diminishes with time".

(b) Local experience

Regular inspection of the Orange-Fish Tunnel (hard water conditions) since 1974 revealed the following (Metcalf and Jordaan, 1990):

The lining was slippery and close inspection revealed a thin (<1 mm) coating of slime. More noticeable was the calcite deposits. "These calcite deposits, whilst soft enough to gouge, are not eroded by tunnel flow. Calcification is inhibited when the tunnel is full, exfiltration being replaced by infiltration". The impact of calcification on the total roughness was estimated to be less than 5% however (Metcalf and Jordaan, 1990).

Metcalf and Jordaan (1990) noted: "Silt/slime deposits presented when dry as fawn concave flakes, are estimated 0.5 mm thick. When wet the deposit is dark brown and continuous. A mottling phenomenon on the crown appears to be due to local dampness. When the surface of the dry lining is abraded, it typically yields a cream coloured powdery coating of calcitic silt. Under the coating, the surface of the concrete, at the occasional points sampled, exhibited a rough (sand paper) feel".

Most of the calcite deposits in the Orange-Fish Tunnel are attributed to the one year (1976) of no flows in the tunnel, after commissioning in 1974. During the first 10 years of operation the flows transferred were less than 10 m³/s, with associated flow velocities < 0.5 m/s, (relative to the design capacity of 47.6 m³/s. During later years' operation, the peak summer flows increased to about 25 m³/s ($v = 1.1$ m/s). At these low flow velocities, sediment deposition is expected with related relatively high annual rate of roughness increase, as observed.

Results from the 1998 tests at the Orange -Fish Tunnel indicate that a short period of high flow managed to flush the tunnel and that tunnel roughness was decreased in comparison to that of 1988. This was also experienced to some extent during the 1988 tests.

4.3 Concrete deterioration

4.3.1 Cavitation

Cavitation results from the collapse of vapor pressure bubbles formed by pressure changes within a high velocity water flow, but is generally not important in tunnels.

4.3.2 Abrasion erosion

Abrasion erosion of concrete is caused by water transported silt, sand and debris. Ordinarily, concrete in properly designed, constructed, used and maintained tunnels will undergo years of erosion-free service and due to the relatively low velocities and generally fine sediment in SA tunnels, abrasion erosion is not a problem in water transfer tunnels. Damage has however been reported at high head diversion tunnels at dams, where the tunnel invert is most heavily damaged (Graham, 1998).

4.3.3 Chemical erosion

a) Sources of chemical attack

The compounds present in hardened Portland cement are attacked by water and by many salts and acid solutions. Fortunately, in most hydraulic structures the deleterious action on a mass of concrete with a low permeability is so slow that it is unimportant. However, there are situations where chemical attack can become serious and

accelerate deterioration and erosion of the concrete, and possibly also increase the hydraulic roughness.

Acidic environments can result in deterioration of exposed concrete surfaces. The acidic environment may range from low acid concentrations found in mineral-free water to high acid concentrations found in mine water.

Hydrogen sulfide corrosion, a form of acid attack, is common in septic sanitary systems.

b) Erosion by mineral free water (soft water corrosion)

Hydrated lime is one of the compounds formed when cement and water combine. It is readily dissolved by water and more aggressively dissolved by pure mineral-free water, found in some mountain streams, such as in the Western Cape and Lesotho Highlands. Dissolved carbon dioxide is contained in some fresh waters to make the water slightly acidic and add to its aggressiveness.

Scandinavian countries have reported serious attacks by fresh water, both on exposed concrete surfaces and interior surfaces of conduits where porosity of cracks have provided access (Graham,1998). In the USA, there are many instances where the surface of the concrete has been etched by fresh water flowing over it, where in general USA raw water typically has a nearly neutral acid-alkaline balance (pH).

In South Africa the Theewaterskloof-Franschhoek Tunnel has in many places been damaged severely by soft water attack, leaving long etch marks and in some places deep holes in the concrete.

Decaying vegetation is the most frequent source of acidity in natural waters. Running water that has a pH as low as 6,5 will leach lime from concrete, reducing its strength and making it more porous on the surface. If the concrete surface remains intact, the hydraulic roughness will probably not change. However, high flow flushing could develop bed forms in the soft concrete surface or erode the sediment, or mechanical cleaning operation can damage the surface and expose coarse aggregate. During cleaning of part of the Roode Elsberg Tunnel as part of this study, cleaning by brooms by hand actually removed the coarse aggregate leaving 20 to 30 mm diameter holes in the surface!

The amount of lime leached from concrete is a function of the area exposed, the volume of concrete, as well as aggressiveness of the water. As the surface mortar is leached from the concrete, more coarse aggregate is exposed, which naturally decreases the amount of mortar exposed. With less mortar exposed, less leaching occurs, and hence major structural problems do not usually result, but the hydraulic roughness can increase.

4.3.4 Bacterial action

Some anaerobic bacteria are able to reduce the sulfates that are present in natural water and produce hydrogen sulfides as a waste product. Another group of bacteria takes the reduced sulfur and oxides back so that sulphur acid is formed. The genus *Thiobacillus* is the sulfur-oxidizing bacteria that is most destructive to concrete.

Sulfur-oxidizing bacteria are likely to be found wherever warmth, moisture, and reduced compounds of sulfur are present. Generally, a free water surface is required, in combination with low dissolved oxygen in sewage and low flow velocities that permit the buildup of scum on the walls of a tunnel or pipe, in which the anaerobic sulfur-reducing bacteria can thrive.

Newly constructed concrete has a strongly alkaline surface with a pH of about 12. No species of sulfur bacteria can live in such a strong alkaline environment. Natural carbonation of the free lime by carbon dioxide in the air slowly drops the pH of the concrete surface to 9 or less. At this level of alkalinity, the sulfur bacteria *Thiobacillus thioautotrophicus* generates thiosulfuric and polythionic acid. The pH of the surface moisture steadily declines, and at a pH of about 5, *Thiobacillus concretivorus* begins to proliferate and produce high concentrations of sulphuric acid, dropping the pH to a level of 2 or less.

The destructive mechanism in the corrosion of the concrete is the aggressive effect of the sulphate ions on the calcium aluminates in the cement paste (Graham, 1998). The main concrete corrosion problem in a sewer, therefore, is chemical attack by this sulphuric acid which accumulates along the crown of the sewer.

Sediment obtained from the LHWP DTN was tested during this study for possible bacterial action (see Appendix B).

5 HYDRAULIC ROUGHNESS

5.1 Boundary layer theory

Colebrook (1938, 1958) stated that while the boundary layer dominates in new pipes and tunnels, it is not the case under aged conditions. Field data showed clearly that aged pipes and tunnels operate in the transitional or rough turbulent zone.

In laminar flow, the friction is transmitted by pure shearing action. Consequently, the roughness of the solid surface has no effect, except to trap small 'pools' of stationary fluid in the interstices, and thus slightly increase the thickness of the stationary layer of fluid.

In a turbulent flow, a laminar sub-layer forms close to the solid surface. If the average height of the surface roughness is smaller than the height of the laminar layer, there will be little or no effect on the overall flow.

Turbulent flow is a process of momentum transfer from layer to layer. Consequently, if the surface roughness protrudes through the laminar region into the turbulent region, then it will cause additional eddy formation and therefore greater energy loss in the turbulent flow. This implies that the apparent frictional shear will be increased.

The $\lambda - Re$ curves for artificially roughened pipes are shown in Figure 5.1-1. At the lower range of Reynolds numbers the curves for the rough pipes merge into the single smooth pipe curve. Thus, rough pipes can behave like smooth ones at low Reynolds numbers. This phenomenon is explained by the presence of a sub-layer, formed adjacent to the boundary, in which the flow is laminar. The presence of such a layer was also confirmed by the velocity distributions obtained by Nikuradse. At low Reynolds numbers the sub-layer is sufficiently thick to cover the boundary roughness elements so that the turbulent boundary layer is, in effect, flowing over a smooth boundary, figure 5.1-2.

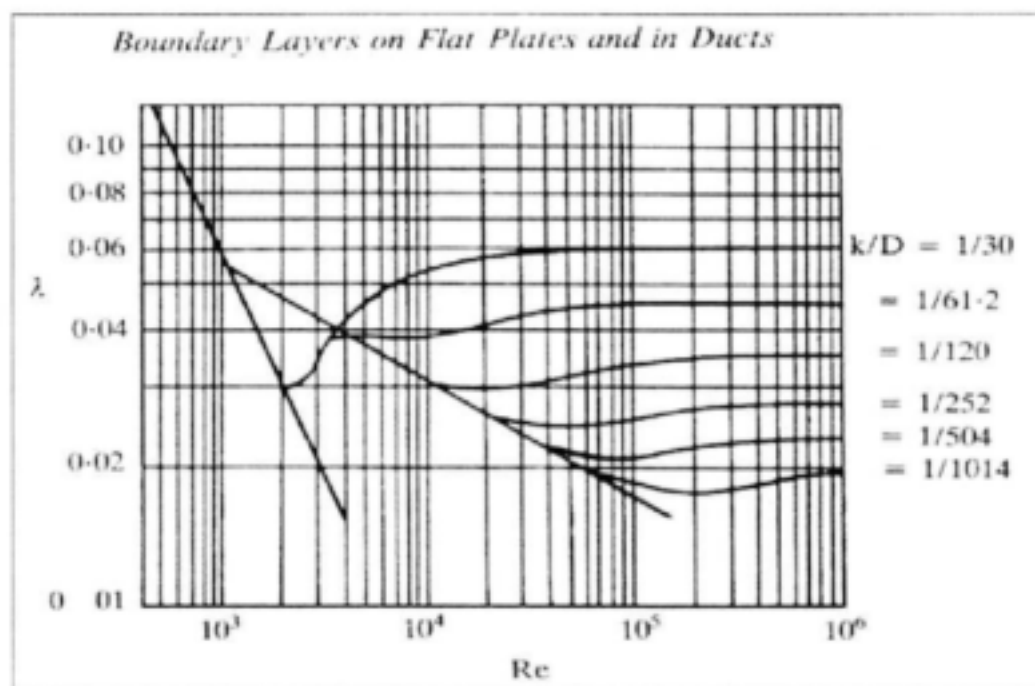


Figure 5.1-1 Variation of λ with Re for artificially roughened pipes

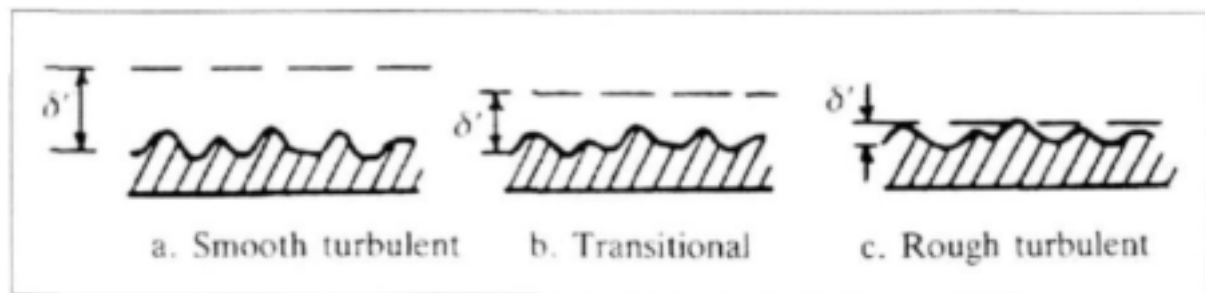


Figure 5.1-2 Variation in thickness of laminar sub-layer with Reynolds number

The sub-layer thickness decreases with increasing Reynolds number and at high Reynolds numbers the roughness elements are fully exposed to the turbulent boundary layer. At intermediate Reynolds numbers, in the transitional turbulent zone, the friction factor is influenced by both the relative roughness and Reynolds number.

Figure 5.1-3 shows a sub-layer formed on a smooth boundary beneath a turbulent boundary layer.

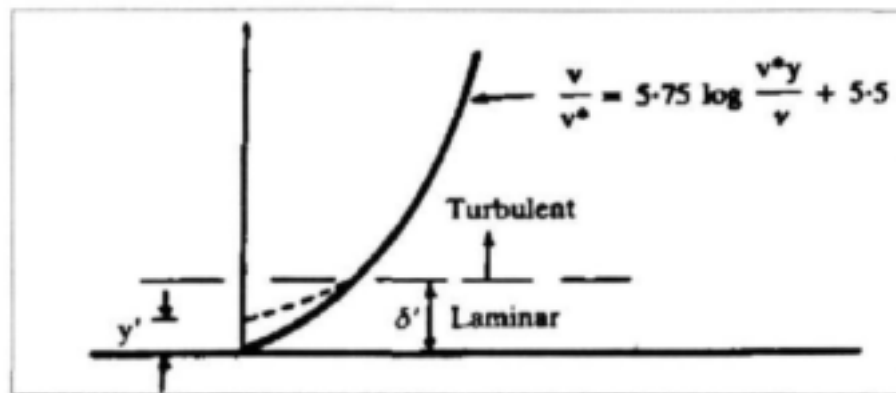


Figure 5.1-3 Laminar sub-layer on smooth boundary

In the laminar sub-layer,

$$\tau = \mu \frac{dv}{dy} \quad (5.1-1)$$

whence $\frac{dv}{dy} = \frac{\tau}{\rho v}$ and $v = \frac{\tau y}{\rho v}$

Where τ = shear stress, V = flow velocity, μ = viscosity, y = distance from wall / bed and ρ = fluid density. At the upper boundary the velocity of the upper turbulent flow is the same as in the laminar layer and if these are equated, the laminar layer thickness (δ') can be determined:

$$\delta' = \frac{11.6\nu}{v^*} \quad (5.1-2)$$

Where v^* = shear velocity

Substituting $v^* = V \sqrt{\frac{\lambda}{8}}$ and introducing flow depth (D) on both sides yields:

$$\frac{\delta'}{D} = \frac{32.8}{\text{Re} \sqrt{\lambda}} \quad (5.1-3)$$

The thickness of the laminar layer therefore decreases with Reynolds number. For rough surfaces the zone of turbulent flow is clearly related to the ratio of the magnitudes of δ' and the hydraulic roughness (k_s). Since $\frac{\delta'}{D}$ is inversely proportional to $\text{Re} \sqrt{\lambda}$, the quantity

$$\frac{\text{Re} \sqrt{\lambda}}{D} \text{ will be proportional to } \frac{k_s}{\delta'}$$

Each of the curves in fig 5.1-1 should therefore deviate from the smooth pipe law at the same value of $\frac{k_s}{\delta'}$. Thus superimposing all curves by plotting figure 5.1-4, shows that the transition from the smooth law begins when δ' is approximately $4k_s$ and ends when k_s is approximately $6\delta'$ (See figure 5.1-4). It is only at the latter condition when $\delta' = k_s/6$ that the hydraulic roughness is no longer controlled by viscous effects at the boundary.

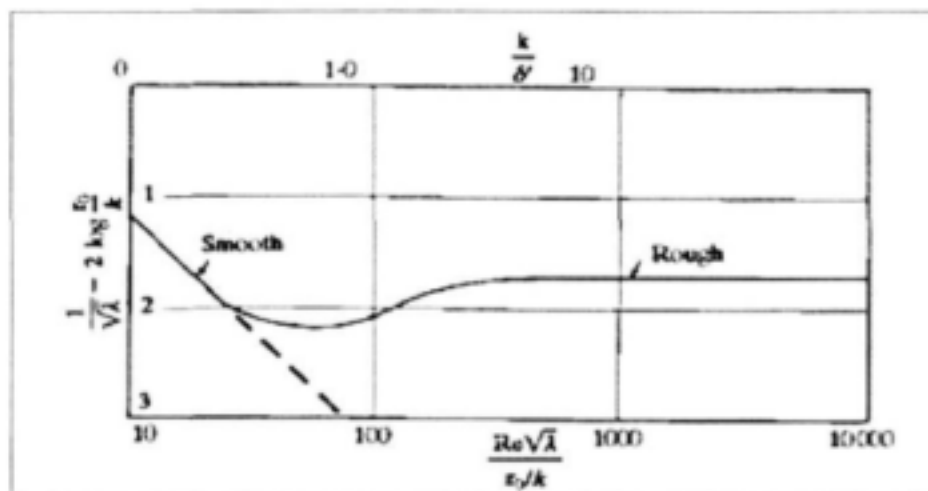


Figure 5.1-4 The transition zone for artificially roughened pipes

The boundary layer theory can be used to design operational rules of tunnels. Figure 5.1-5 shows the hydraulic roughness versus required discharge to achieve rough turbulent flow conditions in the Orange-Fish Tunnel. In the new tunnel with 0.13 mm roughness, all flow conditions will have a laminar sub-layer, even at the maximum discharge capacity of 57 m³/s. However, when the roughness reaches say 1.5 mm as the tunnel ages, the discharge should be less than 27 m³/s to maintain a laminar boundary layer.

In the case of soft water corrosion, possible damage to the concrete will be limited by operation with a laminar boundary layer, since the settling velocity of sediment is higher in laminar conditions than in turbulent flow. In the Orange-Fish Tunnel case, assuming the water is soft, the tunnel should be operated initially at say $50 \text{ m}^3/\text{s}$ and as the roughness increases later, the maximum discharge can be decreased to maintain a laminar layer, but the minimum dominant discharge should not be less than the discharge associated with the ultimate roughness caused by sediment deposition, to limit hydraulic ageing. Therefore the ideal condition would be to maintain a laminar sub-layer when the tunnel is still new and to allow some sediment deposition to occur which would protect the soft corroded upper layer of the concrete.

In the case of transfer of hard water where damage to the tunnel by the flow is not possible, the operating discharge should be as high as possible and in the rough turbulent zone when the tunnel ages, to limit sediment deposition.

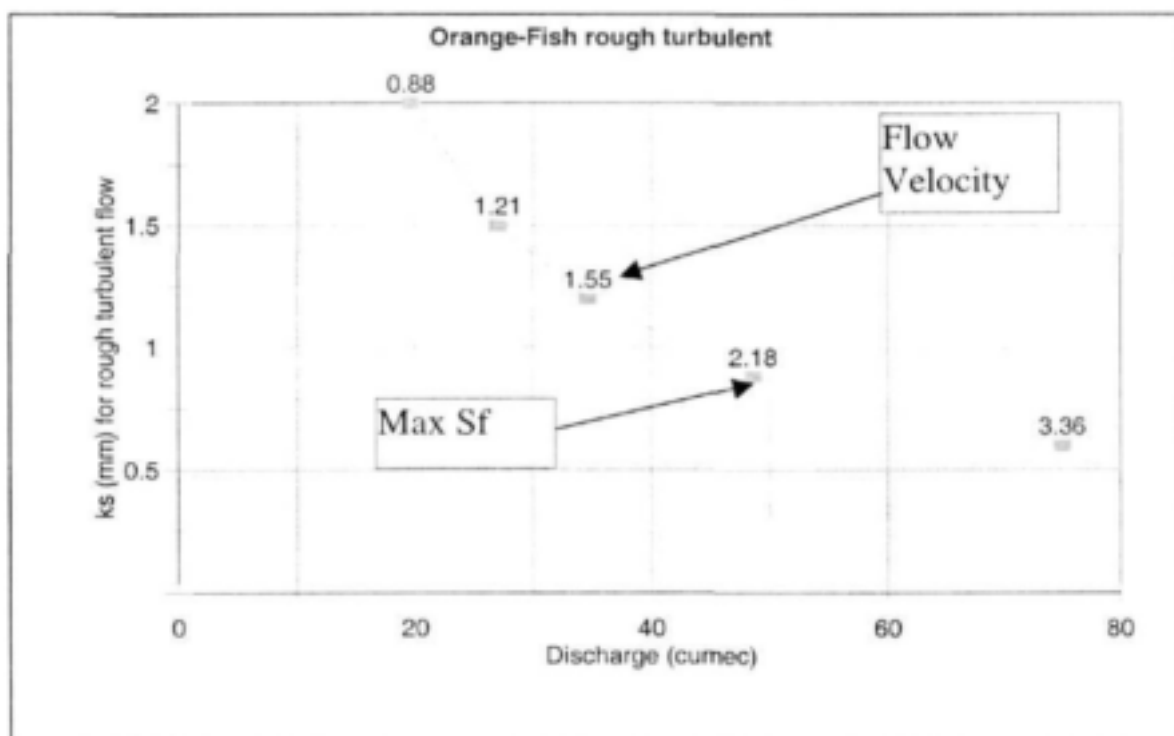


Figure 5.1-5 Laminar sub-layer formation at Orange-Fish Tunnel

5.2 Friction loss equations

One of the earliest attempts at providing a scientific basis for hydraulic channel design was Chezy's work round about 1775. Many empirical equations have since been developed, some of which have remained in use to this day. Exponential forms such as the Manning equation have been the most popular because of the ease of manipulation. However, most empirical formulae are based on limited data, and, as in general they have no sound physical basis, extrapolation outside the range of experimental confirmation may lead to serious error.

When the need for dimensional homogeneity was appreciated, and non-dimensional parameters such as the Reynolds number were available to represent the relationships in problems containing many variables, the physics of fluid friction became more apparent. In the 1930s Von Karman and Prandtl published theories of turbulent flow, which, coupled with experiments on smooth pipes and on artificially roughened pipes (Nikuradse), provided the foundation for major advances in design techniques. These theories were used as basis by Colebrook and White (1937) in their derivation of an equation describing the frictional resistance of pipes.

The Prandtl- Karman theory of turbulence led to the following flow equations:

Smooth turbulence:

$$\frac{1}{\sqrt{\lambda}} = 2 \log \text{Re} \sqrt{\lambda} - 0.8 \quad (5.2-1)$$

Rough turbulence:

$$\frac{1}{\sqrt{\lambda}} = 2 \log \frac{D}{2 k_s} + 1.74 \quad (5.2-2)$$

where

λ = friction coefficient, $2gDS/V^2$

k_s = a linear measure of effective hydraulic roughness

D = tunnel diameter

Re = Reynolds number, VD/ν

A rough pipe's resistance follows equation 5.2-2 at high Reynolds numbers when viscous effects are negligible. The same pipe's resistance, at low velocities or when carrying a more viscous fluid would deviate further from equation 5.2-2 as the Reynolds number reduced. Provided its roughness was not excessive, the pipe's roughness would tend to act increasingly like that of a smooth pipe, approaching equation 5.2-1. There is a gradual transition between rough-turbulence and smooth-turbulence which depends to some extent upon the character of the surface. In this region roughness and viscosity operate jointly in determining resistance over a range of conditions. Most commercial surfaces follow very similar transition curve, and the equation obtained by Colebrook reads:

Transitional flow:

$$\frac{1}{\sqrt{\lambda}} = -2 \log \left(\frac{k_s}{3.7D} + \frac{2.51\nu}{\text{Re} \sqrt{\lambda}} \right) \quad (5.2-3)$$

The transition equation incorporates the smooth-turbulent and rough-turbulent formulae and is therefore applicable to virtually any surface, conduit size, velocity of flow, or fluid. While empirical equations such as the Chezy and Manning equations are applicable in rough turbulent flow, equation 5.2-3 has a wide range of application.

If the nondimensional variables of equation 5.2-3 are replaced by their dimensional equivalents, the following expression results:

$$V = -2\sqrt{(2gDS)} \log \left(\frac{k_s}{3.7D} + \frac{2.51\nu}{D\sqrt{(2gDS)}} \right) \quad (5.2-4)$$

where

V = mean flow velocity

g = gravitational acceleration

S_f = hydraulic gradient

ν = kinematic viscosity of fluid

Design charts have been developed in the past to solve problems using equation 5.2-4 (HR, 1990). With modern computers and calculators however, it is quite simple to determine D or S, and a trial-and-error solution or the use of charts is no longer required.

5.3 Hydraulic roughness of new conduits

Before one can establish the rate of ageing and ultimate roughness of a tunnel, it is important to know the initial conduit roughness.

5.3.1 New pipes

Several references give recommended hydraulic roughness values for new pipes.

The Wallingford charts (see Table 5.3.1-1) provide such new pipe data and are widely used (HR, 1990). The table of surface roughnesses is based on recommendations by Colebrook (1939), Rouse (1943), Lamont (1954) and Perkins and Gardiner (1982). The surfaces listed are classified as good, normal and poor examples of their respective categories, thus leaving it to the engineer's judgement to estimate the actual value to be used in any particular scheme. The range of roughnesses covered by good and poor, takes into account not only the quality of the jointing but also the variation of surface roughness to be found in pipes that are normally of the same material.

Table 5.3.1-1 Recommended new pipe roughness values k_s (mm) (HR, 1990)

Classification:	Good	Normal	Poor
Description			
Smooth material: Drawn non-ferrous pipes of aluminium and copper, and non-metallic pipes of glass, Perspex etc.	-	0.003	-
Asbestos-cement	0.015	0.03	-
Metal			
Spun bitumen or concrete lined	-	0.03	-
Uncoated steel	0.015	0.03	0.06
Coated steel	0.03	0.06	0.15
Galvanised iron, coated cast iron	0.06	0.15	0.3
Uncoated cast iron	0.15	0.3	0.6
Tate relined pipes	0.15	0.3	0.6
Concrete			
Precast concrete pipes with "O" ring joints	0.06	0.15	0.6
Spun precast concrete pipes with "O" ring joints	0.06	0.15	0.3
Monolithic construction against steel forms	0.3	0.6	1.5
Monolithic construction against rough forms	0.6	1.5	-
Clayware			
Glazed or unglazed pipes with sleeve joints	0.03	0.06	0.15
With spigot and socket joints and "O" ring seals: dia > 150 mm	-	0.06	-
Glass fibre	-	0.06	-
UPVC			
With chemically cemented joints	-	0.03	-
With spigot and socket joints, "O" ring seals at 6 to 9 m intervals	-	0.06	-

The hydraulic roughness values recommended for new pipeline design as given in Table 5.3.1-1, are useful when considering the use of different materials in the design of a new tunnel or where relining is considered. For concrete lined tunnels it is proposed that observed new tunnel roughness values for South African conditions are used.

5.3.2 New tunnels: insitu concrete lined

The observed hydraulic roughness of new tunnels, cast insitu, are indicated in Table 5.3.2-1.

Table 5.3.2-1 Observed roughness of new (or near new) cast in-situ concrete tunnels

Tunnel	Diameter (m)	Manning n
New large concrete conduits (USBR, 1965)	0.6 m to 14 m	0.0111 to 0.013
Appalachia, USA (Elder, 1956)	5.5	0.0135
Colorado River Aqueduct, USA (Horowitz and Lee, 1971)	6.5	0.011 to 0.012
Sante Etlah, USA	3.4	0.011
Chicago North, USA	Unknown	0.012
Large conduits: US Army Waterways Experimental Station	-	0.010 to 0.013 (age 0 to 8 years)
Slayand Inverawe, Scotland (Williams & Partners, 1952, 1964)	unknown	0.0125
Glenlec (BW Engineers, 1984)	3.4	0.0137
Fasnakyk (BW Engineers, 1984)	4.5	0.0132
Rheidol, Wales	Unknown	0.0125
Orange-Fish	5.33	0.0119
Kendoon, Scotland	1.6 m to 7 m	0.0120
LHWP Transfer Tunnel	4.35	0.0111 (design: 0.0123)
LHWP Delivery Tunnel	4.50	0.0116

5.3.3 New tunnels: prefabricated segmentally lined concrete segments

Segmental linings are being used increasingly for construction of tunnels to carry water and sewage, yet there seems to be disagreement about the most important measure of hydraulic performance – hydraulic roughness. As a result, many authorities impose apparently arbitrary penalties on the use of such linings. For example for a nominal 6 m diameter tunnel in the USA, an increase of 9 % in diameter was specified if a precast segmental lining were to be used rather than an in-situ lining (Pitt and Ackers, 1982).

Other factors, such as durability and speed of construction and available construction equipment, as well as hydraulic performance, can also influence the size and cost of tunnels. In South Africa, the Delivery Tunnel North (Figure 5.3.3-1) followed the segmentally lined construction technique. Commissioning test data of this tunnel indicated a $k_s = 0.15$ mm, much less than expected during design. When the commissioning test data were compared with other international tunnel data, the general relationship of the equation proposed by Pitt and Ackers (equation 5.3.3-1) was found, but with a completely different coefficient.



Figure 5.3.3-1 Segmentally lined Delivery Tunnel North

When segmentally constructed tunnels age, the secondary losses created by the panel joints could actually decrease with time due to sediment deposition and sliming. For accurate prediction of the ageing rate it is important to know the panel roughness and the effective roughness caused by the steps between panels.

The Pitt and Ackers (1982) equation related k_s to the mean step height $|s|$ (ignoring sign) and the segment length l . Hydraulic data obtained from 5 tunnels were used by Pitt and Ackers to calibrate their equation:

$$k_s = k_0 + x(|s|)^2 / l \quad (5.3.3-1)$$

with k_0 the new panel roughness, calibrated as 0.3 mm, x = calibrated coefficient and k_s in mm. The constant of 0.3 mm is the roughness of the segments only, excluding the joint losses. The tunnels tested were constructed during the 1970s and the initial roughness of current in situ and segmentally lined tunnels (panels only) is typically 0.03 to 0.08 mm, based on the LHWP commissioning tests of the Transfer and Delivery Tunnel North respectively.

The calibration of equation 5.3.3-1 is shown graphically in Figure 5.3.3-2.

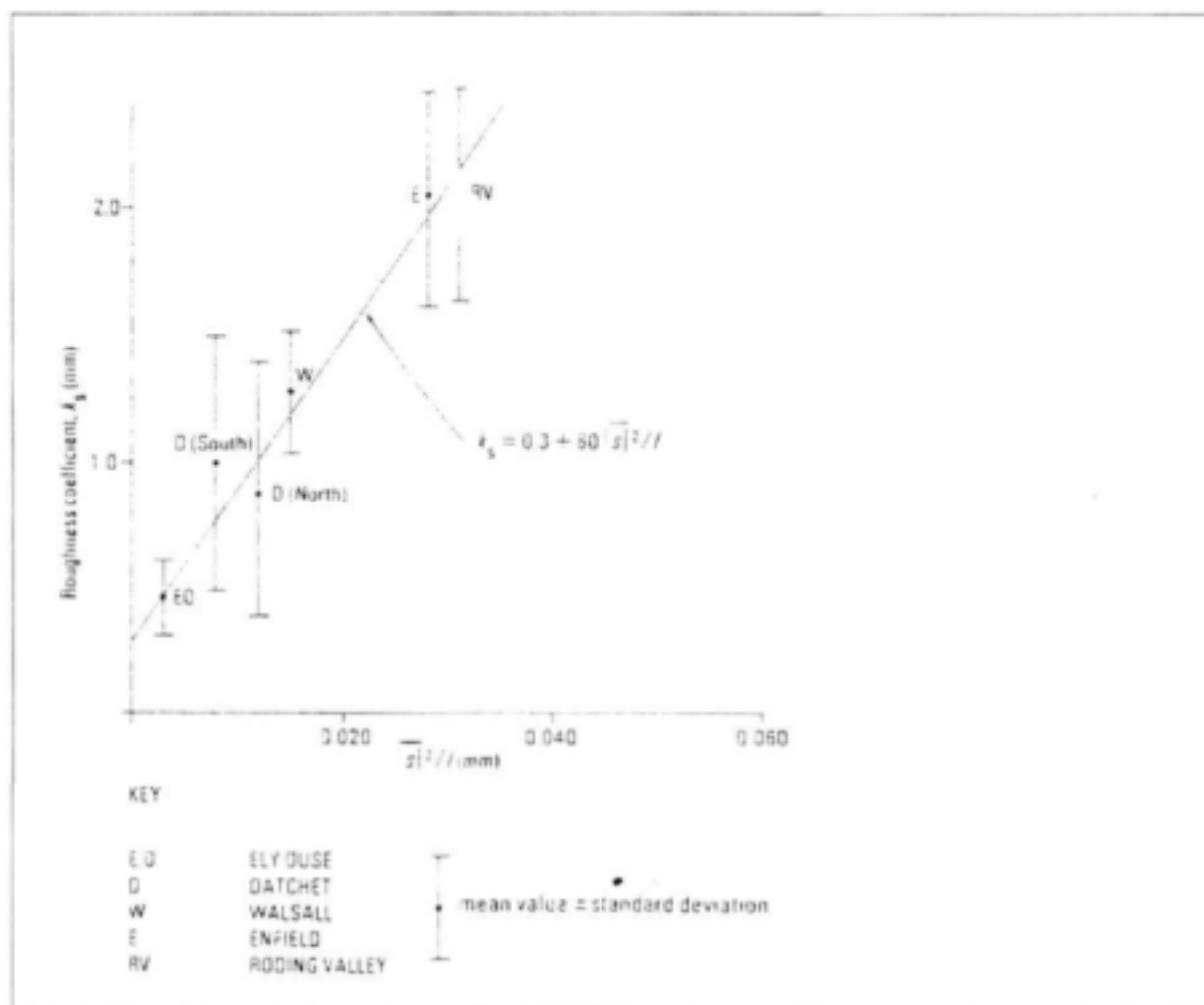


Figure 5.3.3-2 Roughness height as a function of step height to step length (Pitt and Ackers, 1982)

Viljoen and Metcalf (1999) proposed a recalibrated equation, using the DTN commissioning test data as well as data of the Orange-Fish Tunnel. While the latter tunnel has been constructed in situ, it has a lining pattern similar to a segmentally lined tunnel with 1.52 m segments (between shutter joints) and mean step height of 6 mm. They proposed $k_0 = 0.03$ mm, but calibrated $x = 0.65$, which is significantly different from the value proposed by Pitt and Ackers of 60. The British and South African tunnel data therefore appear to differ, probably because of different hydraulic conditions, and the s/l relationship on its own does not seem to be a good predictor of the total roughness.

Viljoen and Metcalf (1999) proposed equation 5.3.3-2 which gives the percentage increase in head loss (y), over that due to wall friction k_s alone, due to mean step size s , in mm units. Data of only the Orange-Fish Tunnel and the Delivery Tunnel North were used in the calibration, with $k_0 = 0.08$ mm. A correlation coefficient of only 0.63 was obtained and the equation is only applicable for tunnel diameters of 4.6 m to 5.5 m.

$$y = 1.75 \ln[s] + 4.2 \quad (5.3.3-2)$$

Equation (5.3.3-2) has been calibrated on a small data set and with a fixed segment length to diameter ratio of 0.3. A new approach based on applied stream power will be followed here in order to establish a new equation calibrated on British and SA tunnels.

For equilibrium sediment transport conditions under steady, uniform open channel flow conditions, when the power applied to maintain suspension near the bed is equated with the power required to suspend a sediment particle, the following relationship is found:

$$\left(\tau \frac{dv}{dy} \right)_0 = (\rho_s - \rho)gw \quad (5.3.3-3)$$

where

- τ = bed shear stress
- v = velocity
- y = depth measured from bottom
- ρ_s = sediment density
- ρ = density of water
- g = gravitational acceleration
- w = particle settling velocity

The left hand side of equation 5.3.3-3 represents applied stream power at the bed and can be rewritten as:

$$30\rho g S_f D \sqrt{g S_f D} / \kappa k_s = (\rho_s - \rho)gw \quad (5.3.3-4)$$

where

- κ = Von Karman coefficient = 0.4 for clear water
- D = open channel flow depth
- S_f = energy slope

When equation 5.3.3-4 is written in terms of relative roughness, equation 5.3.3-5 is obtained:

$$\frac{D}{\kappa k_s} = \frac{\rho_s - \rho}{30\rho} \frac{w}{\sqrt{g S_f D}} S_f \quad (5.3.3-5)$$

Simons and Senturk (1992) found through dimensional analysis, that in cobble and boulder bed rivers with large scale roughness, the Darcy-Weisbach friction factor is related to the density ratio, the shear velocity and the d_{50}/d_{84} ratio of the boulders. The Darcy-Weisbach factor is related to the Chezy C which is a function of D/k_s , the left hand side of equation (5.3.3-5). Although there is no (or limited) sediment transport in cobble bed rivers and none in the segmentally new tunnels under consideration, the power is balanced and with a calibrated equation (5.3.3-5) the total hydraulic roughness of a segmentally lined tunnel can be predicted.

The data considered for the calibration are indicated in Table 5.3.3-1.

Table 5.3.3-1 Segmentally lined tunnel data

Tunnel	Type	Hydraulic radius (m)	Flow velocity (m/s)	S_f (m/m) ($\times 10^3$)	Observed new k_s (mm)	Mean step height (mm)	Step length L (m)	Age when tested (yr)	Pitt & Ackers (1982) k_s calc
Dacht North	Raw water & full	0.635	1.2	0.47	1.0	2.7	0.6	8	1.03
Dacht South	Raw water & full	0.635	0.58	0.10	1.0	2.2	0.6	8	0.78
Ely Ouse	Raw water & full	0.635	0.62	0.11	0.45	2.3	0.6	5	0.83
Enfield	Sewer & part full	0.14	0.8	1.75	1.7	4	0.6	2	1.90
Roding Valley	Sewer & part full	0.145	0.75	1.55	2.1	4.3	0.6	6	2.15
Wallsall	River water & part full	0.23	1.33	2.11	1.3	3.5	0.6	0	1.53
Orange-Fish	Raw water & full	1.33	2.54	0.60	0.13	6	1.52	0	1.72
Delivery Tunnel North	Raw water & full	1.15	2.95	0.90	0.15	8	1.4	0	2.83

Of the 7 tests, the two which yielded the highest values of k_s were in sewer tunnels after 2 to 6 years operation, and not new pipelines. It may be argued that hydraulic resistance was increased as a result of slime on the surface, but inspection showed that there was little sliming.

The last column of Table 5.3.3-1 shows the calculated k_s values based on the Pitt and Ackers equation. It is clear that the equation cannot predict the hydraulic roughness of the South African Tunnels, even if the k_o value in the equation is adjusted from 0.3 mm to 0.03 mm.

The calibration of equation 5.3.3-5 was carried in a number of steps, as indicated in Table 5.3.3-2. Firstly only the UK data were used with the proposed equation 5.3.3-5. Then a dimensionless parameter R/s was introduced to account for the joint losses and calibrated on the UK data with good results. An additional dimensionless parameter was then introduced: L/s to account for the suddenness of the joint transitions (expansion/convergence), and UK and SA data gave an improved calibration 5.3.3-6.

$$\frac{R}{k_s} = 4.8 \times 10^{-6} V^{*6.377} S_f^{-3.31} \left(\frac{R}{s} \right)^{-1.79} \left(\frac{L}{s} \right)^{1.76} \quad (5.3.3-6)$$

Table 5.3.3-3 Calibration of segmentally lined tunnel relative roughness (R/ks)

Tunnels	V*	S	R/s	L/s	R/L	r ²
UK	yes	yes	no	no	No	0.94
UK	yes	yes	yes	no	No	0.95
UK & SA	yes	yes	yes	yes	No	0.98
UK & SA	yes	yes	no	no	yes	0.98

With $V^* = \sqrt{gRS_f}$, w , κ , ρ , ρ_s = constant.

Based on equation 5.3.3-6 k_s decreases when V^* increases, when S decreases, when R decreases, and when L increases.

The mean step heights in equation 5.3.3-6 however cancel out (from the regression analysis) and that is why an additional equation 5.3.3-7 was calibrated with a third parameter R/L to account for the number of joints. Equation 5.3.3-7 gives the same prediction accuracy as equation 5.3.3-6. Theoretically, it does not make sense that the step height is not part of the equation, but its unimportance is probably due to the available data, where step heights to L ratios have a narrow range of values. It is proposed that equation 5.3.3-6 is recalibrated in future when more data for larger steps are also available.

$$\frac{R}{k_s} = 0.977V^{*6.376} S_f^{-3.31} \left(\frac{R}{L}\right)^{-1.79} \quad (5.3.3-7)$$

The predicted and observed roughness values k_s of the calibrated tunnels are shown in Table 5.3.3-3 and Figure 5.3.3-3.

Table 5.3.3-3 Predicted and observed segmentally lined tunnel roughness

Tunnel	Observed k_s (mm)	Predicted k_s with eq 5.33-6 (mm)	Predicted k_s with eq 5.3.3-7 (mm)
Dachet North	1.0	0.85	0.85
Dachet South	1.0	0.71	0.71
Ely Ouse	0.45	0.71	0.71
Enfield	1.7	1.81	1.81
Roding Valley	2.1	1.76	1.76
Wallsall	1.3	1.52	1.52
Orange-Fish	0.13	0.12	0.12
Delivery Tunnel North	0.15	0.16	0.16

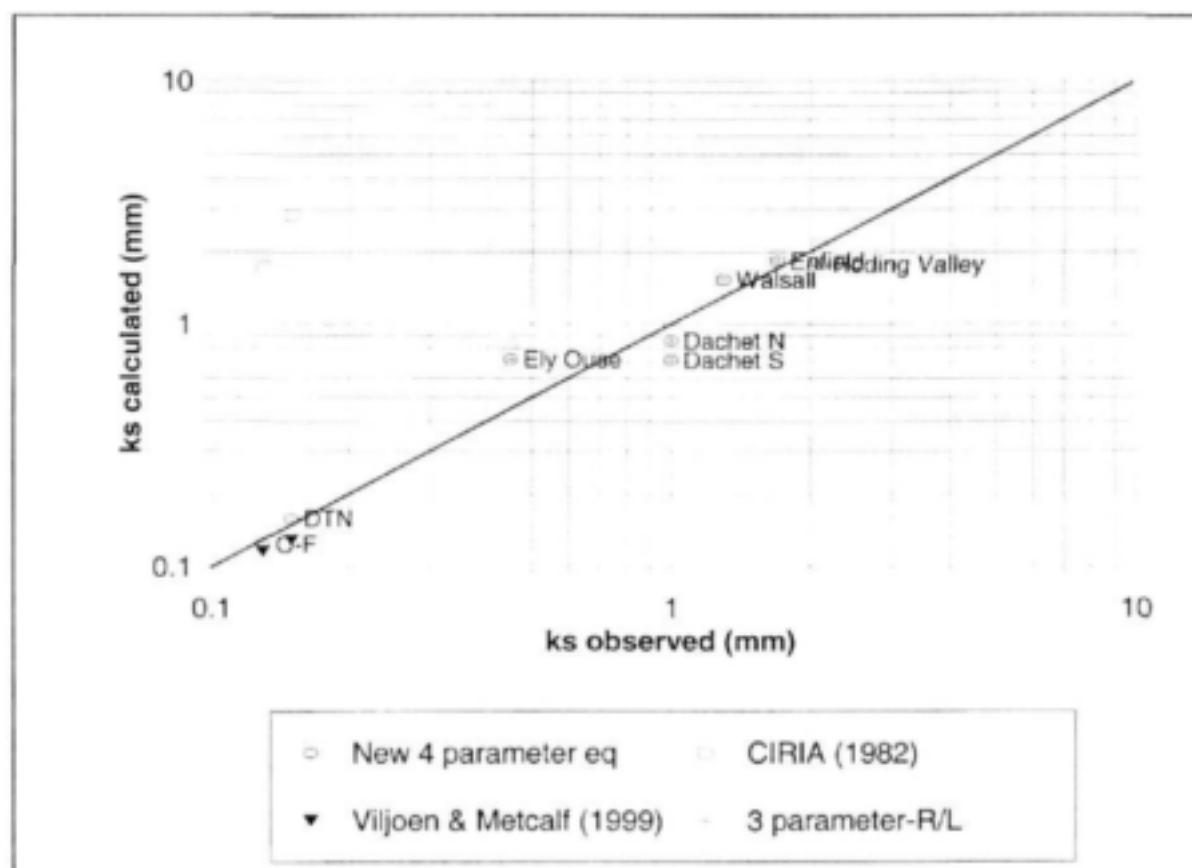


Figure 5.3.3-3 Segmentally lined tunnels new roughness prediction

The proposed methodology gives reliable k_s predictions for UK and South African tunnels for a wide variety of conditions, for diameters ranging from 1.0 m to 5.33 m, for full flow and partially full flow conditions, with mean step heights at joints ranging from 2.2 to 8 mm, and with a wide range of new tunnel roughness $k_s = 0.13$ to 2.1 mm. It is believed that the UK data used in the calibration is less robust than the South African data, due to the free flow in some, and slightly aged conditions in these tunnels. In future more commissioning test data should be gathered to refine the calibration.

It is proposed that for larger joint step heights ($s > 10$ mm), the secondary losses created by the joints should be calculated separately by using theory for sudden contractions and expansions.

5.4 Critical conditions for deposition of sediment

In pipe sediment transport, especially at high sediment concentrations, there is a critical relationship between velocity, energy slope and concentration. For a specific sediment concentration, there is a minimum energy slope and when the velocities decrease further, sediment is deposited at the bottom of the pipe which decreases the flow area and increases the hydraulic roughness (Figure 5.4-1). The sediment deposition is due to oversaturated conditions which is not typical what is found in tunnels. Critical conditions for sediment deposition for tunnels should be based on the discussion on critical conditions for re-entrainment of non-cohesive sediment and sediment transport.

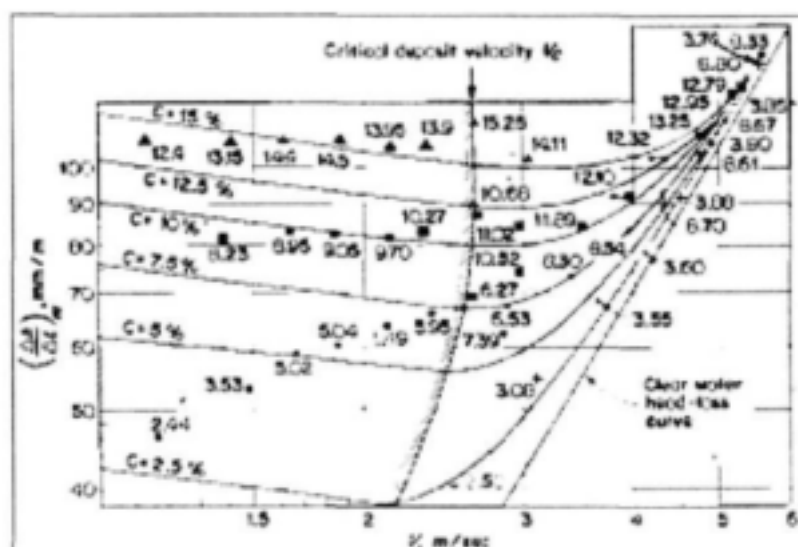


Figure 5.4-1 Relationship between velocity, energy slope and stream power (Graf, 1971)

5.5 Critical condition for re-entrainment of non-cohesive and cohesive sediment

5.5.1 Non-cohesive sediment

The re-entrainment of non-cohesive sediment can be determined by using the modified Liu diagram (Figure 5.5.1-1). When the hydraulic conditions in the tunnel plot above the line in the graph, sediment transport of a particular size sediment will occur.

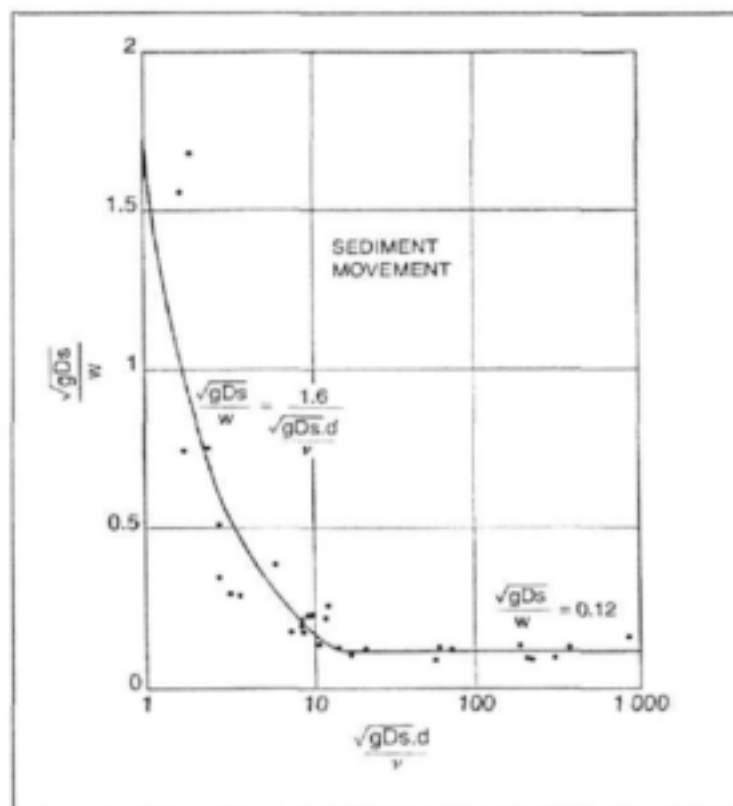


Figure 5.5.1-1 Modified Liu diagram (Rooseboom, 1992)

Figure 5.5.1-1 has been used to analyse a case study: Orange-Fish Tunnel. The calculation is for non-cohesive sediment, while the sediment in the field in this tunnel is in fact fine and cohesive. The required discharge versus sediment diameter is shown in figure 5.5.1-2. As the tunnel ages and its roughness increases, the required discharge to scour a particular sand fraction becomes slightly smaller. When the discharge is less than $10 \text{ m}^3/\text{s}$, a laminar sub layer exists and the stream power required for re-entrainment of the sediment is relatively high. In the rough turbulent zone there is initially a sharp rise in discharge required to scour 2 mm versus 1 mm diameter sediment. At the current maximum tunnel discharge capacity, sediment of 20 mm in diameter will be re-entrained and transported through the tunnel.

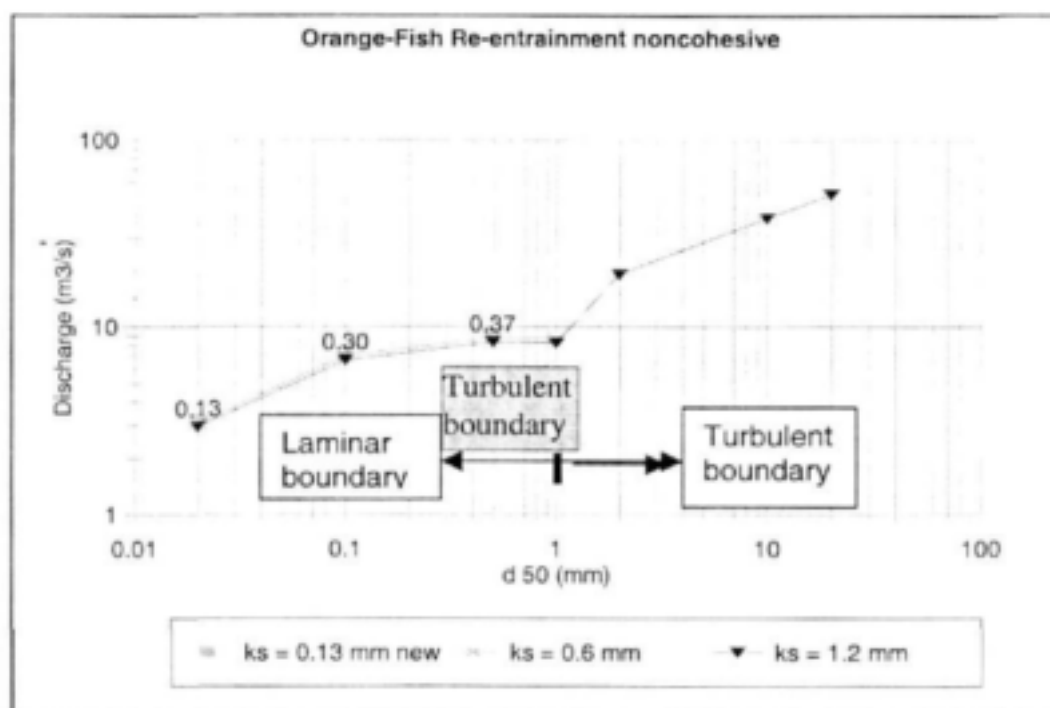


Figure 5.5.1-2 Re-entrainment of sediment with laminar and turbulent boundary conditions

5.5.2 Cohesive sediment

Critical conditions to scour cohesive sediment is usually expressed in terms of the bed shear stress. The critical shear stress depends on a number of factors such as the physical and chemical characteristics of the sediment, the density of the sediment, its shear strength and percentage clay. Most of the South African tunnels transfer water from reservoirs after deposition of coarse sediment. It is the fine sediment that is transported into the tunnels and deposits in uniform layers in the tunnels. Field data should as far as possible be used in the determination of a critical shear stress for mass erosion of the cohesive sediment. Data obtained from reservoir sediment deposits (fine sediments) can however also provide useful information as shown in Figure 5.5.2-1. The data were obtained from South African and Chinese Reservoirs. The maximum and minimum lines can be used to scour sediment in hard water conditions or to limit re-entrainment of sediment in soft water conditions, respectively.

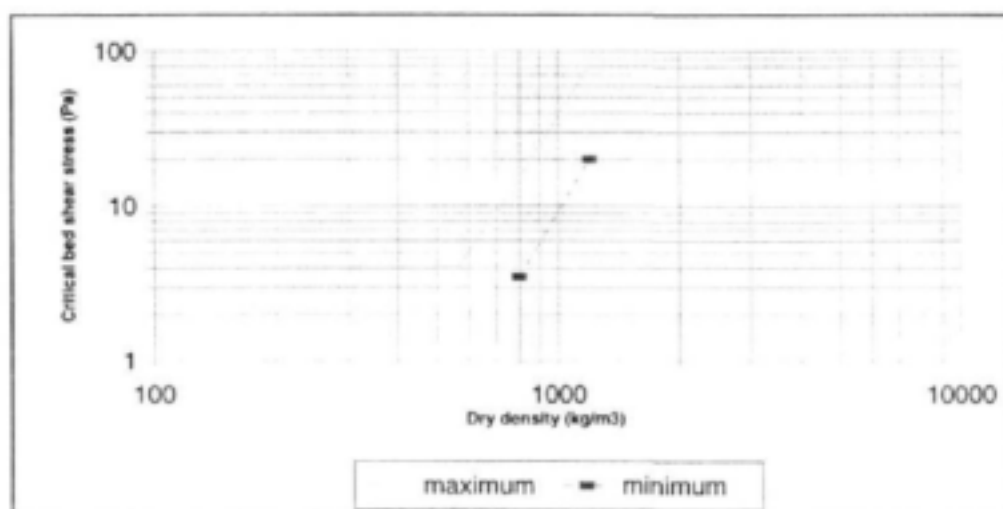


Figure 5.5.2-1 Critical shear stress for mass erosion of cohesive reservoir sediment in open channel flow

The field data obtained during this study during tunnel flushing also provide useful information on critical shear stresses and input stream power in a tunnel, as shown in Table 5.5.2-1. Unit input stream power (P) is better correlated with sediment transport than shear stress and is described by

$$P = \rho g V S_f$$

where V = mean flow velocity and S_f = energy slope

Table 5.5.2-1 Tunnel shear stress and stream power

Tunnel	Flow velocity (m/s)	Wall shear stress (Pa)	Input stream power (W/m)	Comments on field or laboratory observations
Roo de Els berg	0.57	6.5	7.4	Some scour = surface erosion with $C = 300$ mg/l
Theewaterskloof: Banghoek to Jonkershoek	2.0	5.5	12.6	Surface erosion limiting scour with max. $C = 2000$ mg/l
Orange-Fish	2.2	4.6	7.5	Near maximum discharge; flushed out most fine sediment previously deposited
Corrosion lab pipe test: poor quality concrete	1.06	6.9	210	No scour of soft concrete
	3.0	95	8115	Increased hydraulic roughness
Corrosion lab pipe test: DTN	2.8	19	1515	No scour of concrete
Dasbos pipe flushing test	3.2	72	9133	Almost no scour of sediment

Table 5.5.2-1 shows that at between 4.5 and 7 Pa shear stress, cohesive sediment is scoured by surface erosion. Laboratory tests indicated it would be safe to operate at a shear stress as high as 19 Pa without damaging good quality concrete in soft water conditions.

Input stream power values of between 7.4 and 12.6 W/m lead to surface erosion of the sediment, at observed concentrations of 300 to 2000 mg/l. The maximum stream power without any scour of soft concrete that should be allowed in soft water corrosion conditions is 210 W/m with a poor quality concrete, while with a good quality concrete such as the LHWP tunnels, the stream power could be 1515 W/m.

5.6 Sediment transport in closed conduits

Sediment transport in pipes can be classified as heterogeneous or homogeneous flow, depending on the sediment characteristics and hydraulic conditions. Fine sediment is usually transported as homogeneous flow, with a near uniform distribution across the pipe or tunnel. Coarse sediment however is transported with a higher sediment concentration near the invert of the pipe, known as heterogeneous flow.

The Durand-Condolios relation (1953) can be used to describe heterogeneous transport of sand.

$$\phi_D = 180 \left[\frac{V^2}{gD} \sqrt{\frac{gd}{w^2}} \right]^{-1.5} \quad (5.6-1)$$

where ϕ_D is a dimensionless sediment-transport parameter (refer to equation 5.6-2), describing the excess head loss due to the presence of solid particles, V is the flow velocity, g is the acceleration due to gravity, D is the pipe diameter, d is the particle diameter and w is the terminal falling velocity of a particle in water.

$$\frac{\left(\frac{\Delta h}{\Delta L} \right)_m - \left(\frac{\Delta h}{\Delta L} \right)_l}{C \left(\frac{\Delta h}{\Delta L} \right)_l} = \phi_D \quad (5.6-2)$$

with C = volumetric sediment concentration, L = liquid, M = mixture and h/L the energy slope.

This equation is empirical and based on 310 test runs, where the pipe diameter varied between 40 and 580 mm, the sediment diameter varied between 0.2 to 25 mm, and the concentration varied from 50 g/l to 600 g/l, supplemented with particle relative density variation of 1.60 to 3.95. Figure 5.6-1 shows the calibration data used by Durand.

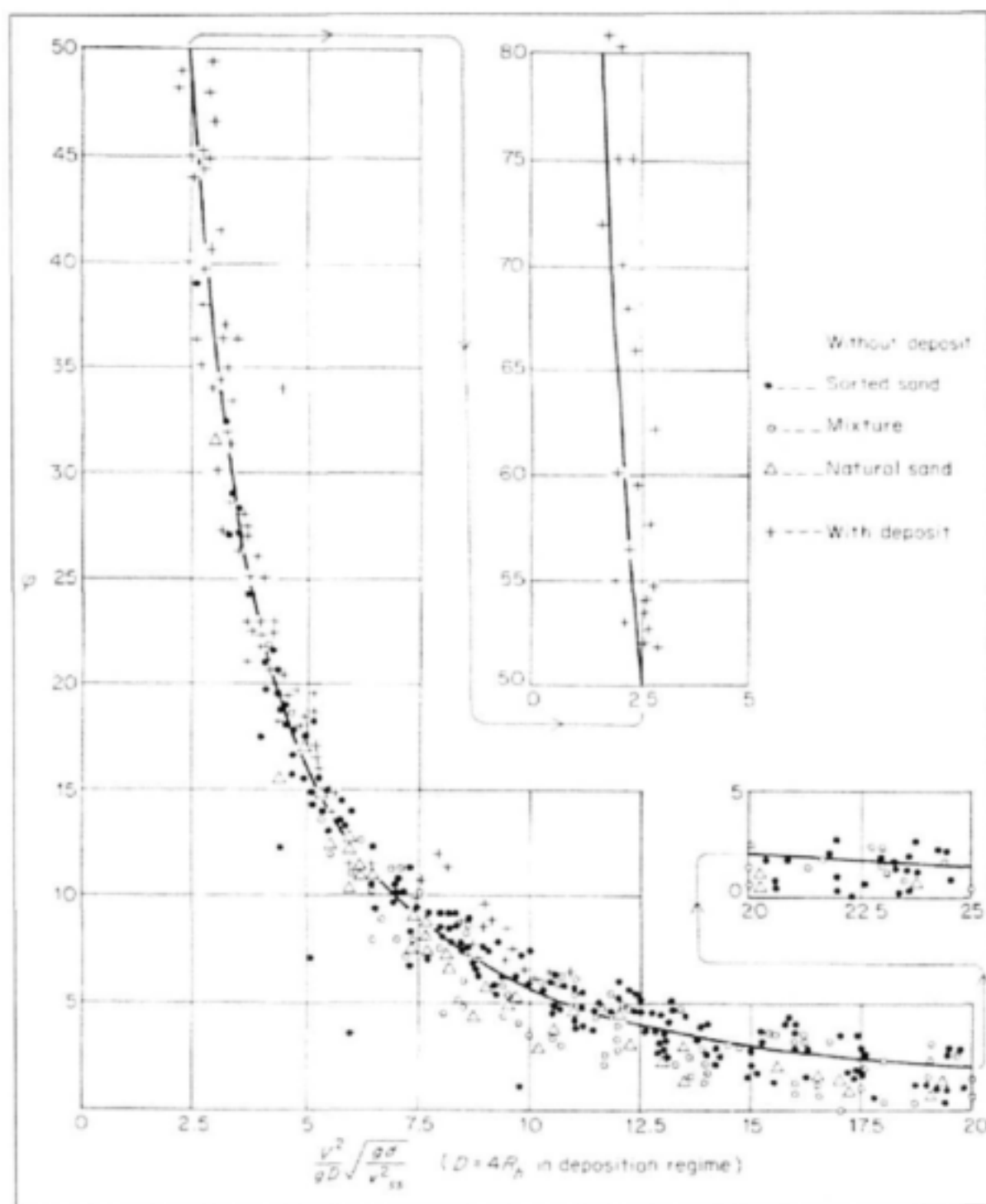


Figure 5.6-1 Head loss as given by Durand-Condolios relation for sand (Durand and Condolios, 1953)

The lower limit of the equation where sediment deposition is limited was given by Durand as

$$\frac{V_c}{\sqrt{2gD(s_s - 1)}} = F_L \quad (5.6-3)$$

where V_C is the critical velocity and s_s is relative weight of the sediment-water mixture. This velocity separates the deposit-free regime from the deposit regime and is indicated in Figure 5.6-2.

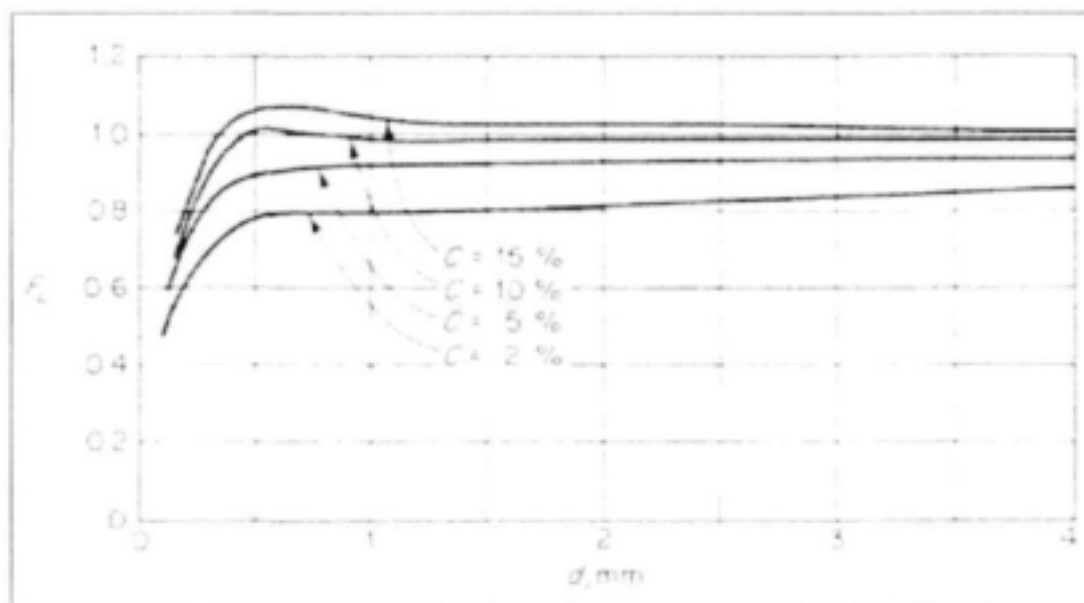


Figure 5.6-2 F_L values (Durand, 1956).

The relationships for sediment transport developed by Durand can be used to determine whether sediment entering the tunnel would be transported through. It is applicable to sand and gravel fractions.

Suspensions of very fine (0.003 mm to 0.03 mm) sediment particles behave as homogeneous fluids. Newitt et al. (1955) proposed an equation (5.6-4).

$$\frac{\left(\frac{\Delta h}{\Delta L}\right)_m - \left(\frac{\Delta h}{\Delta L}\right)_L}{C \left(\frac{\Delta h}{\Delta L}\right)_L} = (s_s - 1) \quad (5.6-4)$$

where s_s is the ratio of the density of the solids to that of the liquid.

5.7 The rate of ageing

5.7.1 Pipelines

Colebrook (1938) presented ageing data of the Thirl Aqueduct in the UK. The roughness was determined at intervals of a few months over a period of 14 years, and show the gradual increase in resistance in two tarred cast-iron pipes. Both pipes, with the largest having a 1.76 m diameter, showed a very clear linear relationship between hydraulic roughness and time. The data of one of the pipes were plotted by Colebrook as shown in Figure 5.7.1-1.

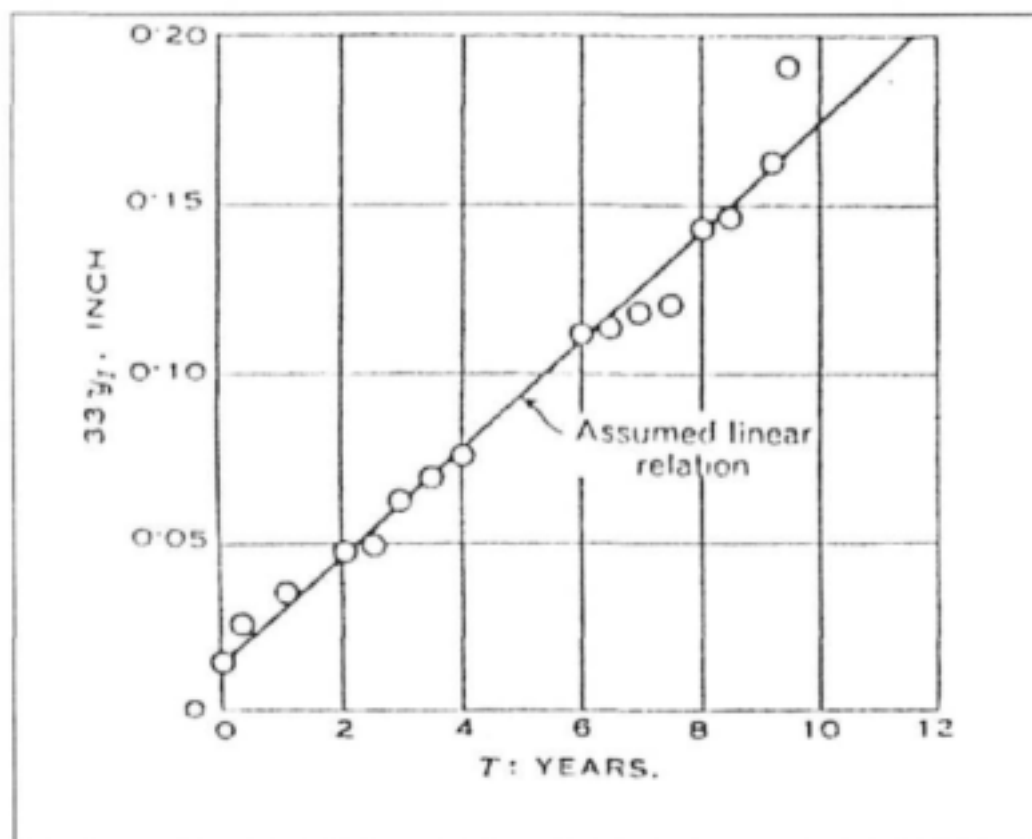


Figure 5.7.1-1 Observed linear roughness increase with time of Thirlmere pipeline (Colebrook, 1938)

Perkins and Gardiner (1982) carried out hydraulic roughness ageing tests on slimed sewers and found that slime builds up on a sewer very quickly and once this initial sliming has taken place, there is no evidence of a subsequent continuous increase. The roughness varies, suggesting that there is a continuous process of slime building and removal taking place. This is not what is expected in tunnels where sediment availability limits the rate of ageing and the process is often not dominated by sliming.

5.7.2 Effect of different pipe material

The pipe or tunnel material could have an effect on the hydraulic ageing. During tests carried out to determine the ageing of slimed sewers, Perkins and Gardiner (1982) found that slime built up quickly and that the pipe material had an effect on its roughness as shown in Table 5.7.2-1.

Table 5.7.2-1 Roughness values for different sewer pipe material (Perkins and Gardiner, 1982)

Material	Roughness values k_s (mm)		
	High	Median	Low
Vertically cast concrete	3.8	1.8	1.3
Spun concrete	4.2	2.3	1.8
Asbestos-cement	2.8	1.8	1.2
Clay	2.3	1.1	0.6
uPVC	1.1	0.6	0.6
Note: velocities about 0.75 m/s			

5.7.3 Development of a hydraulic predictor of the rate of ageing by sliming and sediment deposition

a) Colebrook (1938,1958)

Decreased hydraulic roughness with time could be due to the effects of sliming which has been found to be more severe as pH decreases, or due to corrosion of the concrete due to soft water attack.

The rate of ageing is usually taken as linear over time in mm/year. If the initial new tunnel roughness k_0 is known, and the rate of ageing α , it is possible to calculate future aged tunnel roughness using equation 5.7.3-1.

$$k_t = k_0 + \alpha T \quad (5.7.3-1)$$

where T is years after commissioning.

This equation was tested against pipe data by Colebrook (1938), first with Thirlmere pipeline data (Figure 5.7.3-1), and also with typical USA pipeline data (Figure 5.7.3-2). He also combined the data and plotted them on a logarithmic time scale (Figure 5.7.3-3). The data showed some scatter due to seasonal temperature changes and chemical variations, superimposed upon the already large experimental errors to be expected in such work (Colebrook, 1938). The data show clearly that a linear relationship exist between the increase of roughness with time and age of a conduit, and that an equation in the form of equation 5.7.3-1 can be used to determine hydraulic ageing of pipes or tunnels.

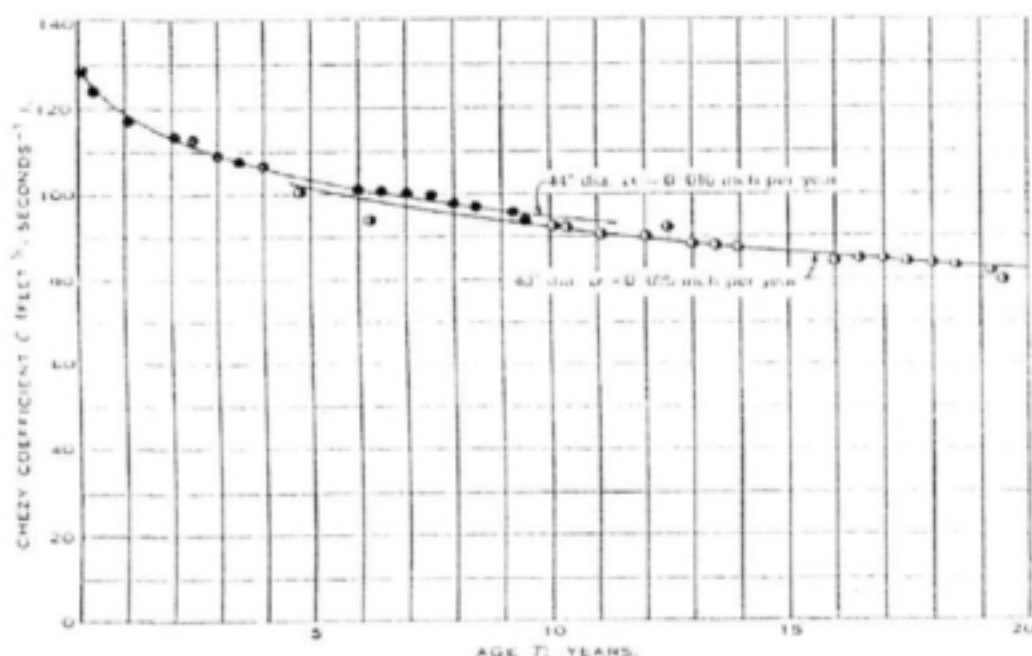


Figure 5.7.3-1 Thirlmere pipeline rate of ageing (Colebrook, 1938)

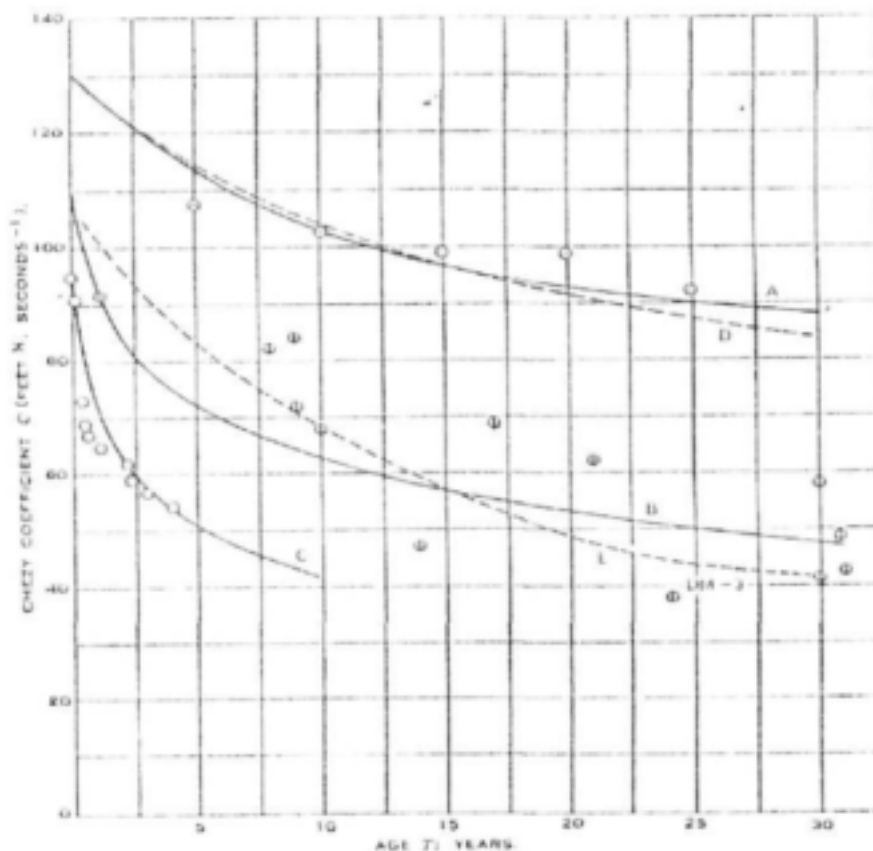


Figure 5.7.3-2 USA pipeline rate of ageing (Colebrook, 1938)

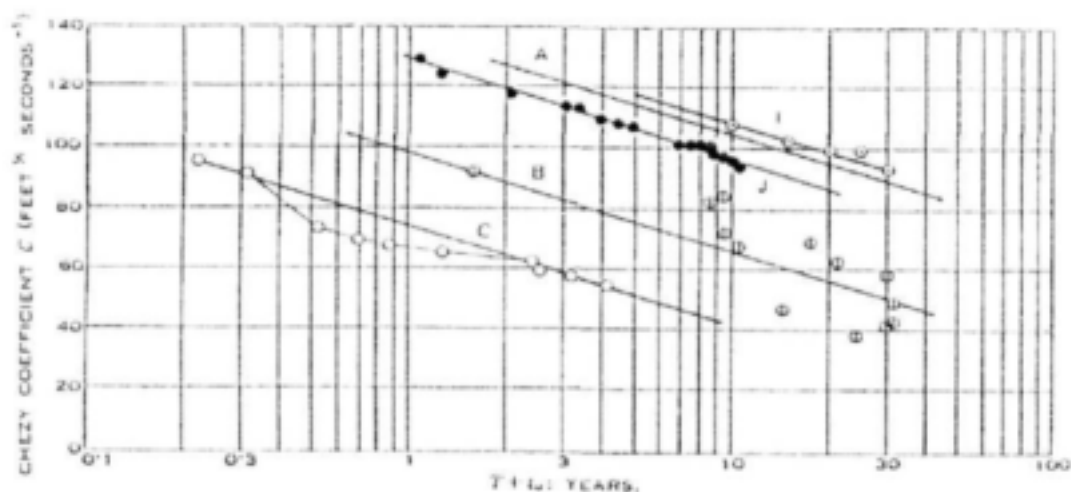


Figure 5.7.3-3 UK and USA pipeline rate of ageing (Colebrook, 1938)

Colebrook (1958) found a relationship between the rate of ageing of a concrete lined tunnel and the water pH. He also established a relationship to determine the ultimate tunnel roughness k_s as function of flow velocity and hydraulic radius:

$$k_s = 0.38R^{0.26}V^{-2.2} \quad (5.7.3-2)$$

in imperial units, and R = hydraulic radius and V = flow velocity. Equation 5.7.5-2 shows that tunnel roughness is inversely related to the flow velocity. This would also apply to the rate of ageing.

In his assessment of pipeline ageing, Colebrook (1938) found that as pH decreases, the rate of ageing increase, based on UK and USA field data (Figure 5.7.3-4). Colebrook proposed the use of a preliminary ageing equation 5.7.3-3, but emphasised that the data showed considerable scatter and that other equally important factors play a role with pH.

$$2 \log \alpha = 3.8 - pH \quad (5.7.3-3)$$

where α is in inches/year for pipe ageing.

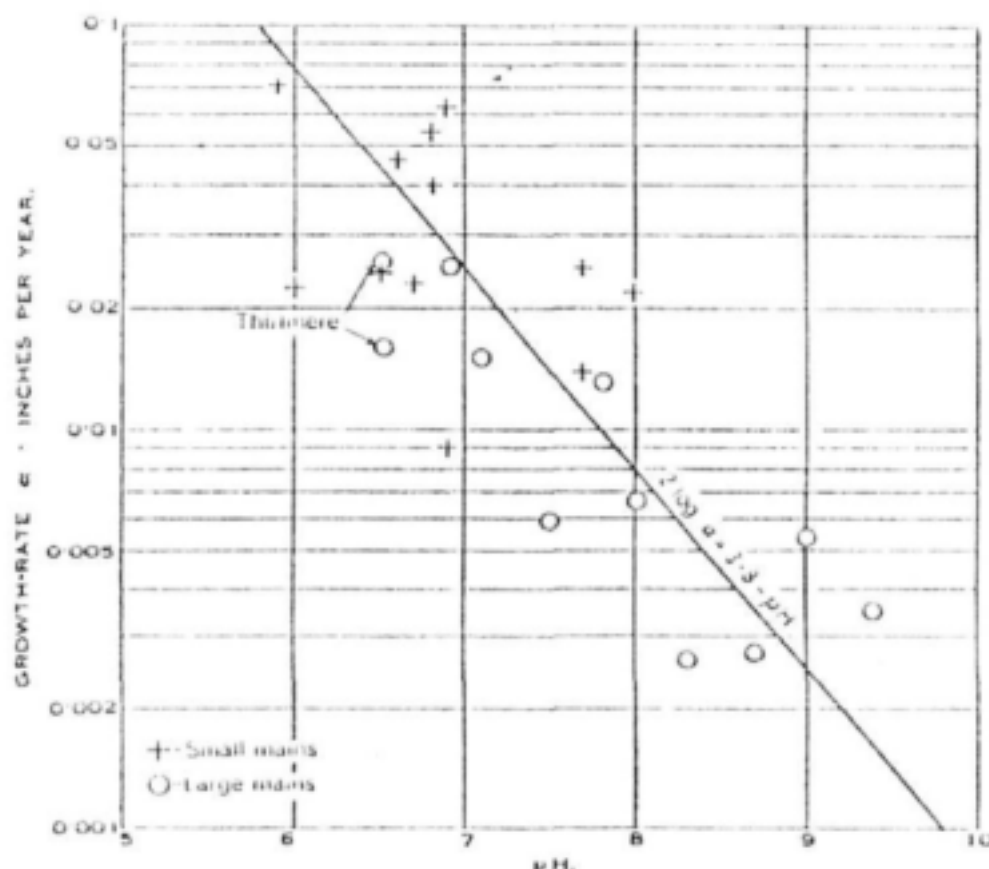


Figure 5.7.3-4 Relationship between pH and hydraulic ageing for pipes (Colebrook, 1938)

Colebrook found a similar relationship (Figure 5.7.3-5) in later work in tunnels (1958), equation 5.7.5-4, and stated that it should be regarded as tentative until further experimental data becomes available.

$$1.25 \log \alpha_{\max} = 3.4 pH \quad (5.7.3-4)$$

where α is in inches/year for tunnel ageing.

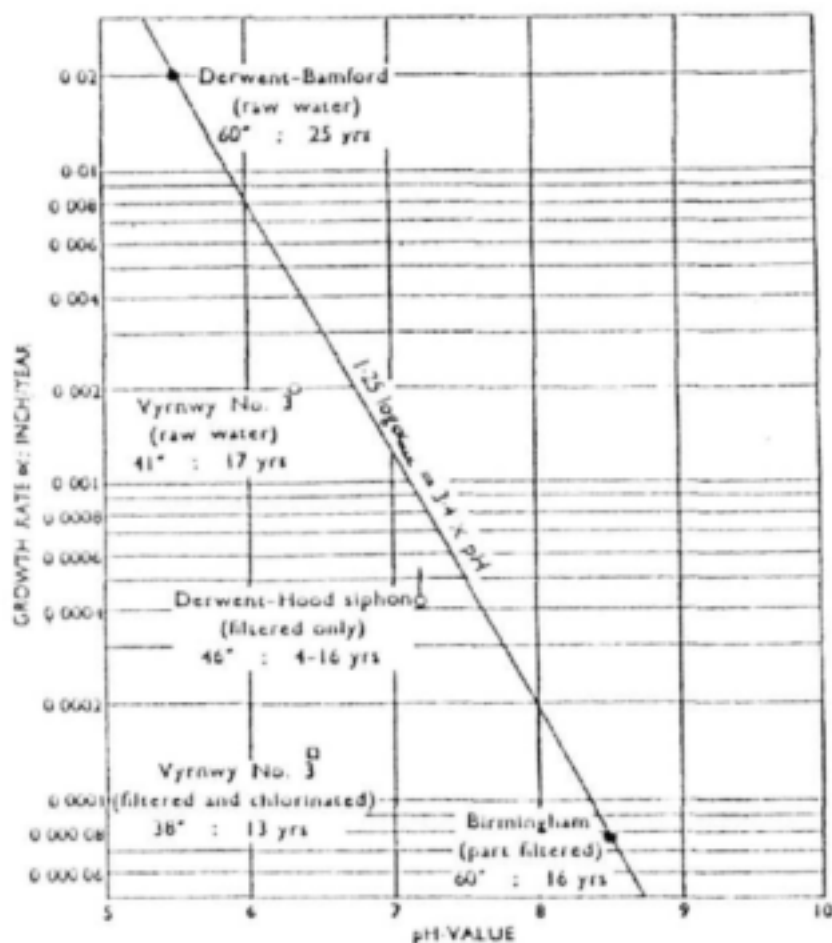


Figure 5.7.3-5 Relationship between pH and hydraulic ageing for tunnels (Colebrook, 1958)

b) Basson (1999)

Using the above observed factors which contribute to tunnel ageing, the following new calibrated equation was proposed by Basson (1999) to determine the rate of tunnel ageing α (mm/year): with V = flow velocity (m/s), ρ = fluid density, g = gravitational acceleration and S_f = energy slope.

$$\alpha = 4 pH^{-0.15} (\rho g V S_f)^{-1.4} \quad (5.7.3-5)$$

The relationship between rate of ageing and input stream power is shown in Figure 5.7.3-6. The rate of ageing is lowest at high input stream power and at high pH values. Equation 5.7.3-5 includes the impact of input stream power which has generally been found to describe flow and sediment transport characteristics with great accuracy. A linear regression carried out to calibrate equation 5.7.3-5 had a low correlation coefficient of only 0.49. The above equation has therefore been calibrated as envelope curve (conservative high rates of ageing) on only seven data sets, including the Orange-Fish Tunnel data, and should be used with caution.

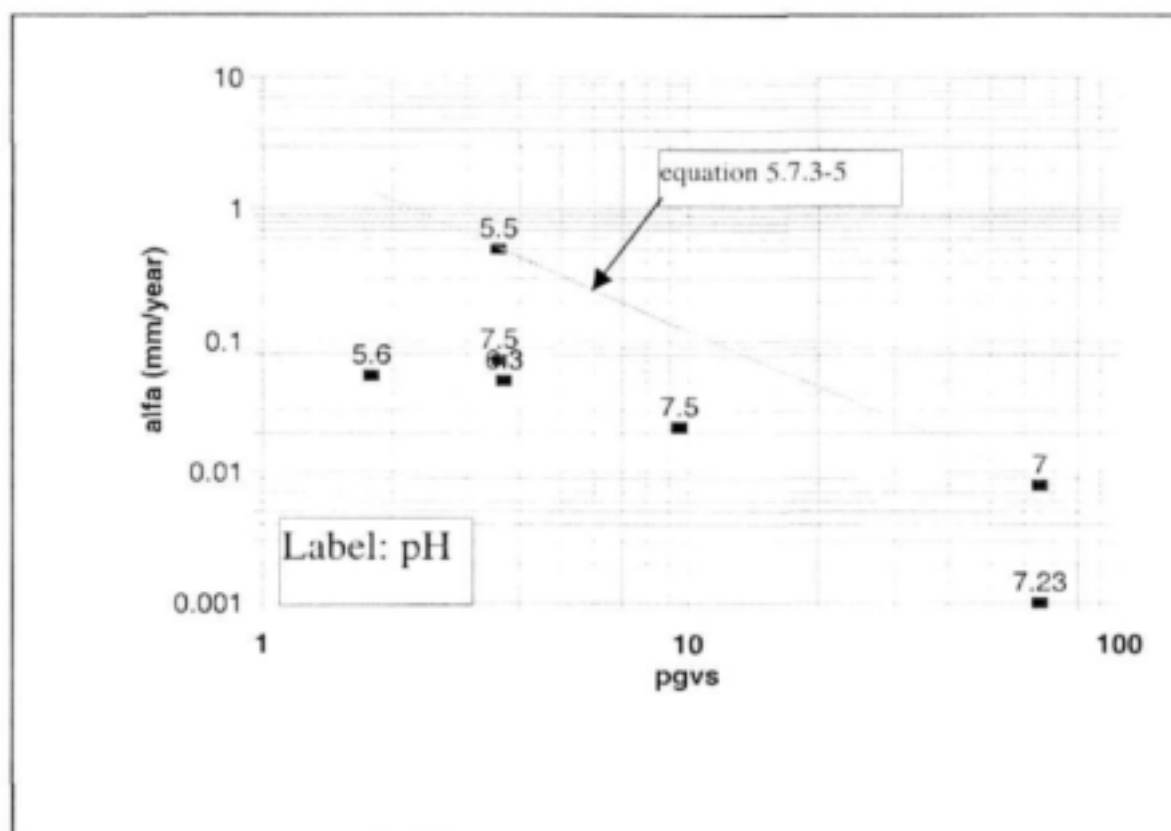


Figure 5.7.3-6 Relationship between input stream power, pH and rate of ageing

The data used to calibrate the above proposed equation include pH values between 5.5 and 7.5. When ageing field observations obtained at the Orange-Fish Tunnel is used to forecast possible future conditions at the LHWP tunnels, it is important firstly to have corresponding pH values. Orange-Fish Tunnel pH values ranged between 7.5 to 8.5 in 1975 at the inlet, while more recent DWAF data indicate a median pH of 7.5, with a range 5.9 to 8.7 at the tunnel outlet. These values are quite similar to pH values obtained at Katse Dam before tunnel commissioning, with mean of 8.1 and range 7.0 to 8.8 (based on a small data set of only 23 samples).

Both the equations proposed by Colebrook (1939) and Basson (1999) showed relatively poor correlations with the data. This might be attributed to variability in data such as pH, or the selection of velocities not representative of typical values experienced in the tunnels and/or the selection of unsuitable theory. One of the problems with equations 5.7.3-4 and 5.7.3-5 is that they are not dimensionless, which could make predictions outside the calibration range highly unreliable.

Applied stream power is a maximum at the bed (tunnel wall) and should be able to describe hydraulic roughness where sediment deposition occurred, perhaps better than input stream power.

c) Applied stream power and rate of ageing

In section 5.3.3 a new equation was proposed relating applied stream power at the bed to the power required to maintain suspension. This resulted in a dimensionless equation 5.3.3-5:

$$\frac{D}{\lambda \kappa_s} = \frac{\rho_s - \rho}{30 \rho} \frac{w}{\sqrt{g S_f D}} S_f \quad (5.3.3-5)$$

If for tunnel flow D is replaced with R , κ is assumed = 0.4 for clear water, ρ = constant for water transfer tunnels, w = settling velocity = constant, it is possible to calibrate equation 5.3.3-5 to determine the rate of ageing, with k_s replaced by α . The settling velocity had to be taken as constant since no data were available of the tunnels used by Colebrook. Based on the description of the nature of the sediment deposition it is similar to the South African conditions, consisting of clay and silt fractions.

The data used for the calibration of the proposed ageing equation were obtained from Colebrook (1958) and aged Orange-Fish Tunnel data, as indicated in Table 5.7.3-1. Tunnel ageing data are extremely limited and are rarely published. When hydraulic tests are carried out, these are often treated as confidential consultants' reports internationally. At many tunnels, as long as the hydraulic capacity exceeds the water demand, no field work is carried out to establish hydraulic ageing.

Table 5.7.3-1 Rate of ageing calibration data

Tunnel	Equivalent diameter (m)	Age (year)	pH	S_t (m/m)	k_s new (mm)	Flow velocity (m/s)	Aged roughness k_s (mm)	Rate of ageing (mm/year)
Fasnakyle	4.5	4.5	6.5	0.0018	0.3	3.44	0.7	0.09
Mourne	1.68	55	5.6	0.0003	0.3	0.64	3.0	0.05
Apalachia	5.49	10	7.23	0.0017	0.925	3.87	0.62	-0.03
Glenlee	3.35	19	7	0.0021	0.575	3.14	0.73	0.01
Derwent	1.52	25	5.5	0.0005	0.3	0.67	12.5	0.49
Vyrnwy	1.02	17	6.3	0.0005	0.3	0.73	0.85	0.03
Orange-Fish 1988	5.33	13	7.5	0.0001	0.13	0.67	1.1	0.07
Orange-Fish 1998	5.33	23	7.5	0.0006	0.13	2.24	0.6	0.02

The Apalachia Tunnel is operated at a high flow velocity which would limit excessive build up of deposited sediment. The hydraulic roughness has however decreased with time, giving a negative rate of ageing, which can probably be attributed to the formation of a smooth thin layer of sediment. The Apalachia Tunnel data were not used because of the negative ageing impact.

The tunnel ages varied between 4 to 55 years, while the pH values varied between 5.5 and 7.5. In some cases the hydraulic gradient was very steep as much as 0.0021, while at the Orange-Fish Tunnel during the first 13 years of operation at low flow velocities, the slope was only 0.0001. The roughness of new tunnels have decreased over the years as construction techniques improved. The UK tunnels had minimum roughnesses of 0.3 mm, while the Orange-Fish Tunnel had an original roughness of 0.13 mm. The recently constructed LHP Transfer Tunnel had a roughness of only 0.03 mm which is extremely good for a cast in situ lining.

Often during the operation of a water transfer tunnel the water demand increases over time which results in higher flow velocities, which again limits sediment deposition. This has been experienced at the Orange-Fish Tunnel. During the 1998 tests the tunnel was flushed by running it for a day at maximum capacity which resulted in a considerable decrease in hydraulic roughness. The averaged rate of ageing over 23 years is therefore much less than the rate over the first 13 years. The question has to be asked therefore is the rate of ageing linear with time and if this is assumed, what

could be the consequences in terms of the design and future operation of tunnels? This aspect will be discussed in more detail at the end of this section.

Typically the rate of ageing is less than 0.1 mm/year, but at the Derwent Tunnel the ageing rate observed was extremely high at 0.49 mm/year, with a final roughness k_s after 25 years of operation of 12.5 mm. The actual rate of ageing of this tunnel might be less since the new tunnel roughness had to be estimated at 0.3 mm.

Since the rate of ageing is also influenced by the type of lining, the initial roughness-hydraulic radius ratio was also incorporated in the calibration. The final calibrated ageing equation is:

$$\frac{\alpha}{R} = 75.96(V^*)^{-2.69} S_f^{1.23} (pH)^{-11.55} \left(\frac{R}{k_{s,New}}\right)^{1.21} \quad (5.7.3-6)$$

The correlation coefficient of equation 5.7.3-6 is relatively high at 0.79, compared with the methods proposed earlier by Basson (1999) and Colebrook (1958). The rate of ageing is minimized when S increases, when R decreases, and when pH increases. The rate of ageing is especially sensitive to the pH of the water. The predicted versus observed rate of ageing-hydraulic radius ratio values are indicated in Figure 5.7.3-7.

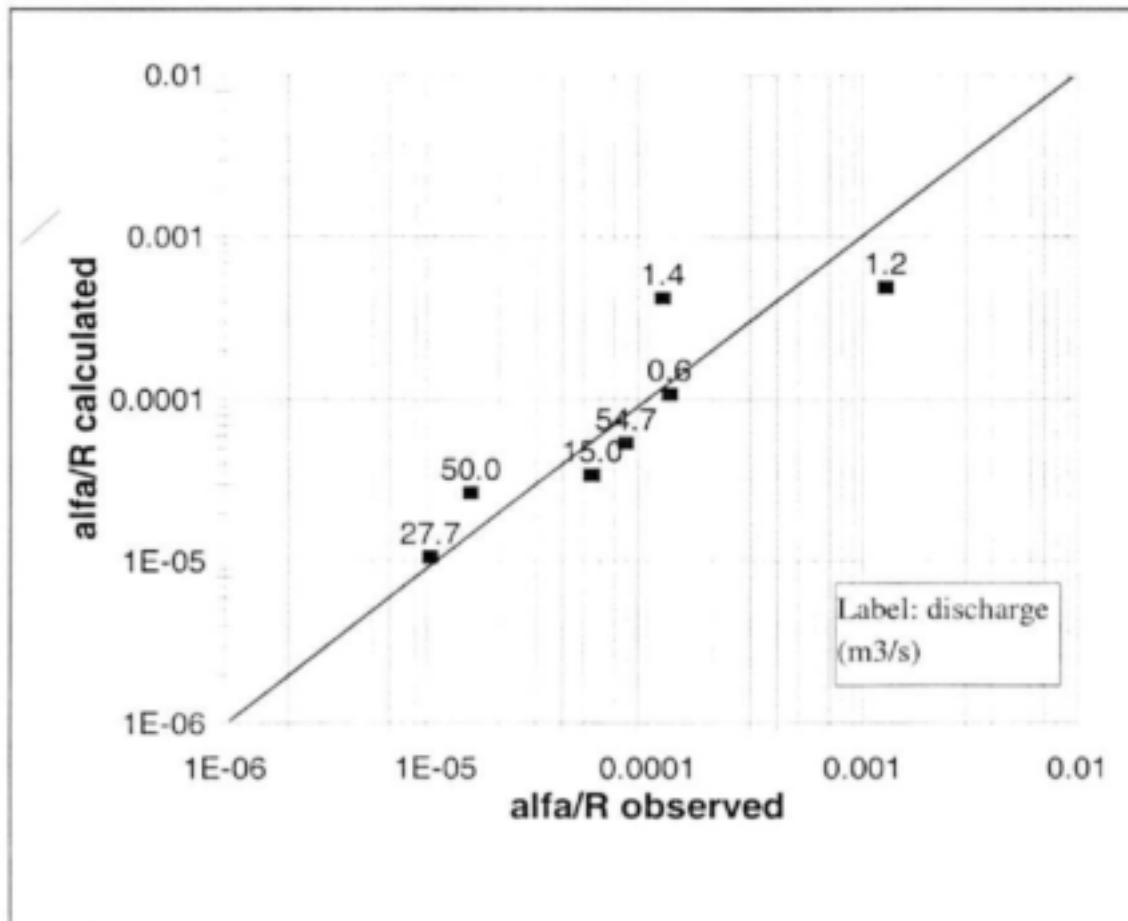


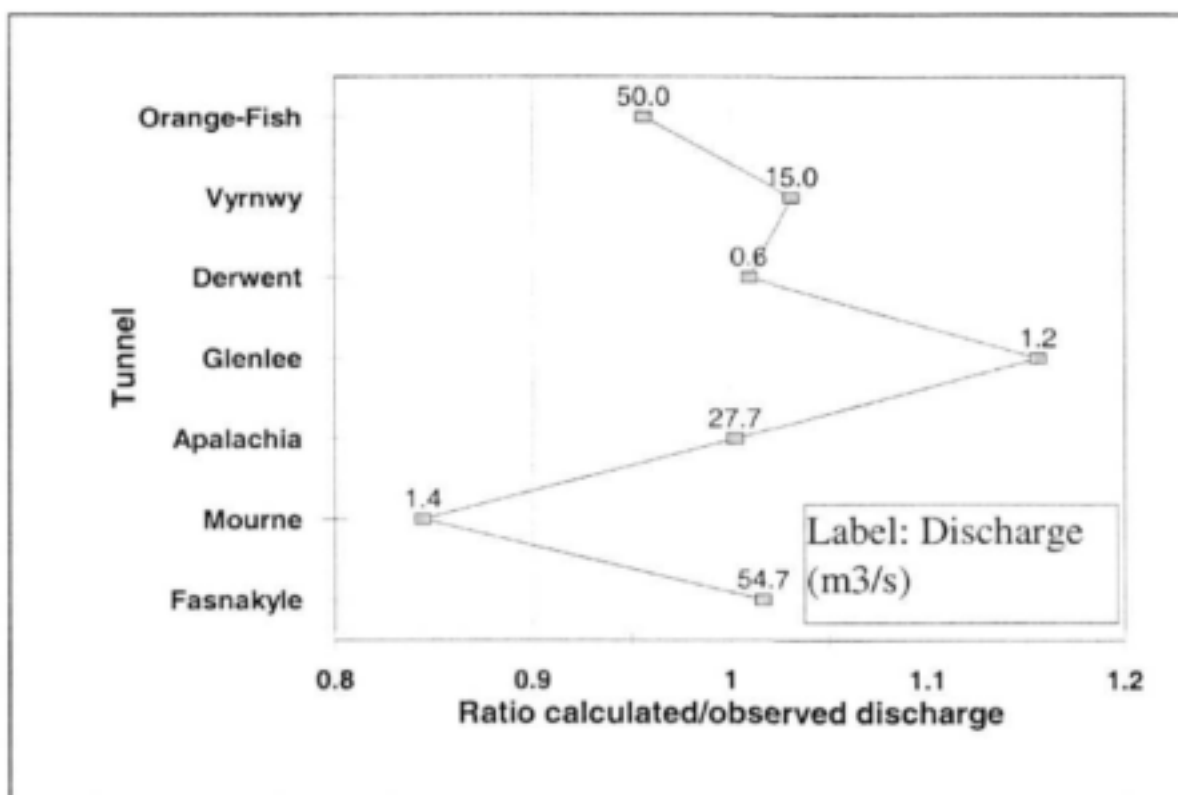
Figure 5.7.3-7 Observed versus predicted rate of ageing-hydraulic radius ratio values

Predicted aged k_s values calculated with equation 5.7.3-6 are shown in Table 5.7.3-2 and are compared with the observed values. In addition discharge calculated with the predicted and observed roughness is also given in Table 5.7.3-2.

Table 5.7.3-2 Predicted and observed roughness and flow

Tunnel	Observed old k_s (mm)	Predicted old k_s (mm)	Observed discharge (m^3/s)	Predicted discharge (m^3/s)
Fasnakyle	0.7	0.57	54.7	55.6
Mourne	3.0	9.99	1.41	1.2
Glenlee	0.73	0.74	27.7	27.8
Derwent	12.5	4.95	1.2	1.4
Vyrnwy	0.85	0.77	0.6	0.6
Orange-Fish 1988	1.1	0.74	15.0	15.5
Orange-Fish 1998	0.6	0.96	50.0	47.8

Figure 5.7.3-8 shows the observed and predicted discharges of Table 5.7.3-2 graphically. From Table 5.7.3-2 it is clear that equation 5.7.3-6 over and underestimates the large roughnesses significantly. However, when the Colebrook-White-Darcy-Weissbach equation is used to calculate discharge from the predicted k_s values, the differences in discharge predicted and observed are relatively small. In fact all the data fall within the range of 0.84 to 1.19 when the ratio of predicted discharge to observed discharge is considered, which is extremely good.

**Figure 5.7.3-8 Predicted and observed discharges based on ageing equation 5.7.3-6**

d) Ageing equation application range

Equation 5.7.3-6 is applicable for a wide range of tunnel diameters (1.0 to 5.3 m), hydraulic gradients and operating velocities. The age of the tunnels used for calibration varied between 5 and 55 years, with an average age of 23 years, and it is proposed that it should only be used to make ageing predictions up to 30 years into the future.

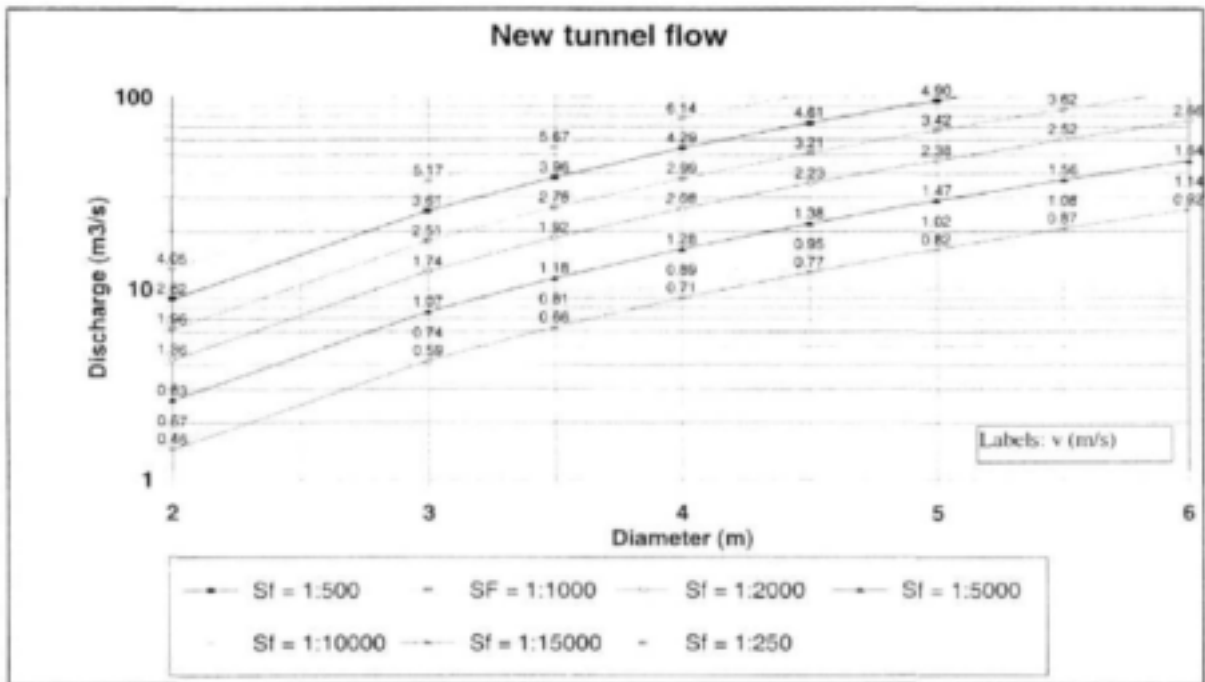
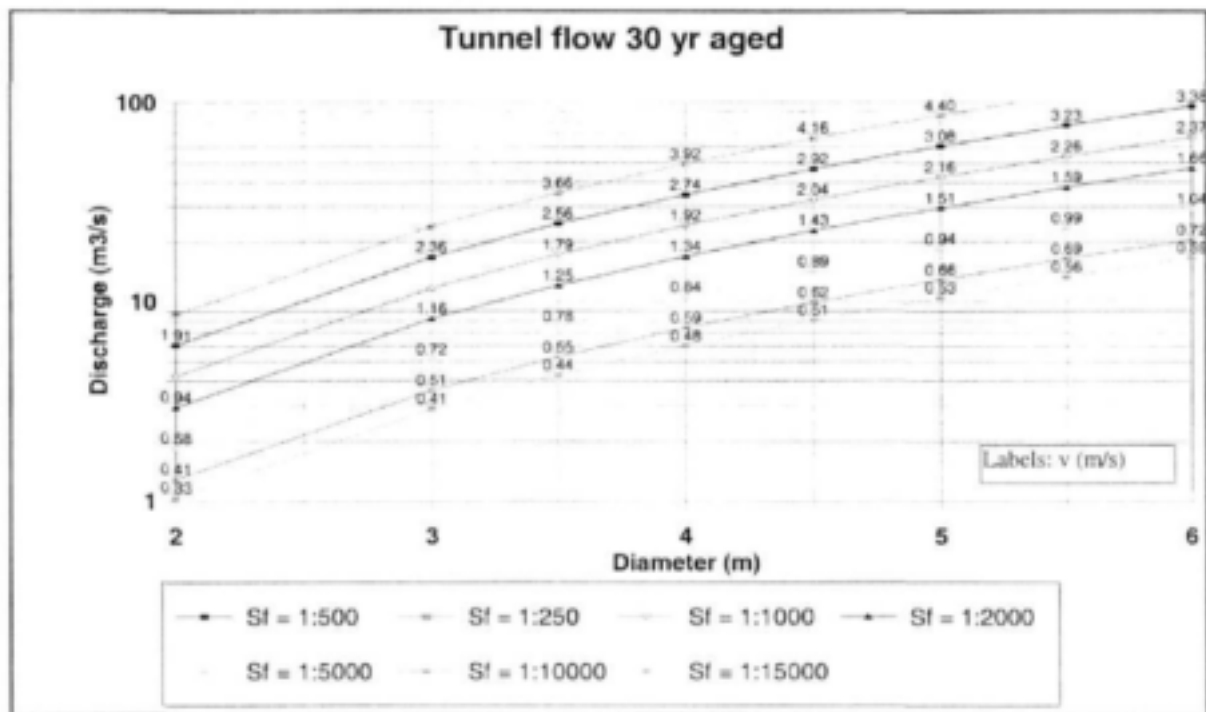
e) Typical tunnel ageing design

The calibrated equation 5.7.3-6 can now be used to calculate typical rates of ageing and flows, for a range of energy slopes and tunnel diameters, as is graphically shown in Figure 5.7.3-10 for a 30 year aged condition, with $k_0 = 0.03$ mm and $\text{pH} = 7.5$. This aged condition can be compared with the new tunnel discharge capacity shown in Figure 5.7.3-9.

In a typical design of a tunnel, the hydraulic gradient S_f is fixed, discharge is often fixed based on the firm yield of the water resources system (or demand is fixed), the water pH is known and for an in situ concrete lined tunnel an initial roughness $k_0 = 0.03$ mm can be assumed. For a pH of 7.5, discharge = $30 \text{ m}^3/\text{s}$ and say $S = 1:1000$ one can use Figure 5.7.3-10 to determine the required tunnel diameter: about 4.4 m. For a new tunnel the required diameter would be 3.7 m to convey this flow.

The ageing is very sensitive to the pH as can be seen from Figure 5.7.3-11 when compared with Figure 5.7.3-10, plotted for pH values of 5.5 and 7.5 respectively, which is also the calibration range. If the ageing calculation above was based on a lower pH of 5.5, the required tunnel diameter would be much larger at 5.5 m to give the required capacity after 30 years of operation.

It is clear that once a tunnel diameter has been selected for a specific aged condition and discharge, the tunnel should be operated for as long as possible at its maximum capacity every year to limit the rate of ageing. It might be possible to restore some of the tunnel capacity later after an initial period of flow less than the design discharge by flushing operation, but efficiency is not guaranteed depending on the critical conditions to re-entrain deposited sediment from the tunnel wall.

Figure 5.7.3-9 Discharge capacity of new tunnel at $k_o = 0.03$ mmFigure 5.7.3-10 Typical 30 year age flow capacity based on rate of ageing equation and $\text{pH} = 7.5$

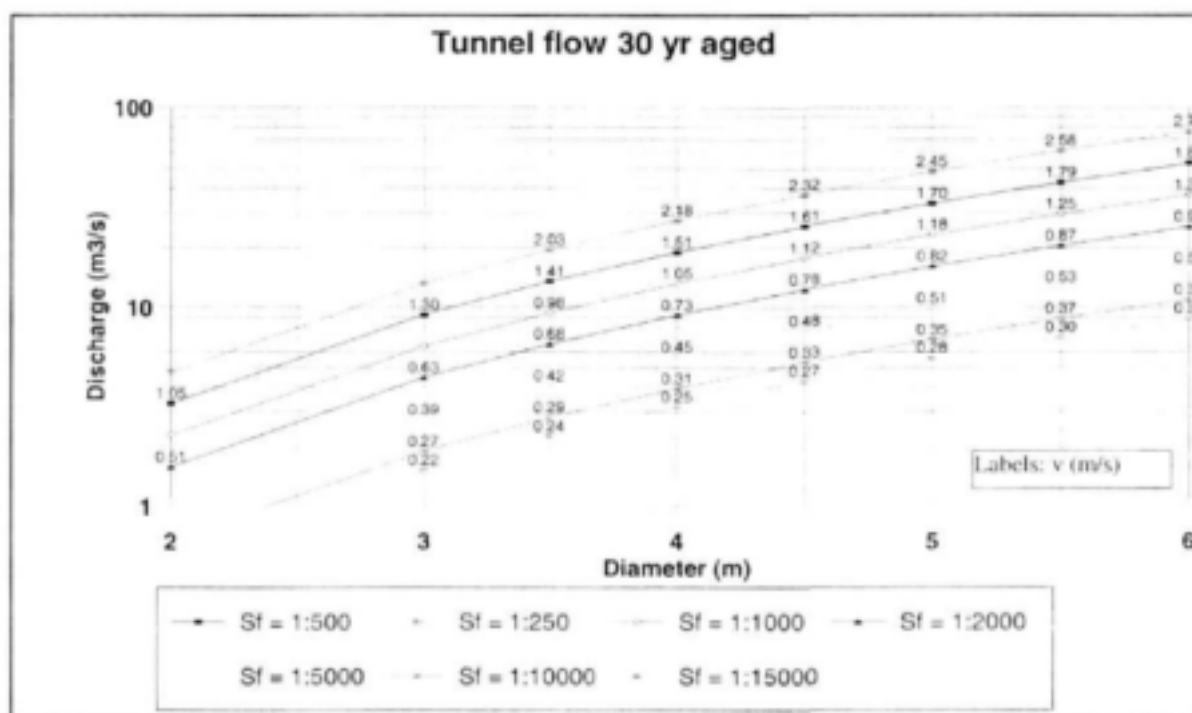


Figure 5.7.3-11 Typical 30 year age flow capacity based on rate of ageing equation and pH = 5.5

f) Linear rate of change in hydraulic roughness assumption

In deriving equation 5.7.3-6, it was assumed that the rate of change in hydraulic roughness occurs linearly with time. It might be possible however that under conditions with an ample supply of sediment being diverted into a new tunnel, bed roughness would reach equilibrium roughness or ultimate roughness for specific flow conditions relatively quickly. This could happen within a year or over a period of 5 to 10 years where after no further increase in hydraulic roughness occurs. So far in this chapter however, ageing (increase in roughness) has been assumed to continue indefinitely and this is probably an incorrect assumption. If the hydraulic roughness is determined by sediment deposition, even if it is a thin layer, the ageing process should be seen as a non-equilibrium phase in the life of a tunnel until the ultimate roughness (equilibrium condition) is reached. The ultimate roughness should remain the same as long as the same boundary conditions apply. If for example the flow velocity is increased in a flushing exercise, the equilibrium roughness should decrease (if the sediment can be re-entrained).

5.8 Ultimate roughness in tunnels / large conduits

5.8.1 Background

Tunnel roughness increases, under steady flow conditions, do not continue indefinitely, but asymptotically approaches an equilibrium long-term ultimate roughness. This equilibrium would not only depend on the tunnel material, but also the water quality, flow velocity, etc.

5.8.2 Pipelines

Several sources can be quoted on final aged roughness values to be assumed for concrete lined tunnels. Many of these are based on experience, with quite often a risk (margin for error) design roughness incorporated. Some typical ultimate roughness values recommended for pipelines are indicated in Table 5.8.2-1.

Table 5.8.2-1 Recommended aged pipeline hydraulic roughness k_s (mm) (HR, 1990)

Classification: Description	Good (mm)	Normal (mm)	Poor (mm)
Old tuberculated water mains with the following degrees of attack: - Slight - Moderate - Appreciable - Severe (Good: up to 20 years use, normal: 40 to 50 years use, Poor: 80 to 100 years use)	0.6 1.5 6.0 15	1.5 3.0 15 30	3.0 6.0 30 60
Slimed gravity sewers Sewers slimed to about half depth; velocity, when flowing half full, approximately 0.75 m/s: - Concrete, spun or vertically cast - Fibre cement - Clayware - UPVC Sewers slimed to about half depth; velocity, when flowing half full, approximately 1.2 m/s: - Concrete, spun or vertically cast - Fibre cement - Clayware - UPVC (The roughness of a slimed sewer varies considerably during one year. The normal value is that roughness which is exceeded for approximately half of the time. The poor value is that which is exceeded, generally on a continuous basis, for one month of the year. The value of k_s should be interpolated for velocities between 0.75 m/s and 1.2 m/s.)	- - - - - - - -	3.0 3.0 1.5 0.6 1.5 0.6 0.3 0.15	6.0 6.0 3.0 1.5 3.0 1.5 0.6 0.3
Sewer rising mains, all materials operated as follows: - Mean velocity 1.0 m/s - Mean velocity 1.5 m/s - Mean velocity 2.0 m/s	0.15 0.06 0.03	0.3 0.15 0.06	0.6 0.3 0.15

Table 5.8.2-1 indicates that a wide range of hydraulic roughness values for water mains are possible and that ultimate roughness increases with older pipes, giving different values for 50 years and up to 100 years use. It is not clear to what extent the values given were calibrated against data, but for tunnels the ultimate hydraulic roughness is expected after say 30 to 40 years, depending on the local hydraulic and water quality conditions. After the ultimate roughness has been reached further sediment deposition could occur which would decrease the flow area. This reduction in area could be of significance in small diameter pipelines, but in tunnels the effect is negligible.

The recommended sewer ageing roughness values indicate that materials with resistance to chemical and biological corrosion associated with sewerage, have much lower hydraulic roughness values than a concrete pipe. The build-up of sediment on adjacent epoxy lined tunnel sections and on concrete sections in the LHWP Delivery and Theewaterskloof Tunnels, were similar and it is doubtful whether different tunnel lining material plays a significant role in ageing as with sewers where the dominant mechanism is sliming.

The recommended roughness data for gravity and rising main sewers in Table 5.8.2-1 indicate that at higher operating flow velocities the ultimate hydraulic roughness decreases. The difference is quite significant. For example, a rising main's poor roughness of 0.6 mm at 1.0 m/s, changes to 0.15 mm at 2.0 m/s. The flushing efficiency improves at higher flow velocities and sediment deposition is limited. Flow velocity on its own is however not a good hydraulic parameter to use when predicting hydraulic roughness and instead a stream power approach should be followed. The aged roughness in sewers flowing full was also found significantly less than with part-full flow, which might be important for the design of aqueducts not flowing full, although the ageing mechanism in sewers is completely different.

5.8.3 Aged tunnel data

Aged tunnel data is shown in Table 5.8.4-1, as obtained from Colebrook (1985) and this study.

5.8.4 Colebrook ultimate roughness theory (1958)

Colebrook (1958) developed a rate of ageing equation based on the pH of the water which he calibrated on UK tunnel data, as discussed in section 5.7. The same data were also used by Colebrook to calibrate an equation for maximum probable ultimate hydraulic roughness due to sliming, in imperial units (equation 5.8.4-1).

$$k_s = 0.40R^{0.28}V^{-2.33} \quad (5.8.4-1)$$

with k_s in ft

Table 5.8.4-1 shows the calculated versus observed ultimate roughness values as determined by the Colebrook equation using the same data his equation was calibrated against, as well as Orange-Fish Tunnel data.

Table 5.8.4-1 Maximum ultimate roughness prediction by Colebrook (1958)

Tunnel	Tunnel age (year)	Observed k_s (mm)	Predicted max k_s (mm)
Fasnakyle	4.5	0.7	0.6
Mourne	55	3.0	23.7
Apalachia	10	0.62	0.5
Glenlee	19	0.73	0.7
Derwent	25	12.5	20.7
Vyrnwy	17	0.85	15.1
Orange-Fish 1988	13	1.1	29.2
Orange-Fish 1998	23	0.6	1.8

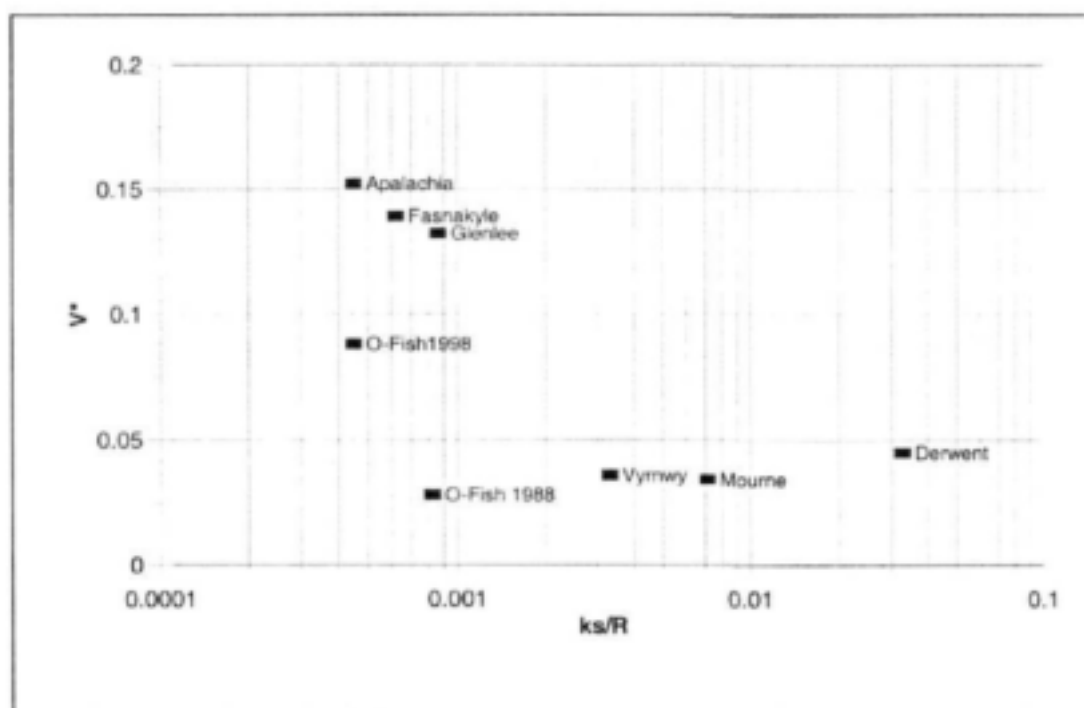
5.8.5 Development of new theory to describe ultimate roughness

The Colebrook equation for ultimate roughness seems to overestimate k_s too much when a tunnel is operated at low flow velocity. There are also some cases where the predicted maximum k_s is slightly less than the observed value. It is proposed that a new equation is derived similar to the rate of ageing equation proposed earlier that is based on conservation of applied stream power, such as equation 5.8.5-1.

$$\frac{R}{k_s} \propto \left(\frac{\rho_s - \rho}{30\rho} \right) \left(\frac{w}{\sqrt{gS_f D}} \right) (S_f) (pH) \quad (5.8.5-1)$$

with $\sqrt{gS_f D} = V^*$ = shear velocity

By assuming that w and density are constant, the equation can be calibrated against tunnel data given in Table 5.8.5-1. The relationship between k_s/R and the variables on the right hand side of equation 5.8.5-2 are shown in Figures 5.8.5-1 to 5.8.5-3.

**Figure 5.8.5-1 Relationship between k_s/R and shear velocity**

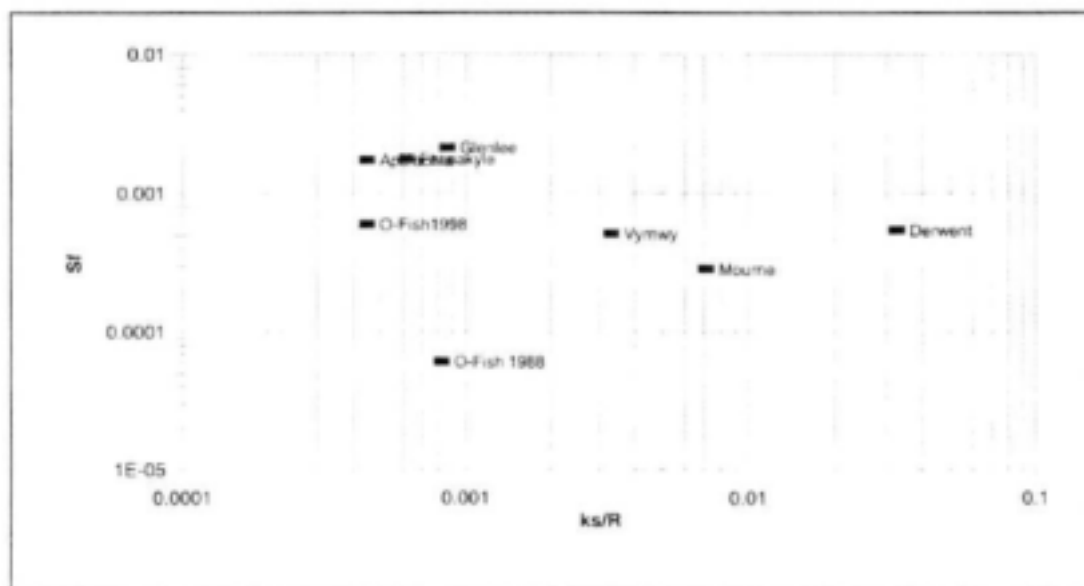


Figure 5.8.5-2 Relationship between k_s/R and energy slope

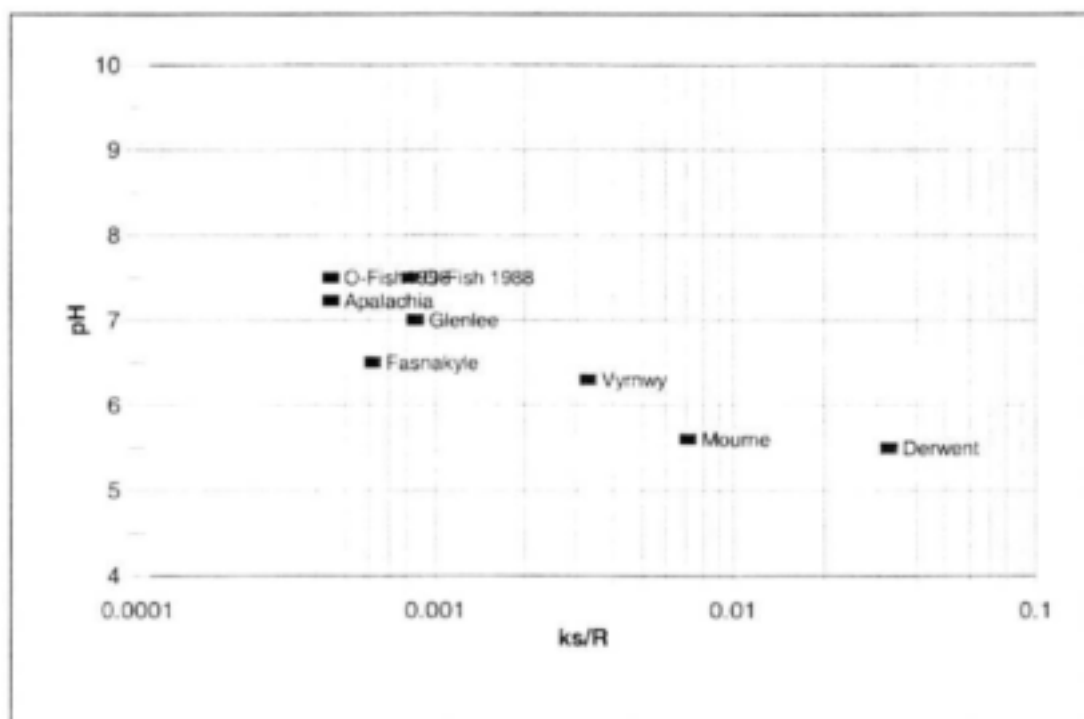


Figure 5.8.5-3 Relationship between k_s/R and pH

The figures show that k_s decreases as V^* increases. When considering S_i , increasing S_i seems to indicate lower k_s , but the relationship is not that clear since the SA data seem to be giving a slightly different relationship than the UK data. The variable pH gives a clear relationship for all the data with high pH resulting in a small ultimate roughness.

The initial roughness/ R to k_s/R relationship was also considered but it was found, as was expected, that it plays no role in determining the ultimate roughness (based on the available data).

Equation 5.8.5-1 was calibrated with the data given in Table 5.8.5-1 and a high correlation coefficient of 0.89 was obtained. Probable ultimate ageing roughness is based on regression analysis, while it is proposed that maximum probable roughness should also be determined (envelope line of data). Maximum probable roughness prediction can be achieved by adjusting the regression constant with a factor of about 2.5. Two equations are therefore proposed to evaluate the probable and maximum probable ultimate ageing of a tunnel, equations 5.8.5-2 and 5.8.5-3 respectively.

Probable ultimate roughness:

$$\frac{k_s}{R} = 17619 (\sqrt{gRS_f})^{-0.972} (S_f)^{0.198} (pH)^{-9.182} \quad (5.8.5-2)$$

Maximum probable ultimate roughness:

$$\frac{k_s}{R} = 44668 (\sqrt{gRS_f})^{-0.972} (S_f)^{0.198} (pH)^{-9.182} \quad (5.8.5-3)$$

Table 5.8.5-1 shows the observed aged roughness and discharge, predicted probable ultimate roughness and associated discharge, and predicted maximum probable ultimate roughness and associated discharge values. The discharge was calculated by using the Colebrook-White-Darcy-Weissbach equation.

Table 5.8.5-1 Ultimate tunnel roughness prediction with new equations

Tunnel	Tunnel age (year)	Observed k_s (mm)	Predicted probable k_s (mm)	Predicted maximum probable k_s (mm)	Observed aged discharge (m ³ /s)	Predicted probable discharge (m ³ /s)	Predicted maximum probable discharge (m ³ /s)
Fasnakyle	4.5	0.70	1.32	3.35	54.7	51.2	46.20
Mourne	55	3.00	5.28	13.38	1.4	1.3	1.14
Apalachia	10	0.62	0.55	1.40	91.5	92.6	84.59
Glenlee	19	0.73	0.54	1.38	27.7	28.6	26.06
Derwent	25	12.50	4.93	12.50	1.22	1.41	1.22
Vyrnwy	17	0.85	1.17	2.97	0.60	0.58	0.52
Orange-Fish 1988	13	1.10	1.02	2.59	15.0	15.0	13.71
Orange-Fish 1998	23	0.60	0.53	1.34	50.0	50.5	46.24

It is possible that the Fasnakyle Tunnel data cover too short a period of ageing and that is why the predicted probable aged condition overestimates the observed values. Due to the small data base this tunnel was however kept in the calibration data base.

The accuracy with which the probable ultimate discharge can be determined is quite remarkable, the ratio of predicted to observed flows falling inside a relatively narrow band between 0.86 and 1.16. This is also shown graphically in Figure 5.8.5-4.

In the past, before the commencement of this study, the ratio (predicted/observed) was based on estimated Manning n values or k_s values with typical prediction observed ratio range of 0.75 to 1.33 (n -values = 0.012 to 0.016).

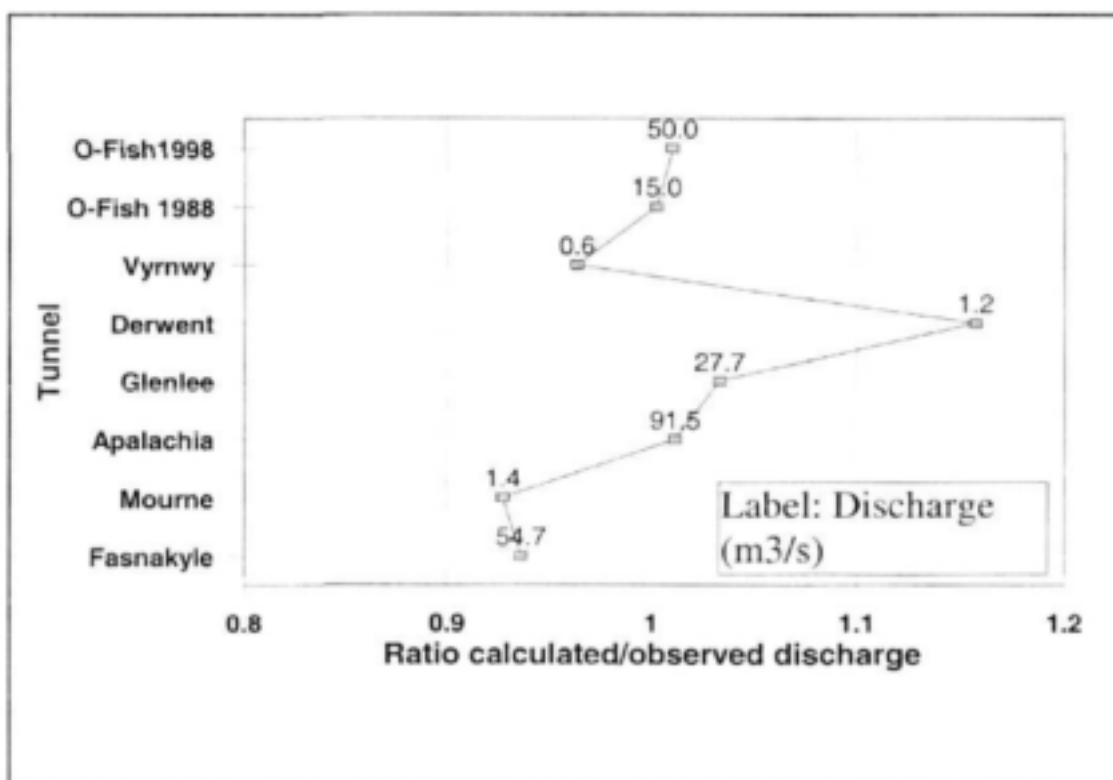


Figure 5.8.5-4 Ultimate roughness tunnel discharge prediction

Ultimate roughness prediction equations have now been derived and calibrated against field data. There however remains one aspect of concern and this is whether the calibration data have in fact reached equilibrium roughness conditions. It is believed that an equilibrium condition is established relatively quickly, if sediment input is sufficient. Evidence of this quick adjustment can be found where tunnels have been cleaned mechanically and where the roughness returns to the pre-cleaning state within a year. The newly calibrated equation that predicts maximum probable ultimate roughness caters to some extent for the possibility that ultimate roughness has not been reached in some of the calibration data.

It is important to note that the ultimate roughness and rate of ageing could be dramatically increased if a tunnel is operated below its design capacity. If in later years the originally predicted discharge capacity is required, it might not be possible to turn the ageing process around.

5.8.6 Verification of the ultimate roughness equation against field data

The Roode Elsberg Tunnel was constructed in the late 1960s and transfers irrigation water from the Roode Elsberg Dam to the Hex River valley (Figure 5.8.6-1). The insitu concrete lined tunnel is 5.2 km in length and 1.98 m in diameter. The tunnel has mostly been operated at flow velocities below 0.5 m/s and a 10 mm uniform layer of fine cohesive sediment deposited on the tunnel wall (Figure 5.8.6-2).

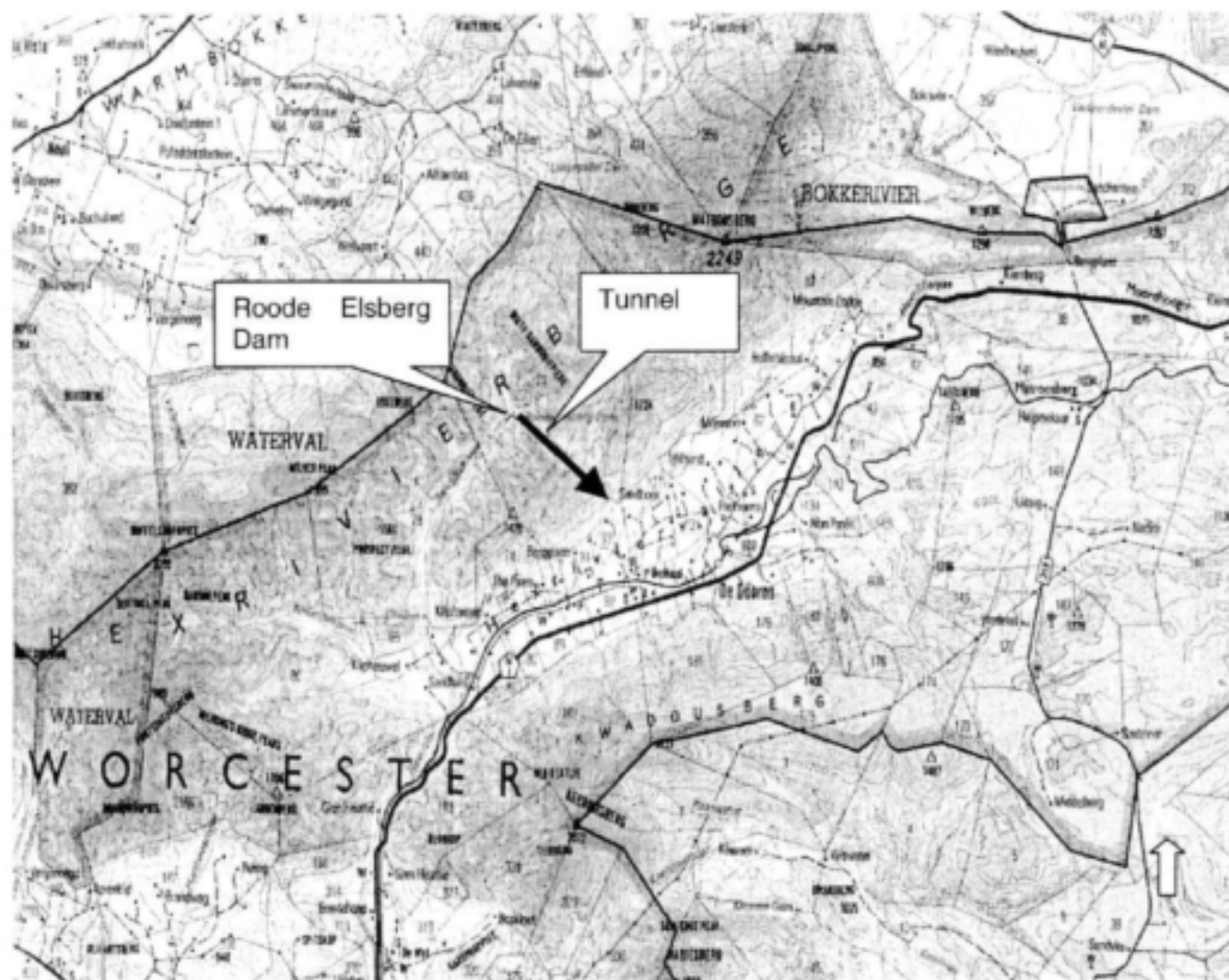


Figure 5.8.6-1 Location plan of Roode Elsberg Tunnel



Figure 5.8.6-2 Sediment deposition in Roode Elsberg Tunnel

Steady flow tests were carried out in 1999 (see full description in Chapter 8) and at the dominant peak discharge of $1.5 \text{ m}^3/\text{s}$ (0.5 m/s), the calculated probable hydraulic roughness is 93 mm (eq. 5.8.5-2). The observed hydraulic roughness is 200 mm. The maximum probable roughness calculated is however 236 mm, at a pH of 5.5 (eq. 5.8.5-3). These equations were calibrated with a maximum roughness data set of 12 mm and the Roode Elsberg Tunnel therefore falls far outside this range. The calculated roughness is however in the right order and certainly indicating a high roughness, as experienced in the field.

5.8.7 Ageing calculation procedure summary

Three new equations have been derived and calibrated with which the following can be determined:

- Rate of ageing (equation 5.7.3-6)
- Probable ultimate roughness (equation 5.8.5-2)
- Maximum probable roughness (equation 5.8.5-3). (Based on envelope line of observed data.)

All three equations should be applied and ageing should continue until the ultimate roughness is reached. Engineering judgement should be used when considering probable or maximum probable roughness, involving a sensitivity assessment and economic evaluation.

Table 5.8.7-1 provides ultimate ageing data which can be used for tunnel design, based on the above equations.

Table 5.8.7-1 Predicted discharge based on probable and maximum probable ultimate roughness

Diameter (m)	S_r (m/m)	Probable ageing Q for pH=5.5 (m ³ /s)	Probable ageing Q for pH=6.5 (m ³ /s)	Probable ageing Q for pH=7.5 (m ³ /s)	Probable ageing Q for pH=7.9 (m ³ /s)	Max probable ageing: Q for pH=5.5 (m ³ /s)	Max probable ageing: Q for pH=6.5 (m ³ /s)	Max probable ageing: Q for pH=7.5 (m ³ /s)	Max probable ageing: Q for pH=7.9 (m ³ /s)
6.0	1:500	96.3	114.4	129.6	134.9	85.2	103.5	118.9	124.5
5.5		77.1	91.7	103.8	108.1	68.2	82.8	95.3	99.7
5.0		60.4	71.9	81.4	84.8	53.4	64.9	74.7	78.2
4.5		46.1	54.9	62.2	64.8	40.7	49.6	57.1	59.8
4.0		34.1	40.8	46.1	47.9	30.1	36.7	42.3	44.2
3.5		24.2	28.9	32.8	34.1	21.3	26.1	30.1	31.5
3.0		16.3	19.5	22.1	23.0	14.4	17.6	20.3	21.2
2.0		5.8	6.9	7.8	8.1	5.1	6.1	7.2	7.5
1.0		1.0	1.2	1.3	1.4	0.8	1.1	1.2	1.3
6.0	1:1000	66.4	79.2	89.9	93.6	58.6	71.5	82.4	86.3
5.5		53.1	63.4	72.0	75.0	46.8	57.2	66.0	69.1
5.0		41.6	49.7	56.5	58.8	36.7	44.8	51.7	54.2
4.5		31.8	38.0	43.1	44.9	28.0	34.2	39.5	41.4
4.0		23.5	28.1	31.9	33.2	20.7	25.3	29.3	30.6
3.5		16.7	20.0	22.7	23.6	14.7	18.0	20.8	21.8
3.0		11.2	13.5	15.3	15.9	9.9	12.1	14.0	14.7
2.0		4.0	4.8	5.4	5.6	3.5	4.3	5.0	5.2
1.0		0.7	0.8	0.9	0.9	0.6	0.7	0.8	0.9
6.0	1:2000	45.8	54.8	62.3	64.9	40.2	49.3	57.0	59.8
5.5		36.6	43.9	49.9	52.0	32.2	39.5	45.7	47.9
5.0		28.7	34.4	39.1	40.7	25.2	30.9	35.8	37.5
4.5		21.9	26.3	29.9	31.1	19.2	23.6	27.4	28.7
4.0		16.2	19.4	22.1	23.0	14.2	17.5	20.2	21.2
3.5		11.5	13.8	15.7	16.4	10.0	12.4	14.4	15.1
3.0		7.7	9.3	10.6	11.0	6.8	8.4	9.7	10.2
2.0		2.7	3.3	3.7	3.9	2.4	3.0	3.4	3.6
1.0		0.5	0.6	0.6	0.6	0.4	0.4	0.6	0.6
6.0	1:5000	27.9	33.7	38.4	40.0	24.5	30.2	35.1	36.8
5.5		22.5	26.9	30.7	32.0	19.5	24.2	28.1	29.4
5.0		17.5	21.1	24.1	25.1	15.3	18.9	22.0	23.1
4.5		13.4	16.1	18.4	19.1	11.7	14.5	16.8	17.6
4.0		9.9	11.9	13.6	14.2	8.6	10.7	12.4	13.0
3.5		7.0	8.5	9.7	10.0	6.1	7.6	8.8	9.3
3.0		4.7	5.7	6.5	6.8	4.1	5.1	6.0	6.2
2.0		1.7	2.0	2.3	2.4	1.4	1.8	2.1	2.2
1.0		0.3	0.3	0.4	0.4	0.2	0.3	0.4	0.4

6 SOFTWATER AGGRESSIVENESS INDEX

6.1 Ageing by soft water (chemical) corrosion

While the relatively high pH values and flow velocities in the LHWP tunnels indicate a relatively slow rate of tunnel ageing, soft water corrosion (leaching) of the concrete surface could be the dominant ageing factor to consider. The Katse water could be potentially aggressive to concrete with Langelier index values less than zero (mean = -0.9), and the Ryznar indices well in excess of 6.5 (which demonstrates a "neutral" water, neither scale forming nor scale removing). The aggressiveness index formulated for evaluating corrosion in asbestos cement pipes indicates a value of 11.24, which is moderately aggressive (DWAF, 1996). (See Table 6.1-1).

Table 6.1-1 Aggressiveness index interpretation (DWAF, 1996)

Aggressiveness index	Water property
>12	non-aggressive
10.0 to 11.9	moderately aggressive
<10	highly aggressive

Basson (1989) recommended limits for assessing aggressiveness, as indicated in Table 6.1-2.

Table 6.1-2 Recommended limits for assessing aggressiveness (Basson, 1989)

Property of water	Degree of aggressiveness of water			
	Moderate	High	Very high	Excessive
pH	6.0 to 8.0	5.0 to 6.0	4.5 to 5.0	< 4.5
pH minus CaCO ₃ -saturated pH	-0.2 to -0.3	-0.3 to -0.4	-0.4 to -0.5	< -0.5
Calcium hardness as mg CaCO ₃ /l	200 to 300	100 to 200	50 to 100	< 50
Total ammonium ion as mg NH ₄ /l	30 to 50	50 to 80	80 to 100	>100
Magnesium ion as mg Mg/l	100 to 500	500 to 1000	1000 to 1500	>1500
Total sulphate ion as mg SO ₄ /l	150 to 1000	1000 to 2000	2000 to 3000	>3000
Chloride ion as mg Cl/l	500 to 1000	1000 to 2500	2500 to 5000	>5000

A 4th corrosion index was established by Basson (1990) for the Portland Cement Institute (PCI). Based on the latest water quality data at the Ash River outfall, the PCI index has a value of about 1700, which indicates a highly aggressive condition (Table 6.1-3).

Table 6.1-3 Aggressiveness index interpretation (Basson, 1990)

Final index	Aggressiveness
< 350	non- to mildly aggressive
350 to 750	mildly to fairly aggressive
750 to 1000	highly aggressive
> 1000	very highly aggressive

The above four indices indicate that the LHWP tunnels would be subject to softwater corrosion with resultant leaching of concrete, with possible increased roughness and friction losses in future.

Unfortunately no way of predicting what the degree of deterioration of the concrete lined portions of the tunnels will be possible without field observations over time at the LHWP tunnels. Muller (1977) states that the rate of corrosion is a function of the concrete permeability. Corrosion resistance is increased against natural aggressive waters if the concrete is well cured, sound and dense, having a relatively high cement content and a minimum water-cement ratio.

The rate of tunnel roughening of the LHWP tunnels is however expected to be slow considering the high quality concrete used. Knowledge of two other tunnels in South Africa in the Western Cape: Theewaterskloof and Roode Elsberg which both convey aggressive soft water, indicates that silt deposition and some sliming creating a slippery surface are present, even under softwater corrosion conditions.

Aggressive ground water zones in South Africa are shown in Figure 6.1-1 and provides an indication of where soft water in surface water can be expected. The water quality of surface water at a specific tunnel should however be investigated to obtain site specific data.



Figure 6.1-1 Soft water zones in ground water in South Africa (DWAF, 1999)

7 SOFT WATER CORROSION LABORATORY TESTS

7.1 Introduction

Soft water corrosion in hydraulic structures is defined as the progressive disintegration of the concrete by chemical reaction with the soft water (ACI 210 R-93, 1998). This chapter is concerned with the deterioration of the exposed concrete surfaces of the concrete linings of the Theewaterskloof Tunnel and the Lesotho Highland Water Project (LHWP) delivery tunnels as a result of soft water attack.

The deterioration of the exposed concrete surfaces of the concrete linings of the tunnels will influence the hydraulic roughness of the tunnels and therefore an in-depth look into the corrosion rate and ultimate hydraulic roughness of the concrete linings is essential.

The objective of this chapter is to give an overview of the mechanism of soft water corrosion, a discussion on the laboratory tests done, the conclusions on the rate of corrosion and the roughness change in the water tunnels as well as a recommendation for a concrete mix design which can best resist soft water attack.

7.2 Background

7.2.1 Mechanism of concrete corrosion in soft water

In mountainous regions where the rock formations are igneous or siliceous or dense lime stones into which water cannot penetrate and there is almost complete absence of bases and soluble salts in the surface soils the waters are almost free of dissolved salts (Ballim, 1993). The drainage waters from these regions are often acidic owing to the presence of humic acid and also carbon dioxide, which is a product of plant decay. A saturated solution of humic acid in water has a pH value of 3.6-4.1. Owing to its low solubility, 0.19 g/l, the amount, which can be carried in water, is small and although it has some action on concrete, this is much less serious than that of very pure (mineral-free) waters or of those containing carbon dioxide (CO₂). These pure waters are aggressive to concrete primarily because of their "ion hungry" nature (Lea, 1998).

The dominant raw materials in ordinary Portland cement is: (Lea, 1998)

Notation:

C = CaO

S = SiO₂

H = H₂O

A = Al₂O₃

F = Fe₂O₃

CH = Ca(OH₂)

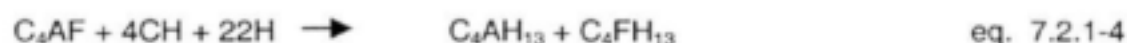
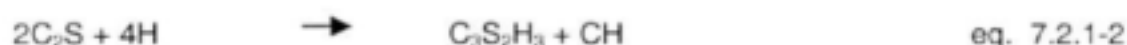
The main compounds in Portland cement can be seen in Table 7.2.1-1

Table 7.2.1-1: The main compounds in Portland cement: (Neville, 1987)

Name of compound	Oxide composition	Abbreviation
Tricalcium silicate	$3\text{CaO} \cdot \text{SiO}_2$	C_3S
Dicalcium silicate	$2\text{CaO} \cdot \text{SiO}_2$	C_2S
Tricalcium aluminate	$3\text{CaO} \cdot \text{Al}_2\text{O}_3$	C_3A
Tetracalcium aluminoferrite	$4\text{CaO} \cdot \text{Al}_2\text{O}_3 \cdot \text{Fe}_2\text{O}_3$	C_4AF

The product of hydration of C_3S is the microcrystalline hydrate $\text{C}_3\text{S}_2\text{H}_3$ with some lime separating out as crystalline $\text{Ca}(\text{OH})_2$; C_2S behaves similarly but clearly contains less lime. Nowadays, the calcium silicate hydrates are described as C-S-H – gel.

The approximate chemical reactions taking place during hydration of ordinary Portland cement is as follows: (Addis, 1989)



The silicates, C_3S and C_2S , are the most important compounds, which are responsible for the strength of hydrated cement paste. At the age of about one year, the two compounds, mass for mass, contribute approximately equally to the strength of the hydrated cement. C_3A and C_4AF contribute very little to the strength of the hydrated cement paste.

The direction of chemical reactions is always towards establishing equilibrium. In the case of concrete wetted by distilled or very soft water the concentration of calcium compounds in the cement paste as seen above is many orders of magnitude higher than that of the dissolved calcium salts in the soft water which is almost zero. The concentration gradient (the difference in concentrations of calcium compounds between the concrete and water phases) creates a leaching of the calcium hydroxide ($\text{Ca}(\text{OH})_2$) (Basson, 1989).

The hydrated calcium silicates (C-S-H) are stable only in highly alkaline conditions; if calcium hydroxide $\text{Ca}(\text{OH})_2$ is removed, the pH is lowered and decomposition of some more hydrated calcium silicates (C-S-H) takes place with the liberation of more $\text{Ca}(\text{OH})_2$ which in turn is removed by leaching. Under favourable conditions these reactions can continue until most of the hydrated calcium silicates (C-S-H) are decomposed (Muller, 1977).

Any of the hydrated compounds present in the cement can be leached from the cement, leaving a residue made up of $\text{SiO}_2 \cdot x\text{H}_2\text{O}$, $\text{Al}_2\text{O}_3 \cdot y\text{H}_2\text{O}$ and $\text{Fe}_2\text{O}_3 \cdot z\text{H}_2\text{O}$. This residue performs a protective role against leaching on account of the gel-like nature of its compounds (Lea, 1989).

The durability of cement-based materials, subject to the action of mineral-free water, depends essentially on the initial amount of calcium hydroxide ($\text{Ca}(\text{OH})_2$) in the paste (Carde, 1996).

7.2.2 Factors influencing the rate of concrete erosion in mineral-free water

a) The aggressiveness of the soft water

The aggressive action of soft water is increased if it contains carbon dioxide CO_2 in a free state. Initially carbon dioxide CO_2 dissolves in water to form carbonic acid H_2CO_3 . The carbon dioxide CO_2 reacts with calcium hydroxide $\text{Ca}(\text{OH})_2$ to form insoluble calcium carbonate, but on further action the much more soluble calcium hydrogen carbonate $\text{Ca}(\text{HCO}_3)_2$ is formed. This will not dissolve calcium carbonate CaCO_3 but will form more calcium carbonate by reacting with calcium hydroxide, according to the following reactions: (Lea, 1989)



It will be seen that equation 7.2.2-3 is reversible, and can proceed to the right or left depending on conditions. The direction will be to the right for as long as the calcium hydrogen carbonate $\text{Ca}(\text{HCO}_3)_2$ can remain in solution, and for this some free carbon dioxide CO_2 is required. If any of this is lost, the reaction will reverse to the left and calcium carbonate CaCO_3 will be precipitated until sufficient carbon dioxide CO_2 has been released to stabilize the calcium hydrogen carbonate $\text{Ca}(\text{HCO}_3)_2$ remaining in solution (Ballim, 1993).

A certain amount of dissolved CO_2 is required to stabilize the calcium hydrogen carbonate $\text{Ca}(\text{HCO}_3)_2$ and therefore only the CO_2 in excess of this amount will contribute to the erosion of concrete (Lea, 1989).

The hardness of water, as it is applied to concrete erosion, is determined by the temporary hardness, which is a measure of the concentration of the calcium carbonate CaCO_3 . This is different from the concept of total hardness, which measures the concentration of calcium and magnesium cat-ions (Lea, 1989).

The potential aggressiveness of natural water is difficult to assess since it is dependent on a number of interrelated factors. The discussion above shows that in the absence of organic and inorganic acids the aggressiveness of soft water is determined by a combination of the temporary hardness and the concentration of dissolved CO_2 as well as the natural pH of the water. As a guide, water with a temporary hardness of not more than 50 mg/l of CaCO_3 and a pH value not lower than 6,0 is unlikely to cause damage unless its free CO_2 exceeds 50 mg/l. On the other hand soft water, with a low dissolved solids content and a temporary hardness of < 3 mg/l of CaCO_3 can have a marked solvent action even if the free CO_2 content is negligible and the pH value is 7,0 or higher. Table 7.2.2-1 shows a classification of the aggressiveness of the soft waters presented by Lea (Lea, 1989).

Table 7.2.2-1: Swedish classification of natural waters containing carbon dioxide (Lea, 1989).

Class of water	Temporary hardness CaCO ₃ (mg/l)	Aggressive carbon dioxide (mg/l)	Degree of aggressiveness to concrete
I	>35	<15	Very slight
II	>35	15-40	Slight
	3.5-35	<15	Slight
III	>35	40-90	Severe
	3.5-35	15-40	Severe
	<3.5	<15	Severe
IV	>35	>90	Very severe
	3.5-35	>40	Very severe
	<3.5	>15	Very severe

There are other methods such as the Calcium carbonate precipitation/dissolution potential method and the Langelier saturation index, which are currently used to determine whether water is potentially aggressive or not (Ballim,1993).

b) Movement of water relative to the concrete

In the absence of any movement of the water layer in immediate contact with the concrete surface, any erosion action will soon cease because of the depletion of reactants and establishment of equilibrium conditions. Erosion rates proceed much more slowly under stagnant conditions than would be the case if the water was in motion and the interface water layer was to be constantly replenished (Basson, 1989).

The humic acid in some of the mineral-free drainage waters reacts with calcium salts to form calcium humate, which is practically insoluble in water. These insoluble calcium salts tend to block the pores of the concrete surface and prevent further loss of calcium hydroxide from the cement – a protective skin is formed. Flowing aggressive solutions are more damageing. The reaction products are carried away from the concrete surface, thus preventing a protective skin (Lea, 1989).

If the concrete is porous and the soft water can percolate through the material the leaching effect become very serious (Muller,1977).

c) Temperature of the water

Chemical reaction rates are temperature dependant and generally double for every 10-degree rise in the Kelvin temperature. Warm waters will therefore have higher erosion rates than cold water (Basson,1989)

On the other hand the aggressive action of water is increased if it contains carbon dioxide CO₂ in a free state but the solubility of CO₂ in water increases with a decrease in temperature. CO₂-gas at atmospheric pressure is soluble in water with a temperature of 18 °C, to the extent of 1.8 g/l, giving a solution of 1.0577 g/l carbonic acid (Lea, 1989).

d) Cement extender content in the concrete

Replacement of cement with fly ash, blast-furnace slag and silica-fume respectively increase the resistance of the cement-based paste to soft water attack. As mentioned before the durability of cement – based materials, subject to the action of soft water, depends essentially on the initial amount of calcium hydroxide ($\text{Ca}(\text{OH})_2$) in the cement paste. The beneficial effect of the cement extenders is that they contain much less calcium hydroxide ($\text{Ca}(\text{OH})_2$) compared to the ordinary Portland cement. In some cases the calcium hydroxide ($\text{Ca}(\text{OH})_2$) is consumed during the pozzolanic reaction.

In the case of a paste sample made with an OPC cement which leads to about 20% content of $\text{Ca}(\text{OH})_2$, the leaching of this $\text{Ca}(\text{OH})_2$ is the essential parameter governing decrease in strength. The loss of strength due to the decalcification of C-S-H is only 6 % and can be neglected in relation to the loss of strength due to the leaching of $\text{Ca}(\text{OH})_2$. In the case of the use of a paste sample with the admixture of silica fume to consume all the $\text{Ca}(\text{OH})_2$, the decalcification of the C-S-H reduces the strength with about 30 %. The effect of the decalcification of C-S-H is therefore not negligible (Carde, 1996)

Leaching of concretes with cement extenders occurs on the hydrated calcium silicates C-S-H only, causing its decalcification and this is a beneficial factor because this reaction is much slower than the $\text{Ca}(\text{OH})_2$ leaching (Carde, 1999).

e) Permeability and Porosity of concrete

The rate of leaching of calcium hydroxide ($\text{Ca}(\text{OH})_2$) is a function of permeability. It slows down when concrete is compact and strong. Leaching of $\text{Ca}(\text{OH})_2$ increases both the porosity and permeability of the cement paste and this, in turn, causes a decrease in the strength and durability of the concrete (Lea, 1989).

Leaching of massive crystals of the calcium hydroxide ($\text{Ca}(\text{OH})_2$) creates a macro-porosity, which has a greater effect on the loss of the strength than on the porosity increase. The decalcification of C-S-H creates a micro-porosity, which has less influence on the loss of strength than on the porosity increase (Carde, 1996).

f) Other factors

The nature of the aggregate also plays also an important role, since it was found that the depth of corrosion of a paste made with limestone was three times higher than that made with quartz as aggregate (Lea, 1989).

It has been shown that the degree of deterioration of concrete under attack by aggressive water depends on whether or not the water is replenished, flowing, warm or pressurized, and is worse if the water flows from a wet surface to dry surface (Muller, 1977).

7.2.3 Conclusion

Concrete wetted by distilled or very soft water is corroded because the difference in concentrations of calcium compounds between the concrete and water phases creates leaching of calcium hydroxide ($\text{Ca}(\text{OH})_2$). Decomposition of the cement binder takes place as the calcium hydroxide ($\text{Ca}(\text{OH})_2$) is removed and eventually all the hardened cement can be decomposed.

The main factors influencing the rate of leaching of calcium hydroxide ($\text{Ca}(\text{OH})_2$) in mineral – free water are:

A combination of the CaCO_3 content (temporary hardness) and the concentration of dissolved CO_2 as well as the natural pH of the water which determine the aggressiveness of soft water. The degree of aggressiveness of water with a very high dissolved CO_2 content to concrete is very severe. On the other hand soft water, with a very low CaCO_3 content ($< 3 \text{ mg/l}$) can have a marked solvent action even if the free CO_2 content is negligible and the natural pH value is 7,0 or higher.

Erosion rates proceed much more quickly if the soft water was in motion relative to the concrete surface and the interface layer was to be constantly replenished.

Warm waters will have higher erosion rates than cold waters except in the case where the aggressive action of the water is increased by the carbon dioxide CO_2 because the solubility of CO_2 in water increases with a *decrease* in temperature.

Replacement of cement in the concrete mixture with fly ash, blast-furnace slag and silica-fume respectively increases the resistance of the cement-based paste to soft water attack. The beneficial effect of cement extenders is that they contain much less calcium hydroxide ($\text{Ca}(\text{OH})_2$) compared to the ordinary Portland cement.

The rate of leaching of calcium hydroxide ($\text{Ca}(\text{OH})_2$) is a function of permeability of the concrete. It slows down when concrete has a low permeability.

Limestone used as aggregates in the cement paste is more prone to corrosion than quarts used as aggregates.

7.3 Laboratory tests

7.3.1 Aims

The aim of the laboratory tests was to investigate the rate of concrete deterioration and ultimate hydraulic roughness in the Theewaterskloof Tunnel and the LHWP pre-cast linings (without joint effects) of the Delivery Tunnel due to soft water attack.

Soft water attack on concrete is a very slow process and therefore a pilot study was to be conducted firstly to determine if an accelerated attack on the concrete can be reached with tests in the laboratory.

The literature review indicates that there are two main factors influencing the deterioration of the concrete namely the aggressiveness of the water and the composition of the concrete under attack.

The aim of these tests is to keep the aggressiveness of the water constant by using very soft (de-ionised, distilled) water to determine the mechanism and rate of leaching of the Ca-ions from the different concrete mixtures at different water operating velocities.

With the leaching data and the concrete composition data known, theoretical rates of concrete deterioration depths can be determined for the different concretes at different water operating velocities. The aim with these available data is to correlate it with field data at Theewaterskloof Tunnel and predict the deterioration of the concrete in the future. In terms of hydraulic roughness, calibration with field data is not possible because of sediment deposition in the field.

Originally Roode Elsberg Tunnel was also selected as case study for laboratory testing, but information on the actual concrete mix used was difficult to obtain and the cement

used in the late 1950s to construct the tunnel was quite different from what is currently available.

The aim with the tests on the LHWP is to predict concrete deterioration in the future and to recommend a concrete mixture design, which can resist soft water attack to a higher degree than the mixture at present.

7.3.2 Experimental set up

A 150 mm nominal diameter pipe system was installed at the hydraulic laboratory of the University of Pretoria. (See Figure 7.3.2-1)



Figure 7.3.2-1: Pipe system at the hydraulic laboratory of the University of Pretoria

It consists of a straight length of uPVC pipe of about 15 m, and a 5 m concrete test section. Firstly a pilot concrete test section was cast and thereafter, concrete test sections representing both the Theewaterskloof Tunnel and LHWP delivery tunnel were cast. This pipe system has a maximum pump capacity of about 100 l/s. The test sections were cast in the concrete laboratory of the University of Pretoria and were transported to the hydraulic laboratory where they were installed in the pipeline.

A test section is made up of two halves: top and bottom cast around a steel pipe 150 mm nominal diameter as mould. The Department of Water Affairs and Forestry (DWAF) constructed the mould. The two concrete halves weigh close to 0,5 ton each and they were clamped together and made watertight, while special flanges were used to align and connect the test section with the pump line (see Figure 7.3.2-2).

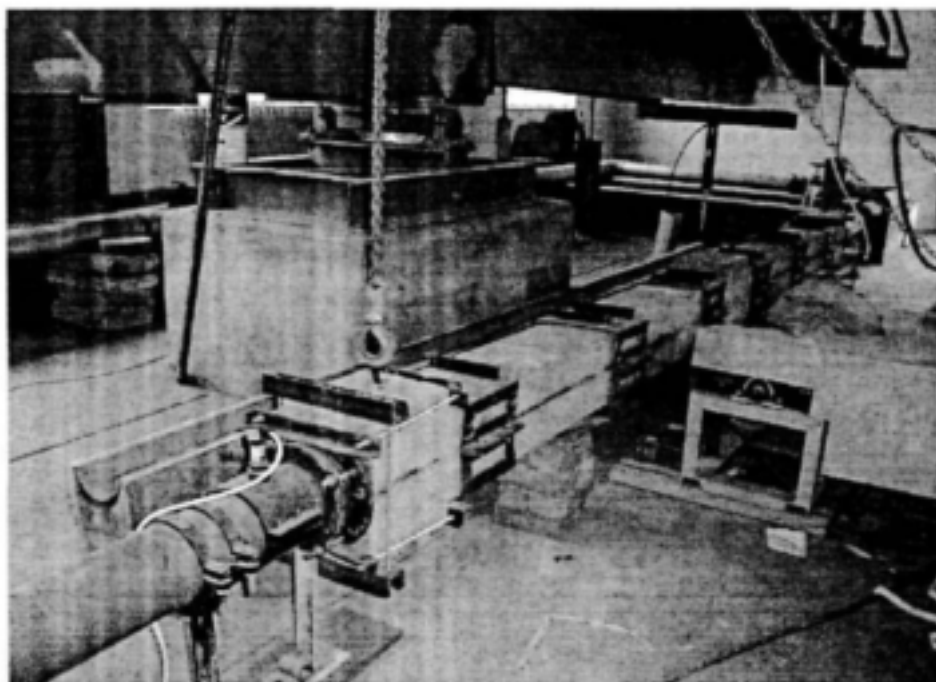


Figure 7.3.2-2: The experimental set up in the laboratory

Two submersible pumps with about 30 ℓ/s discharge capacity each were connected to the 150 mm diameter pipeline.

About 7 k ℓ of de-ionised distilled water was obtained from Spoorinet in Koedoespoort and was transported to the laboratory. The total Ca-ion content of the water was in the order of 4 mg/ ℓ . The water used for these experiments is much more aggressive than the actual water flowing through the two tunnels. Soft water attack is a very slow process and therefore de-ionised distilled water was used to accelerate the chemical attack on the concrete. The water had to be replenished as a result of evaporation from the sump. Special precautions had to be taken to ensure that the mineral-free water does not corrode the pump well by painting it with epoxy paint.

A 90° V-notch was installed at the end of the pipeline to measure discharge. (See Figure 7.3.2-3)

The head loss across the 5 m test section was measured by two electronic pressure gauges, immediately upstream and downstream of the concrete pipe.

7.3.3 Pilot test section

During November 2000 a pilot study was carried out to determine if soft water attack could be simulated in the laboratory. Concrete sections were cast at DWAF and transported to the University of Pretoria.

No mixture content data were available for the concrete used to cast the 150 mm diameter pipe and as no cubes were cast and the 28-day strength was not tested. Four small test cubes had to be cut from the outside of the concrete pipe and put into the water for experimental purposes.

Distilled (ionised) water with a Ca-ion concentration of 0.42 mg/l and a pH of 7 was used as soft water in the pipe system. Appendix A-6 shows the running time of the pump, the velocity as well as the volume of the water. The temperature of the water



Figure 7.3.2-3: 90 ° V-notch at the end of pipe system

was also noted. The volume of water in the system decreased due to evaporation and therefore it was necessary to replenish it with distilled water from time to time (see Appendix A-6).

a) Experimental methods

X-Ray Fluorescence analysis

Two samples – one before leaching of Ca-ions and one after 10 days of leaching of Ca-ions were tested with the ARL 9400XP+ Wavelength dispersive X-Ray Fluorescence Spectrometer. No sample preparation was done, the surface of the material was exposed to the spectrometer and analysed.

Calibration: The wide confidence limit program (QUANTAS) functions by executing scans over the total wavelength span of the spectrometer using different crystal/wavelength combinations. The overlap and background corrected peaks are quantified after application of the NBSGSC program for matrix correction.

Scanner Electron Microscope

A sample was removed for analysis from the water after 15 days of exposure to soft water and the resultant leaching of Ca-ions from the concrete. Fragments of the surface of the sample, as well as a fragment of the inside of the sample were investigated by using the scanner electron microscope. Sample preparation required the fragments to be placed in a vacuum after which it was imbedded in a thin layer of gold.

Atomic absorption analysis

Water samples were taken in the mornings and in the afternoons. An atomic absorption analysis of the Ca-ions in the water was done on selected samples.

b) Experimental results

X-Ray Fluorescence analysis

A quantitative assessment of the compounds on the surface of the samples can be made with this method. In Table 7.3.3-1 it can be seen that approximately two thirds of the Ca has been leached out of the surface of the sample after 10 days in the water.

Table 7.3.3-1: Leaching of Ca-ions from pilot test section

Sample before leaching of Ca-ions		Sample after 10 days leaching of Ca-ions	
Elements	Normalised concentration	Elements	Normalised concentration
SiO ₂	66,2%	SiO ₂	72.8%
CaO	23.7%	CaO	8.84%
Al ₂ O ₃	4.42%	Al ₂ O ₃	5.83%
Fe ₂ O ₃	1.66%	Fe ₂ O ₃	3.18%
SO ₃	1.2%	SO ₃	0.17%
MgO	1.16%	MgO	1.74%
K ₂ O	0,67%	K ₂ O	1.08%
TiO ₂	0.34%	TiO ₂	0.31%
Na ₂ O	0.24%	Na ₂ O	2.57%
SrO	0.20%	SrO	0.195%

Scanner Electron Microscope

Figure 7.3.3-1 to Figure 7.3.3-4 give visible evidence of Ca-ions loss as seen by means of the microscope.

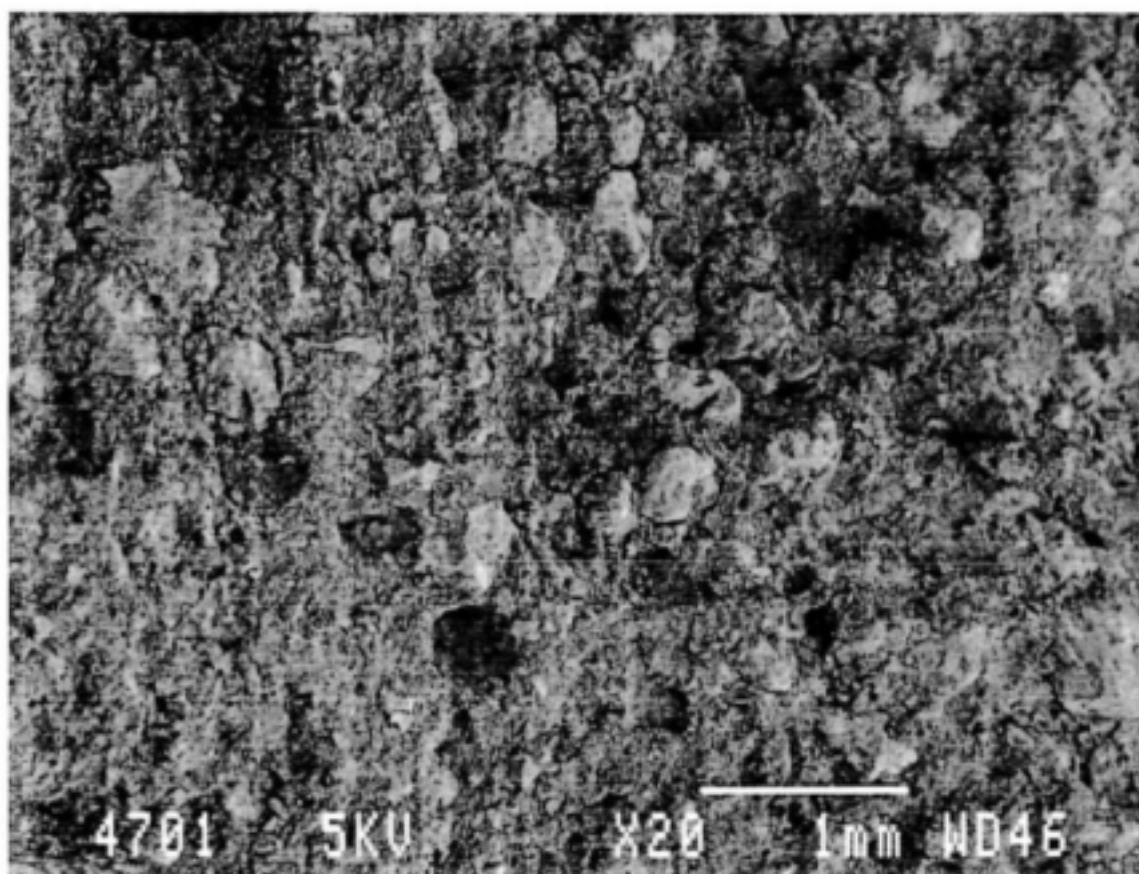


Figure 7.3.3-1: Surface of sample after 10 days leaching of Ca (X20)

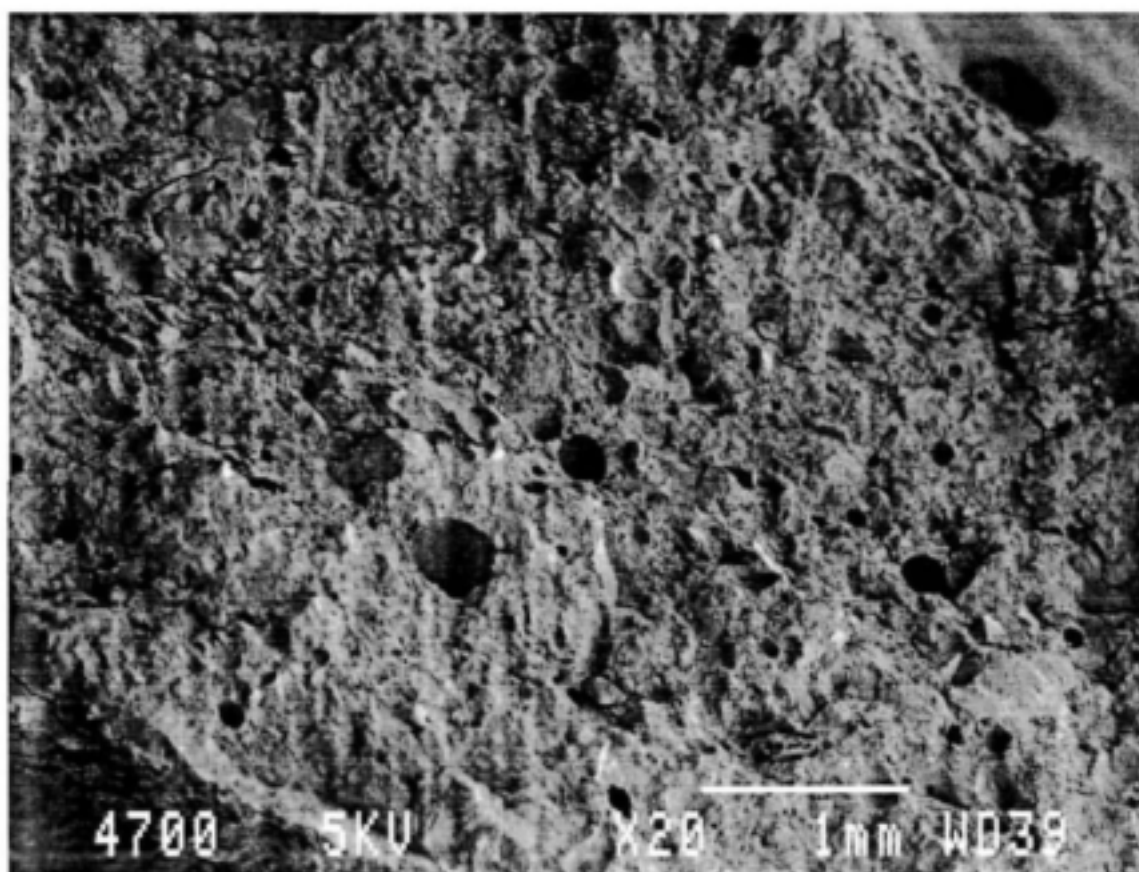


Figure 7.3.3-2: Inside of sample after 10 days leaching of Ca (X20)

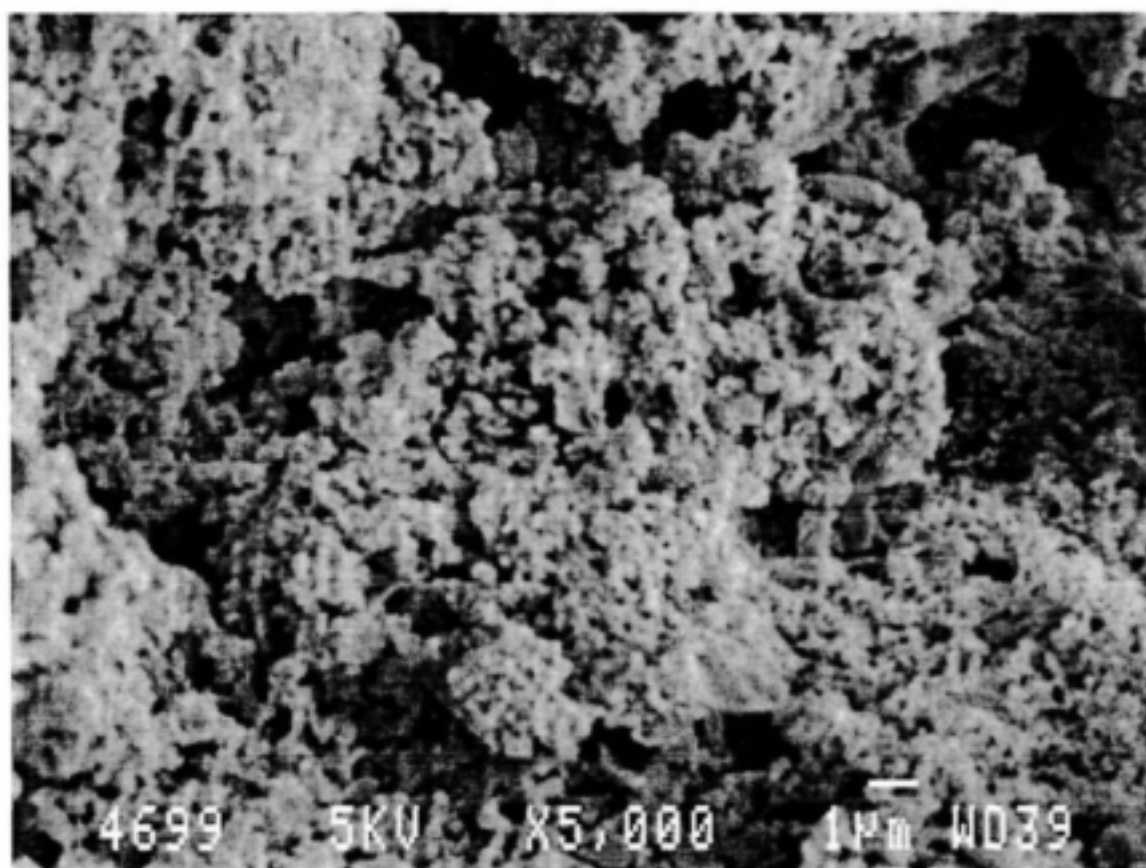


Figure 7.3.3-3: Surface of sample after 10 days leaching of Ca (X5000)

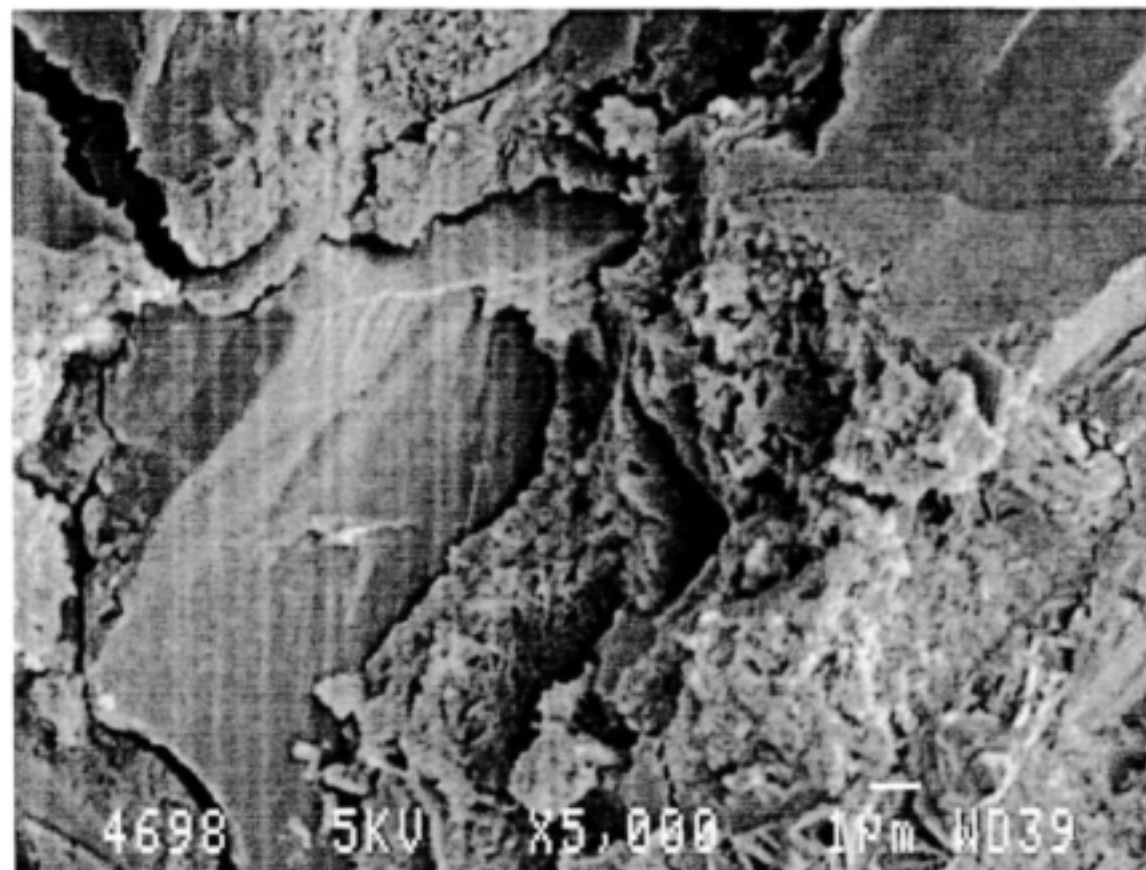


Figure 7.3.3-4: Inside of sample after 10 days leaching of Ca (X5000)

Atomic absorption analysis

Figure 7.3.3-5 shows the rate of Ca-ions (g) being leached from the concrete pipe as determined by the amount of Ca-ions found in the water. In the first 50 hours the leaching rate was high, followed by reduced leaching rates.

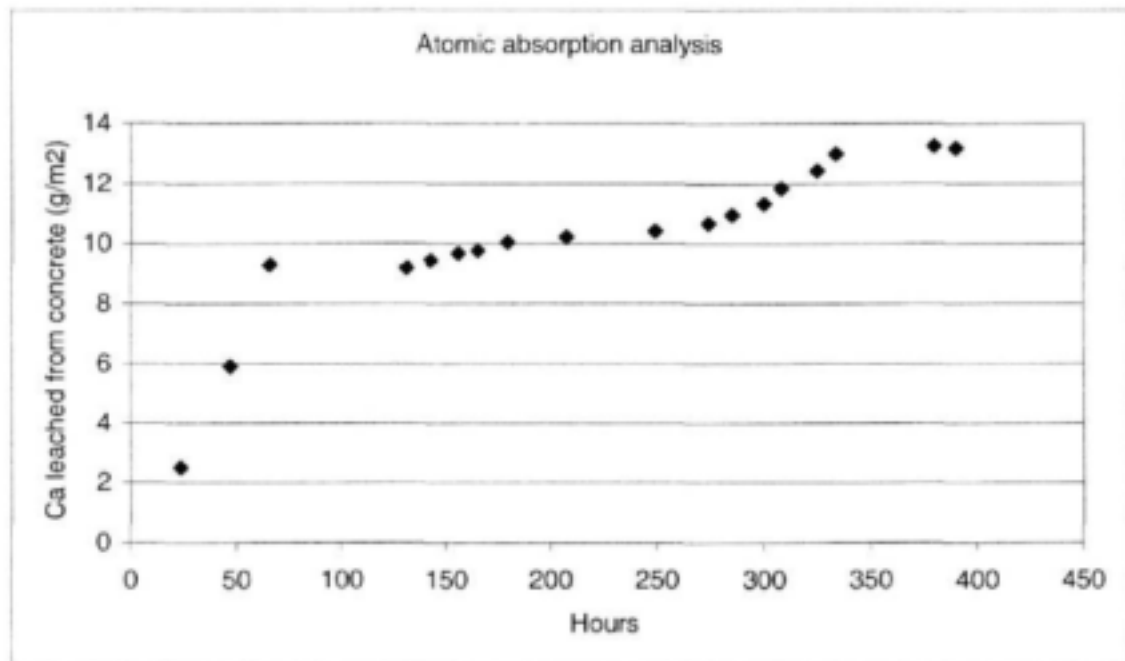


Figure 7.3.3-5 Ca-ions leached from concrete (pilot test section)

From all this data it was found that it is possible to simulate soft water attack on concrete in the laboratory.

7.3.4 Theewaterskloof Tunnel section

a) Materials used in the concrete mixtures for the 5m-test section

The original materials used to cast the Theewaterskloof Tunnel, such as the coarse and fine aggregates have been obtained from the field by DWAF to use in the concrete for the test section.

The cement originally used for the Theewaterskloof Tunnel was produced by De Hoek cement factory in 1980 and would today be classified as a CEM I 32,5. In 1980 it would have contained about 4,5% limestone. Today, the De Hoek cement factory produce cement (not for resale), which is blended with slag for their Surebuild product. This "UBC" as it is called contains 6% limestone (around 4,5% when blended with slag). This UBC cement is the closest to the 1980 cement used in the tunnel. A batch of 300 kg of this cement was obtained from the De Hoek cement factory to use in the concrete mixture for the test section. The chemical composition of this cement is as indicated in Table 7.3.4-1:

Table 7.3.4-1: Chemical composition of cement used for Theewaterskloof Tunnel section

Chemical composition	Percentage %	Mineralogical Composition according to the Bogue calculation	Percentage %
SiO ₂	20,92	Tricalcium silicate (C ₃ S)	63
Al ₂ O ₃	3,53	Dicalcium silicate (C ₂ S)	11
Fe ₂ O ₃	3,69	Tricalcium aluminate (C ₃ A)	3,4
Mn ₂ O ₃	0,06	Tetracalcium Aluminoferrite (C ₄ AF)	11,4
TiO ₂	0,17		
CaO	63,62		
MgO	1,13		
SO ₃	2,34		
K ₂ O	0,67		
Na ₂ O	0,40		

The original concrete mixture used for the Theewaterskloof Tunnel was obtained from DWAF. The composition and properties of the concrete mixture made in the laboratory can be seen in Table 7.3.4-2.

Table 7.3.4-2: Composition and properties of concrete mixture for Theewaterskloof Tunnel section

Composition of mixture	
Water	200,5 kg/m ³
Cement	401,0 kg/m ³
Fine aggregate	729,8 kg/m ³
Coarse aggregate 19 mm-9,5 mm	933,4 kg/m ³
Coarse aggregate 9,5 mm-4,75 mm	49,1 kg/m ³
Properties of mixture	
Design 28-day compressive strength	30 MPa
Coefficient of oxygen permeability (k _{op})	24.57 (m ² * 10 ⁻¹⁸)
Porosity	18,2 %

b) Experimental results

The testing on the Theewaterskloof Tunnel section started at the end of May 2001 and ran 24 h per day until the end of August 2001. Head loss readings, water temperature readings and water samples were taken in the mornings and afternoons. The operating velocities were steady at about 1,2 m/s, 1,9 m/s and 2,9 m/s. The temperature of the

water varied between 24 °C and 33 °C. The natural pH – values of the water varied between 6,7 and 8 with a mean value of 7,4. See Appendix A-1 and A-2 for the data.

c) Observed rate of leaching

The water was analysed for the amount of Ca – ions present in the water by titration with [EDTA] Concentration of Di-sodium Ethylenediamine Tetra-acetate. See Appendix A-2 for leaching data.

The observed rate of leaching of Ca-ions from the concrete for water operating velocities of 1.2, 1.9 and 2.9 m/s can be seen in Figure 7.3.4-1.

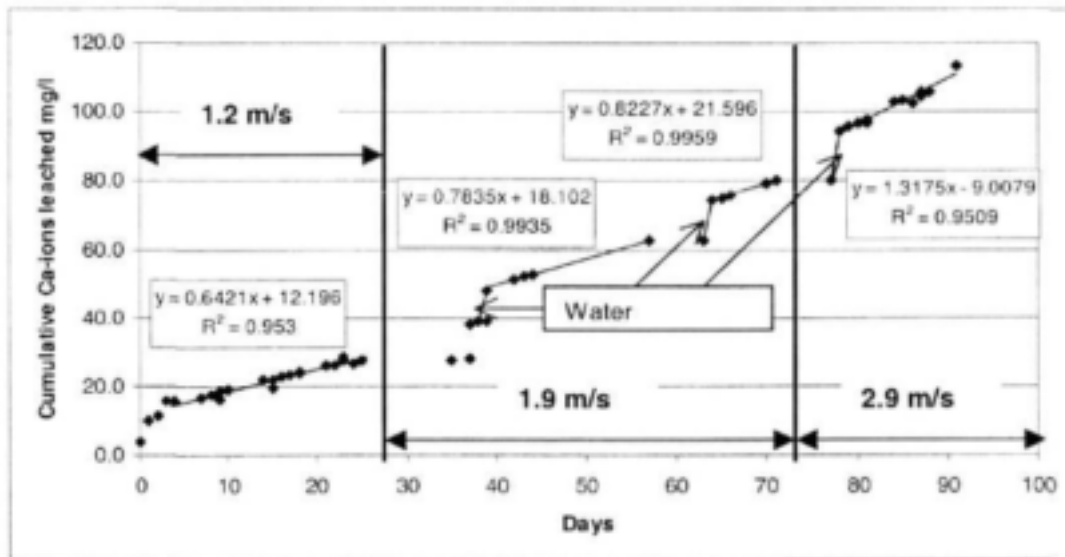


Figure 7.3.4-1 Cumulative leaching of Ca-ions at different operating velocities (Theewaterskloof test section)

It is clear from Figure 7.3.4-1 that during the first three days the calcium hydroxide ($\text{Ca}(\text{OH})_2$) was leached at a much higher rate. It is possible that excess unhydrated cement was present on the surface of the concrete. When the water was replenished as indicated on the graph, the leaching rate of the Ca-ions was very high for a day and then it stabilised at a constant rate for the different water operating velocities.

A linear trendline was added to the graphs and the equation and R-squared value are displayed on the figure. It seems that the leaching rate of Ca-ions for a specific velocity is constant.

The leaching rate of Ca-ions from the concrete for different water velocities can be seen in Figure 7.3.4-2. These leaching rates are for flowing de-ionised distilled water at a mean temperature of 27 °C and a natural pH – value of about 7. An exponential trendline was added to the graph and the equation as well as R-squared value is indicated on the figure. The leaching rate of Ca-ions increases as the operating water velocity increases.

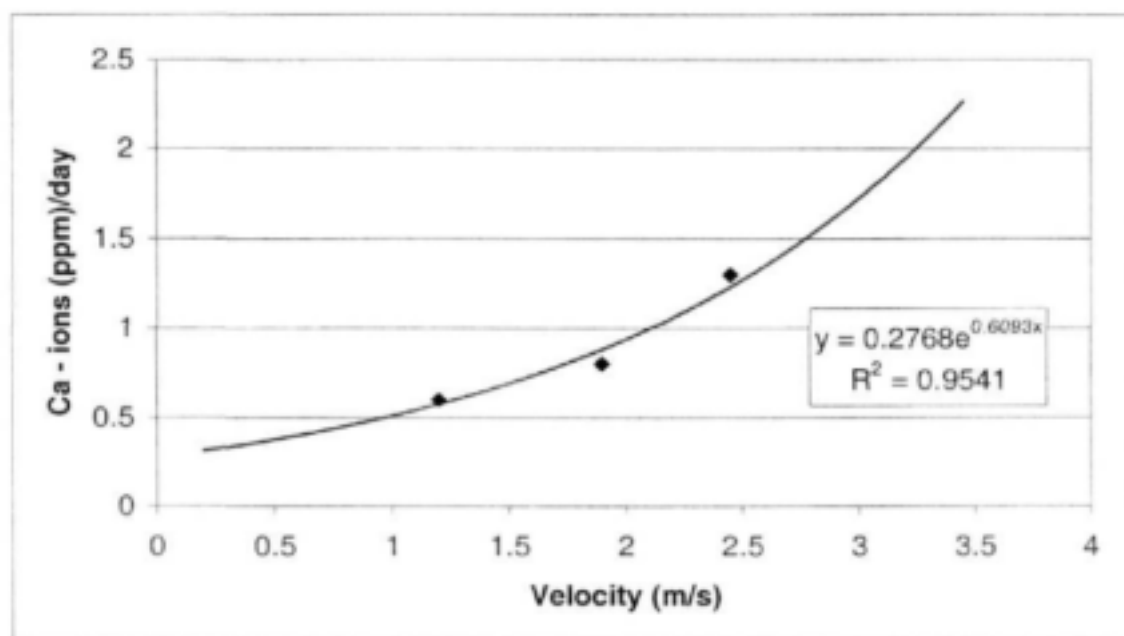


Figure 7.3.4-2 Leaching rate of Ca-ions from the concrete for different velocities (Theewaterskloof test section)

The leaching of Ca-ions from the concrete for a total volume of water through the test section can be seen in Figure 7.3.4-3.

A linear trendline was added to the graphs and the equation and R-squared value are displayed on the figure. It seems that the leaching rate of Ca-ions for a specific volume of water through the test section is constant. The water operating velocities influence this leaching rate of Ca-ions.

The leaching rate of Ca-ions per water volume from the concrete for different water operating velocities can be seen in Figure 7.3.4-4. It seems that the leaching rate of Ca-ions for a constant volume of water decrease for higher operating velocities but more research is necessary.

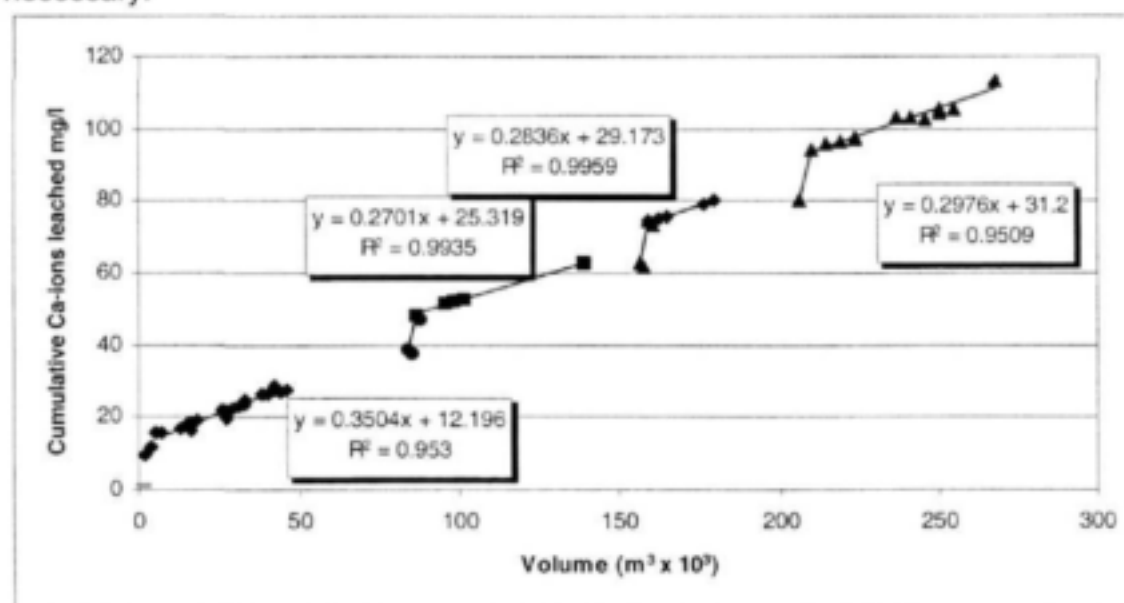


Figure 7.3.4-3 Leaching rate of Ca-ions per total volume of water through Theewaterskloof test section

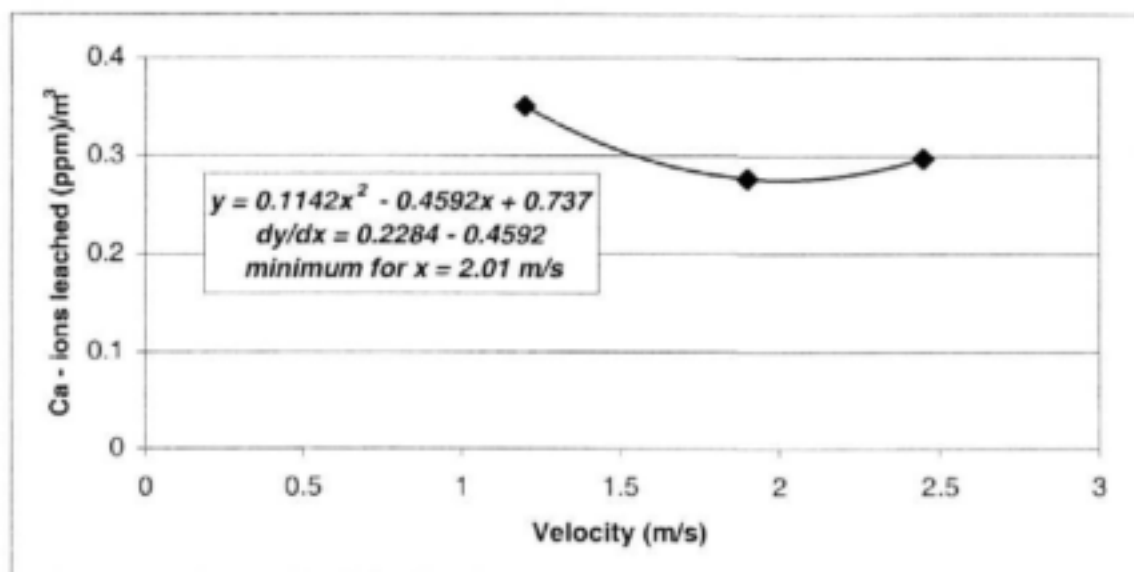


Figure 7.3.4-4 Leaching rate of Ca-ions per water volume for different water velocities (Theewaterskloof test section)

d) Theoretical concrete deterioration

A literature review indicates that it is not only the aggressiveness of the water that influences the leaching rate but also the composition of the cement paste. By predicting the calcium hydroxide ($\text{Ca}(\text{OH})_2$) content of the cement paste a relative chemical vulnerability towards soft water attack can be established (Addis, 1989).

The mineralogical composition of the cement used for the concrete mixture according to the Bogue calculation (see Table 7.3.4-1) can be used to predict the calcium hydroxide ($\text{Ca}(\text{OH})_2$) content of the hydrated paste at an age of 28-days (Addis, 1989).

According to Table 7.3.4-2 the Portland cement used in the Theewaterskloof Tunnel contains 63 % C_3S , 11 % C_2S , 3.4 % C_3A and 11.4 % C_4AF .

See Appendix A-3 for calculations

100 kg of hydrated Portland cement produces:

$$(0,750 \times 63) + (0,994 \times 11) = \mathbf{58,18 \text{ kg } C_3S_2H_3}$$

$$(0,487 \times 63) + (0,215 \times 11) - (0,274 \times 3,4) - (0,609 \times 11,4) = \mathbf{25,17 \text{ kg } Ca(OH)_2}$$

A maximum theoretical depth of deterioration of the concrete can be determined taking into account the following assumptions:

e) Assumptions:

The calcium hydroxide ($\text{Ca}(\text{OH})_2$) content of the hydrated paste at an age of 28-days is 25,17 % (see above) and is very high, therefore according to a literature search (Carde, 1999) the maximum theoretical depth of deterioration of the concrete is calculated for $\text{Ca}(\text{OH})_2$ leaching only.

The $\text{Ca}(\text{OH})_2$ is distributed evenly through the concrete mixture.

The permeability is also a factor influencing the depth of deterioration, but is not taken into account in this calculation.

The maximum theoretical depth of deterioration of the concrete is for de-ionised distilled water with a mean temperature of 27 °C and a natural pH – value of about 7. This water is much more aggressive than the soft water flowing through the tunnel in the field, therefore the maximum theoretical depth of deterioration of the concrete is for an accelerated soft water attack.

Operating water velocity of 1,2 m/s

The Ca-ion leaching rate from Figure 7.3.4-4 is 0,64 g / kℓ

The atomic mass of $\text{Ca(OH)}_2 = 40+16+16+1+1 = 74$

The atomic mass of Ca-ion = 40

Therefore: $\frac{74}{40} = \frac{x}{0,64} \text{ Ca(OH)}_2 / \text{kℓ water}$

$x = 1,18 \text{ g Ca(OH)}_2 / \text{kℓ water}$ is leached per day

The total volume of water is 7 kℓ

Therefore: The total Ca(OH)_2 leached per day is $(1,18 * 7) = 8,29 \text{ g}$

Concrete:

25,17 % of the hydrated cement is Ca(OH)_2 (see above)

401 kg / m³ of the concrete mixture is cement (Table 7.4)

Therefore: $(0,2517 * 401) = 100,25 \text{ kg / m}^3$ is Ca(OH)_2

The volume of the concrete mixture with 8,29 g of Ca(OH)_2

$$\frac{100250 \text{ g}}{1 \text{ m}^3} = \frac{8,29 \text{ g}}{y}$$

$$y = 0,00008269 \text{ m}^3$$

The area of concrete attacked per day is the surface of the inside of the test section:
 $0,15 \text{ m} * \pi * 5 \text{ m}$

The total volume attacked per day is therefore:

$$0,00008269 \text{ m}^3 = 0,15 \text{ m} * \pi * 5 \text{ m} * d_{\text{max}}$$

where: d_{max} = maximum theoretical depth of deterioration of the concrete per day

$$d_{\text{max}} = 35,11 \text{ } \mu\text{m} / \text{day}$$

A minimum theoretical depth of deterioration of the concrete can be determined taking into account the following assumptions:

Assumptions:

The minimum theoretical depth of deterioration of the concrete is calculated by assuming that the excess calcium hydroxide (Ca(OH)_2) in a certain depth of concrete is leached first and then total decalcification of the C-S-H gel occurs.

Operating water velocity of 1,2 m/s

The atomic mass of $\text{Ca}(\text{OH})_2 = 40+16+16+1+1 = 74$

The atomic mass of Ca-ion = 40

Therefore: $\frac{40}{74} = 54\%$ Ca-ions in $\text{Ca}(\text{OH})_2$

The atomic mass of $\text{C}_3\text{S}_2\text{H}_3$ is 342

Therefore: $\frac{3 \cdot 40}{342} = 35\%$ Ca-ions in $\text{C}_3\text{S}_2\text{H}_3$

For 100 kg Cement:

25,17 kg of the cement is $\text{Ca}(\text{OH})_2$

and 58,18 kg of the cement is $\text{C}_3\text{S}_2\text{H}_3$ (see above)

Therefore the total amount of Ca-ions in 100 kg of cement is:

$$(0,54 \cdot 25,17\text{kg}) + (0,35 \cdot 58,18) = 33,956 \text{ kg}$$

Concrete:

But 401 kg / m^3 of the concrete mixture is cement (Table 7.3.4-2)

Therefore: $(0,33956 \cdot 401) = 136,16 \text{ kg} / \text{m}^3$ is Ca-ions

Water:

The Ca-ion leaching rate from Figure 7.3.4-4 is 0,64 g / kℓ

The total volume of water is 7 kℓ

Therefore: The total mass of Ca-ions leached per day is $(0,64 \cdot 7) = 4,48 \text{ g}$

The volume of the concrete mixture producing 4,48 g of Ca-ions is:

$$\frac{13616\text{g}}{1\text{m}^3} = \frac{4,48\text{g}}{y}$$

$$y = 0,00003290\text{m}^3$$

The area of concrete attacked per day is the surface of the inside of the test section:
 $0,15 \text{ m} \cdot \pi \cdot 5 \text{ m}$

The total volume attacked per day is therefore:

$$0,00003290 \text{ m}^3 = 0,15 \text{ m} \cdot \pi \cdot 5 \text{ m} \cdot d_{\min}$$

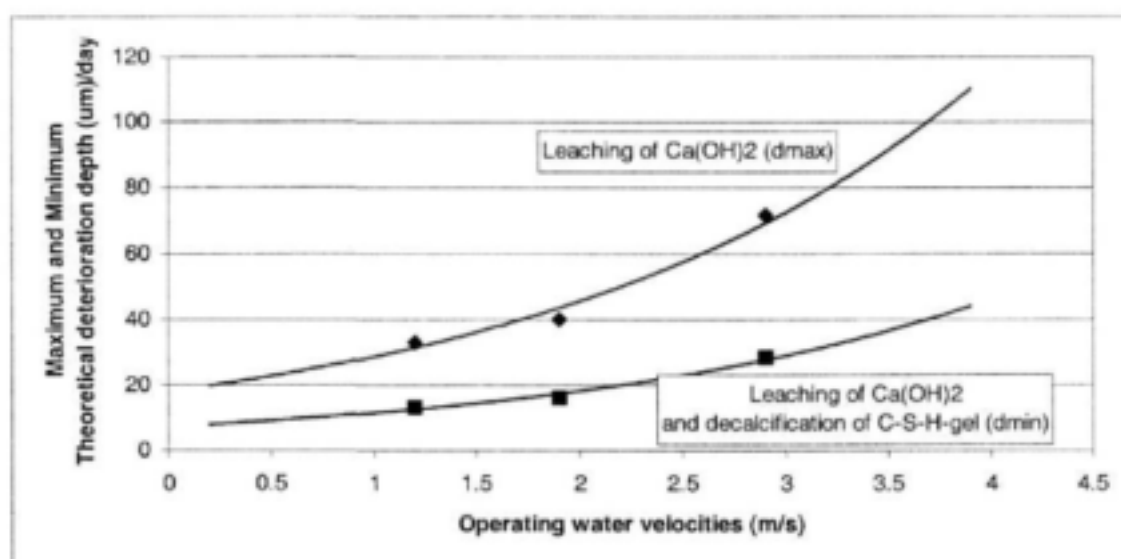
where: $d_{\min}$ = minimum theoretical depth of deterioration of the concrete per day

$$d_{\min} = 13,97 \mu\text{m} / \text{day}$$

Table 7.3.4-3 Theoretical concrete deterioration depths (Theewaterskloof Tunnel section)

Operating water velocity (m/s)	Maximum theoretical depth (μm) of deterioration of the concrete per day (d_{max})	Minimum theoretical depth (μm) of deterioration of the concrete per day (d_{min})
1,2	35,11	13,97
1,9	40,00	16,07
2,9	71,60	28,48

The actual graph indicating the theoretical depth of deterioration of the concrete per day lies between the two graphs on figure 7.3.4-5. The factors influencing the depth are the permeability and porosity of the concrete. The permeability and porosity of the Theewaterskloof Tunnel concrete are very high (see Table 7.3.4-2), therefore it is possible that the actual graph will be closer to the d_{max} -graph than the d_{min} -graph.

**Figure 7.3.4-5 Theoretical deterioration depths of concrete for different water operating velocities (Theewaterskloof test section)**

The concrete of the test section has to be tested to determine the actual depth of deterioration (amount of Ca-ions present) in order to calibrate this model. More research is also necessary to determine the specific influence of the permeability, porosity and aggressiveness of the water on the deterioration rate of the concrete.

f) Comparison with field data

A core, 400 mm in diameter and 700 mm in length, was drilled from the concrete of the Theewaterskloof Tunnel 21 km from the inlet of the tunnel. In the laboratory a 100 mm diameter core was taken from the original core. The 100 mm diameter core was cut into $\pm 3\text{mm}$ thin slices with a diamond saw. Every sample was crushed into a fine powder. The powder was tested for the amount of CaO present by using a titration with [EDTA]. The data can be seen in Appendix A-7.

In the first 3mm (on the surface) of the 100 mm diameter core the CaO content was 6,9 %. At a depth of 7mm – 17mm the CaO content was 9,5 % and at a depth of 210 mm – 213 mm the CaO content was about 8 %.

According to the composition of the cement and the concrete (see Table 7.3.4-1 and 7.3.4-2) the total amount of CaO in the concrete mixture has to be 11 %, therefore it is possible that the concrete had been affected deeper than 213 mm. More research is necessary to evaluate the depths to which deterioration took place.

7.3.5 LHWP delivery tunnel pre-cast concrete lining section

a) Materials used in the concrete mixture for the 5m-test section

The original materials used to cast the LHWP delivery tunnel concrete linings such as the coarse and fine aggregates have been obtained from the field by DWAF to use in the concrete for the test section.

The cementitious materials originally used for the LHWP delivery tunnel concrete linings are cement CEM I 42.5 from Anglo Alpha's Ulco plant in the Northern Cape and fly-ash from Ash Resources' Lethabo plant. The chemical composition of the cement and fly-ash is listed in Table 7.3.5-1 and Table 7.3.5-2 respectively.

Table 7.3.5-1 Chemical composition of cement used for LHWP tunnel section

Chemical composition	Percentage %	Mineralogical Composition according to the Bogue calculation	Percentage %
SiO ₂	20,94	Tricalcium silicate (C ₃ S)	68,7
Al ₂ O ₃	3,99	Dicalcium silicate (C ₂ S)	8,6
Fe ₂ O ₃	2,59	Tricalcium aluminate (C ₃ A)	6,2
Mn ₂ O ₃	1,11	Tetracalcium Aluminoferrite (C ₄ AF)	7,9
TiO ₂	0,23		
CaO	65,03		
MgO	2,41		
SO ₃	2,22		
K ₂ O	0,59		
Na ₂ O	0,08		

Table 7.3.5-2 Chemical composition of Lethabo fly-ash

Lethabo fly-ash	Chemical composition %
SiO ₂	53,8
Al ₂ O ₃	34,3
Fe ₂ O ₃	3,6
Mn ₂ O ₃	0,1
TiO ₂	1,6
CaO	4,4
MgO	1,1
SO ₃	0,1
K ₂ O	0,5
Na ₂ O	0,4
P ₂ O ₅	0,3
LOI	0,8

The concrete mixture used in the LHWP test section is as indicated in Table 7.3.5-3:

Table 7.3.5-3 Concrete mixture for pre-cast linings of the LHWP delivery tunnels from the 1994 records as obtained by DWAF	
Materials	(Kg/m³)
Cement OPC, Ulco	360
Cement Extender (PFA) Lethabo	60
Coarse Aggregate – Dolerite @ 19mm	1050
Coarse Aggregate – Dolerite @ 9.5 mm	405
Fine aggregate rivers sand - Caledon river	190
Fine aggregate Crusher dust - Dolerite	385
Admixture - air entrainment (conplast 316)	0.3
Admixture – Superplasticiser (conplast 337)	4
Water	138
Properties of mixture	
28-day compressive strength	103 MPa
Coefficient of oxygen permeability (k_{og})	6.28 ($m^2 \cdot 10^{-18}$)
Porosity	13,65 %

The test on the LHWP Delivery Tunnel pre-cast concrete lining section started at the middle of September 2001 and ran 24 h per day until the middle of November 2001. Head loss readings, water temperature readings and water samples were taken in the mornings and afternoons. The operating velocities was steady at about 1,2 m/s, 2,3 m/s and 2,73 m/s. The temperature of the water varied between 19 °C and 29 °C. The natural pH – values of the water varied between 6,9 and 8,4 with a mean of 7,8. See Appendix A-4 and A-5 for the data. (The mean pH in the field at the tunnel outlet is 7.9).

b) Observed rate of leaching

The water was analysed for the amount of Ca – ions present in the water by using a titration with [EDTA] Concentration of Di-sodium Ethylenediamine Tetra-acetate. See Appendix A-5 for leaching data.

The observed rate of leaching of Ca-ions from the concrete for water operating velocities of 1.9, 2.3 and 2,7 m/s can be seen in Figure 7.3.5-1.

It is clear from Figure 7.5.3-1 that in the first two days the calcium hydroxide ($Ca(OH)_2$) is leached at a much higher rate. The same happened at the Theewaterskloof test section therefore it is possible that excess unhydrated cement is present on the surface of the concrete. The water was replenished every time the operating velocity was changed. Linear trendlines were added to the graphs and the equation and R-squared values are displayed on the figure.

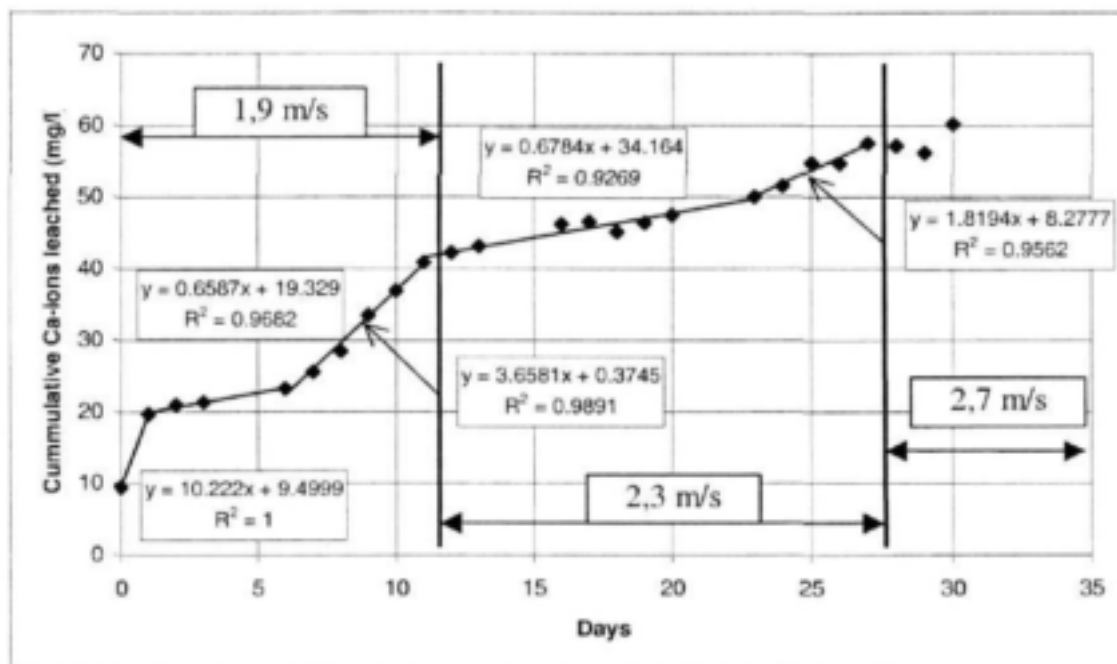


Figure 7.3.5-1 Cumulative leaching of Ca-ions per day (LHWP test section).

Two constant leaching rates for a water operating velocity can be observed from Figure 7.3.5-1. From the literature review it is possible that the lower leaching rate is the rate of decalcification of the C-S-H gel and the higher leaching rate is the rate of $\text{Ca}(\text{OH})_2$ leaching. The decalcification rate is similar for both the operating velocities, but the rate of $\text{Ca}(\text{OH})_2$ leaching for the two velocities differs.

The duration of the test run at an operating water velocity of 2,7 m/s was not enough to determine the leaching rate. More research is therefore necessary to evaluate the leaching rate of higher operating water velocities.

c) Theoretical concrete deterioration

The literature, which has been reviewed indicates that replacement of cement in the concrete mixture with fly ash increases the resistance of the cement-based paste to soft water attack. (Basson, 1989) For LHWP 14% of the cement-based paste in the concrete mixture (see Table 7.3.5-3) consists of fly-ash, which contains much less calcium hydroxide ($\text{Ca}(\text{OH})_2$) compared to the ordinary Portland cement. The theoretical calcium hydroxide ($\text{Ca}(\text{OH})_2$) content of the hydrated paste at an age of 28-days can be determined by using the mineralogical composition (see Table 7.3.5-1 & 7.3.5-2) of the cement and the fly-ash used in this concrete mixture.

According to Table 7.3.5-1 the Portland cement used in the LHWP delivery tunnel contains 68,7 % C_3S , 8,6 % C_2S , 6,2 % C_3A and 7,9 % C_4AF .

See Appendix A-3 for calculations:

100 kg of Ulco Portland cement produces:

$$(0,750 \times 68,7) + (0,994 \times 8,6) = 60,07 \text{ kg } \text{C}_3\text{S}_2\text{H}_3$$

$$(0,487 \times 68,7) + (0,215 \times 8,6) - (0,274 \times 6,2) - (0,609 \times 7,9) = 28,80 \text{ kg } \text{Ca}(\text{OH})_2$$

Potentially: 100 kg of fly-ash reacts with $(185 \times 0,538) = 99,53 \text{ kg } \text{Ca}(\text{OH})_2$

to produce $(285 \times 0,583) = 166,16 \text{ kg } \text{C}_3\text{S}_2\text{H}_3$

1 m³ of the concrete mixture consists of (Table 7.5.3-3):

360 kg Ulco Portland cement, which produces $(3,6 \times 60,07) = 216,25 \text{ kg } \text{C}_3\text{S}_2\text{H}_3$ and $(3,6 \times 28,79) = 103,99 \text{ kg } \text{Ca}(\text{OH})_2$

The 60 kg fly-ash in the mixture reacts with $(0,6 \times 99,53) = 59,72 \text{ kg } \text{Ca}(\text{OH})_2$

to produce $(0,6 \times 166,16) = 99,69 \text{ kg } \text{C}_3\text{S}_2\text{H}_3$

Therefore the total amount is:

$(216,25 + 99,69) = 315,85 \text{ kg/m}^3 \text{C}_3\text{S}_2\text{H}_3$ and

$(103,99 - 59,72) = 44,28 \text{ kg/m}^3 \text{Ca}(\text{OH})_2$

Assumptions:

The theoretical depth of deterioration of the concrete is calculated by assuming that the calcium hydroxide ($\text{Ca}(\text{OH})_2$) in a certain depth of concrete is leached at a rate as determined with the leaching data and then total decalcification of the C-S-H gel in this volume of concrete occurs at a different rate (see figure 7.3.4-4).

The $\text{Ca}(\text{OH})_2$ is distributed evenly through the concrete mixture.

The permeability is also a factor influencing the depth of deterioration but is not taken into account during this calculation.

The theoretical depth of deterioration of the concrete is for de-ionised distilled water with a mean temperature of 27 °C and a natural pH – value of about 7.

The atomic mass of $\text{Ca}(\text{OH})_2 = 40 + 16 + 16 + 1 + 1 = 74$

The atomic mass of Ca-ion = 40

Therefore: $\frac{40}{74} = 54\%$ Ca-ions in $\text{Ca}(\text{OH})_2$

The atomic mass of $\text{C}_3\text{S}_2\text{H}_3$ is 342

Therefore: $\frac{3 \times 40}{342} = 35\%$ Ca-ions in $\text{C}_3\text{S}_2\text{H}_3$

Concrete:

But 420 kg / m³ (360 kg / m³ OPC + 60 kg / m³) of the concrete mixture consists of cement based binder (Table 7.3.5-2)

Therefore: $(0,54 \times 44,28) = 23,91 \text{ kg / m}^3$ Ca-ions in the form of $\text{Ca}(\text{OH})_2$ + $(0,35 \times 315,85) = 110,548 \text{ kg / m}^3$ Ca-ions in the form of C-S-H gel

Operating water velocity of 1,9 m/s:

Water:

The Ca-ion leaching rate for decalcification from Figure 7.3.4-4 is 0,66 g / kℓ

And the Ca-ion leaching rate for $\text{Ca}(\text{OH})_2$ is 3,66 g / kℓ

The total volume of water is 7 kℓ

Therefore: The total Ca-ions leached as a result of decalcification is $(0,66 * 7) = 4,62$ g per day or 0,19 g / hour

And the total Ca-ions leached as $\text{Ca}(\text{OH})_2$ is $(3,66 * 7) = 25,62$ g per day or 1,07g / hour

Concrete:

If a depth of 10 μm of the concrete deteriorates, the volume of concrete affected is $0,15 \text{ m} * \pi * 5 \text{ m} * 0,000010 \text{ m} = 0,00002356 \text{ m}^3$

In $0,00002356 \text{ m}^3$ concrete there is:

$(0,00002356 \text{ m}^3 * 23,911 \text{ kg} / \text{m}^3) = 0,563 \text{ g}$ Ca-ions in the form of $\text{Ca}(\text{OH})_2$ and
 $(0,00002356 \text{ m}^3 * 110,54 \text{ kg} / \text{m}^3) = 2,605 \text{ g}$ Ca-ions in the form of C-S-H gel

Time for leaching:

$(0,563 \text{ g} / 1,07 \text{ g} / \text{hour}) = 0,60 \text{ hour}$

plus $(2,605 \text{ g} / 0,19 \text{ g} / \text{hour}) = 13,71 \text{ hour}$

Total time for 10 μm of concrete to deteriorate = 14.31 hours

$$\frac{10\mu\text{m}}{14.31\text{h}} = \frac{x}{24\text{h}}$$

$$x = 16,77$$

The theoretical depth of deterioration of the concrete with a water operating velocity of 1,9 m/s = **16,77 μm / day**

Operating water velocity of 2,3 m/s

The total Ca-ions leached as a result of decalcification is $(0,68 * 7) = 4,76$ g per day or 0,2 g / hour

And the total Ca-ions leached as $\text{Ca}(\text{OH})_2$ is $(1,82 * 7) = 12,74$ g per day or 0,531 g / hour

Concrete

If a depth of 10 μm of the concrete is deteriorated the volume of concrete affected is $0,15 \text{ m} * \pi * 5 \text{ m} * 0,000010 \text{ m} = 0,00002356 \text{ m}^3$

In $0,00002356 \text{ m}^3$ concrete there is:

$(0,00002356 \text{ m}^3 * 23,91 \text{ kg} / \text{m}^3) = 0,563 \text{ g}$ Ca-ions in the form of $\text{Ca}(\text{OH})_2$ and
 $(0,00002356 \text{ m}^3 * 110,54 \text{ kg} / \text{m}^3) = 2,605 \text{ g}$ Ca-ions in the form of C-S-H gel

Time for leaching:

$(0,563 \text{ g} / 0,531 \text{ g} / \text{hour}) = 1,06 \text{ hour}$

plus $(2,605 \text{ g} / 0,2 \text{ g} / \text{hour}) = 13,03 \text{ hour}$

Total time for 10 μm of concrete to deteriorate = 14,09 hours

$$\frac{10\mu\text{m}}{14.09h} = \frac{x}{24h}$$

$$x = 17,03$$

The theoretical depth of deterioration of the concrete with a water operating velocity of 2,3 m/s = **17,03 μm / day**

There is not enough data available at this stage but according to Table 7.3.5-3 it seems that the concrete deterioration rate increase with an increase of water operating velocity. It is clear that the theoretical rate of deterioration depth per day of the LHWP tunnel with the fly-ash as cement extender is much lower than that of the Theewaterskloof tunnel section with no cement extender.

Table 7.3.5-4: Theoretical depth of deterioration of the concrete for LHWP tunnel section

Operating water velocity	Theoretical depth of deterioration of the concrete
1,9 m/s	16,77 $\mu\text{m/day}$
2,3 m/s	17,03 $\mu\text{m/day}$

7.3.6 Hydraulic roughness and soft water corrosion

a) Test setup and procedure

The hydraulic roughness based on the Colebrook-White equation was determined for the three concrete test sections. Data on the first test pipe were obtained by running the test at a steady flow velocity of about 1.2 m/s. At the end of the tests at this flow, the flow velocity was increased to 3 m/s for one day at a time, and after each high flow the flow was reduced to test the roughness at 1.2 m/s again. The aim was to test what happened during steady low flows and whether equilibrium had been reached.

An OPC cement was used and the concrete mix was a general low strength mix. The original purpose of this test was to finalize the test procedure for the LHWP tunnel and Theewaterskloof Tunnel sections, but very useful data were obtained. This led to several changes such as the use of pressure gauges instead of a manometer, construction of a permanent roof over the pumps, construction of a test section at University of Pretoria instead of at DWAF Materials Laboratory due to transport problems with the test section weighing more than a ton.

Concrete for the second test section was based on the Theewaterskloof Tunnel design, without slagment, with aggregate obtained from Western Cape (Berg River) by DWAF and transported to Pretoria where the test section was cast at the University of Pretoria. The tests were carried out for three steady flow periods until hydraulic roughness equilibrium was more or less reached.

The third and last test section was based on the LHWP Delivery Tunnel segmentally lined concrete. Aggregate for this test section was obtained from site by TCTA who also arranged transport to Pretoria. Three steady flows were tested, increasing with time.

The test section was 5 m in length, cast in a top and bottom half around a steel pipe as mould to get a smooth finish. The test pipe had an inside diameter of 0.14 m.

b) Hydraulic roughness results

The results of the tests carried out on the pilot test pipe are shown in Figures 7.3.6-1 and 7.3.6-2., while the roughness time series of the Theewaterskloof and LHWP DTN pipe section laboratory tests are shown in Figures 7.3.6-3 and 7.3.6-4. The following is concluded from the test results:

Pilot test (Figure 7.3.6-1):

- There was a dramatic increase in roughness over the first 5 test days, which is in agreement with rapid leaching that occurred over the same period (see Figure 7.3.3-3)
- Near the end of the low flow test period there was again a slight increase in Ca-ions leaching that corresponds with a slight increase in roughness
- The low flow tests at 1,2 m/s indicated an initial increase in roughness from 1 mm to about 3 mm, followed by a general scatter of between 1.7 to 3.1 mm. This latter range of values is due to measurement inaccuracy due to the manometer water levels varying. Each point however represents the mean value of at least 20 readings taken at the upstream and downstream gauges.

Pilot test (Figure 7.3.6-2):

- The high flow testing at > 3 m/s increased the roughness and doubled it after 2 days. After repairs were carried out to the pump system and the pipe was empty for several months, high flow testing continued and again after a day the roughness tested higher at 6 mm. No chemical leaching tests were carried out during these tests. One last high flow test lasting a day was carried out and while it was expected that the roughness would stabilise at about 6 mm, this did not happen and instead a dramatic increase occurred to reach 9 mm. The tests were stopped at that stage due to leakage, loss of water and time constraints. It is not clear whether the roughness would have increased with longer duration high flow testing.

The rapid increase in roughness following high flows could not be simulated with higher strength tunnel concrete such as being used in the other two test sections. When the pilot test section was opened up after the test, it was found that the seal between the upper and bottom halves had moved into the flow zone, and this must have happened prior to the start of the tests. This led to relatively high energy losses and a secondary loss as function of the velocity head was incorporated in the analysis to determine k_s . In further tests of the tunnel sections no secondary loss coefficient was used to determine the hydraulic roughness.

The seal material inside the test pipe would create convergence and divergence losses, but would also be oscillating at high flows, changing the velocity profile, resulting in excessive erosion of the concrete. Inspection of the inside of the pipe after tests showed a relatively coarse surface and none of the original smooth finish area was present. The absolute roughness values obtained from the pilot test should not, however, be used as absolute values due to the many assumptions that had to be made and because this was a low strength concrete not typical of tunnel concrete.

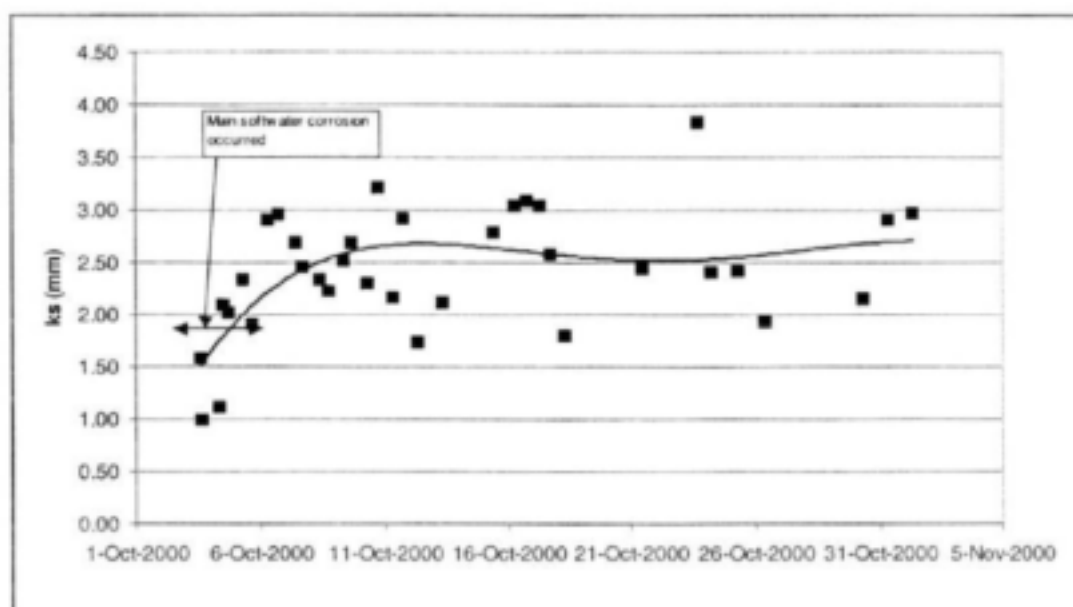


Figure 7.3.6-1 Laboratory hydraulic roughness changes due to softwater at $v = 1.2$ m/s: Pilot test section

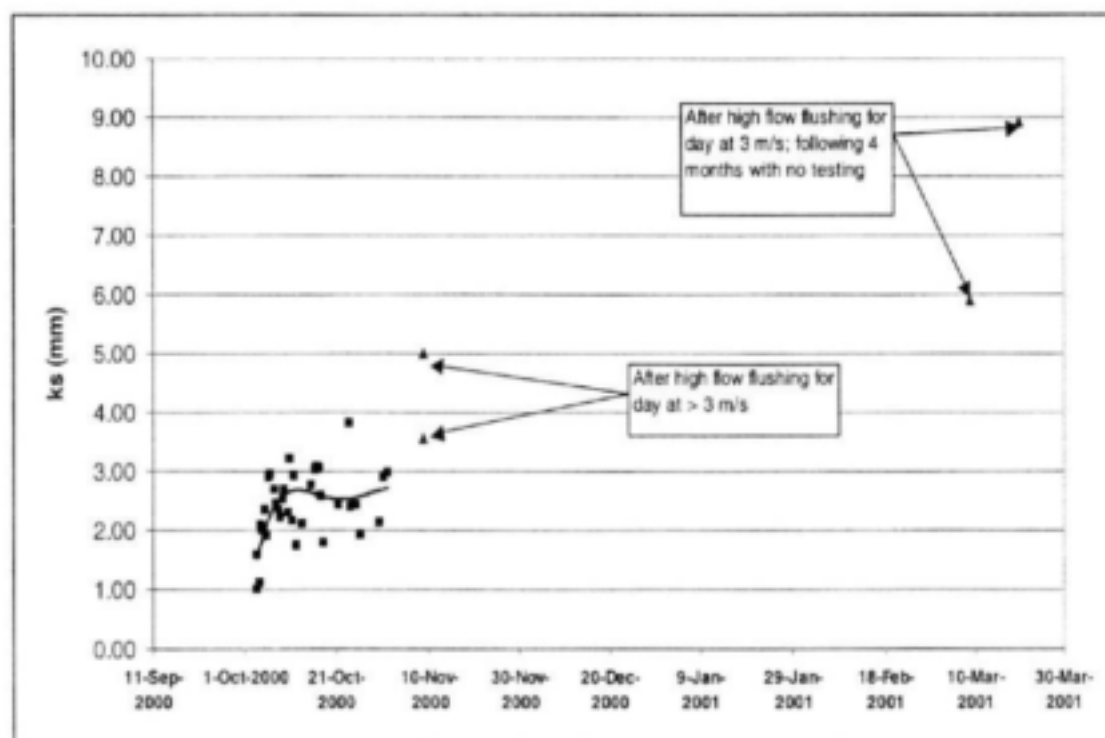


Figure 7.3.6-2 Laboratory hydraulic roughness changes due to softwater at $v = 1.2$ m/s and following 3 m/s flushing: Pilot test section

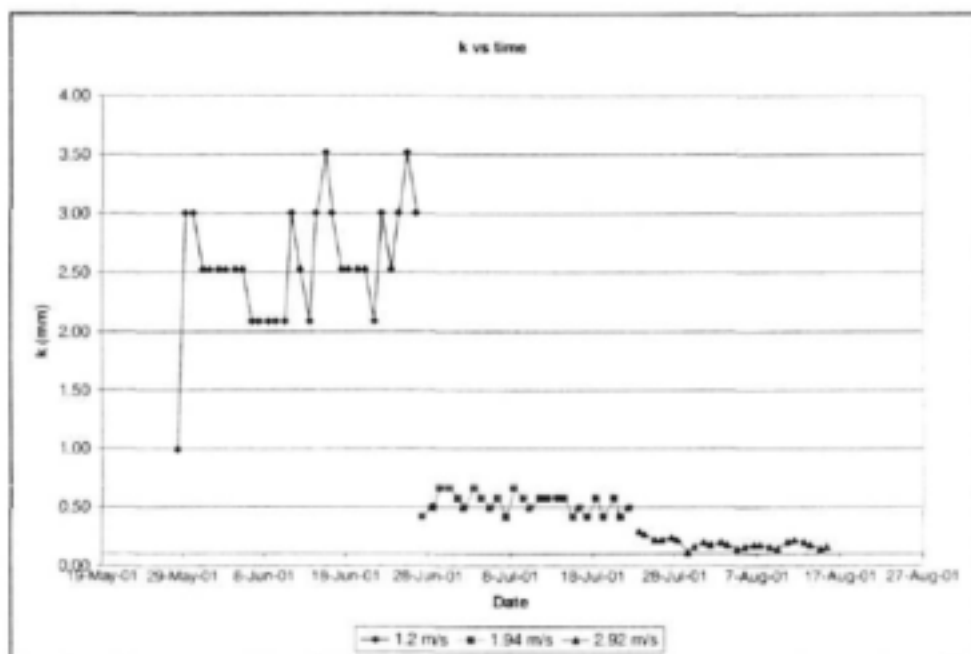


Figure 7.3.6-3 Laboratory hydraulic roughness changes due to softwater: Theewaterskloof Tunnel test section

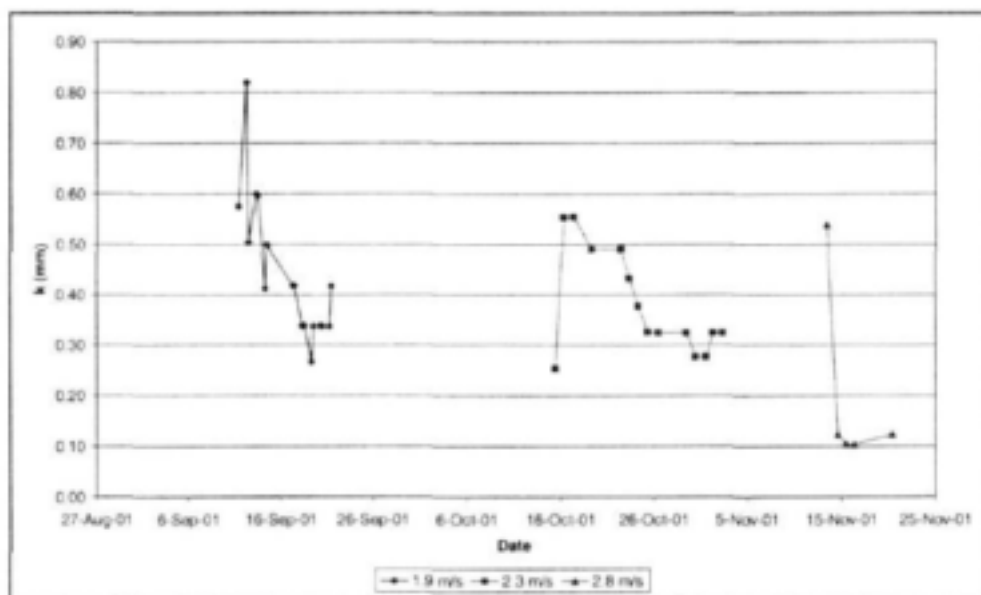


Figure 7.3.6-4 Laboratory hydraulic roughness changes due to softwater: LHWP DTN test section

Theewaterskloof test results (Figure 7.3.6-3):

- During the first day of testing there was a dramatic increase in roughness, from 1 to 3 mm, which agrees with the Ca-ions leaching rate, and then the roughness stabilised between 2 to 3 mm. This pattern is similar to what was experienced during the pilot tests.

- At 1.94 m/s the roughness experienced is much lower at 0.5 mm than at 1.2 m/s. There was again a small increase in initial roughness as the test started. The rest of the test at 2.3 m/s showed steady roughness, in agreement with the Ca-ions leaching rate.
- At 2.9 m/s there was an initial roughness decrease, stabilizing at between 0.1 to 0.2 mm. No deterioration of the pipe leading to high roughness, as experienced during the pilot test, was experienced, probably due to the higher strength concrete.

LHWP DTN test results (Figure 7.3.6-4):

- Again there is an initial increase in roughness in agreement with a high Ca-ions leaching rate, but the increase in roughness was not as high as for the Theewaterskloof test.
- The range of roughness for 1.9 m/s is more or less similar to that of the Theewaterskloof test, about 0.4 to 0.7 mm. There was however a more definite decrease in roughness with time at the LHWP test, stabilising at the end.
- At 2.3 m/s an initial rise in k_s was observed, but not picked up in the Ca-ions leaching rate. With time the roughness decreased from about 0.5 mm to reach roughness equilibrium at 0.33 mm. The Ca-ions leaching however continues after equilibrium roughness has been reached.
- At the highest flow the roughness decrease rapidly to about 0.11 mm and remained steady. The Ca-ions leaching rate became very low at the same time. The final roughness achieved agreed with that at the same velocity of the Theewaterskloof tests. The bed shear stress at 2.8 m/s in the pipe was 38 Pa, which is generally higher than that for field conditions.
- An important conclusion from the laboratory tests is that although Ca-ions leaching occurs and the surface of the concrete softens so that it can easily be damaged by a hard object, high velocity flow conditions did not increase the hydraulic roughness when concrete of high quality and strength is used.

7.3.7 Proposed concrete design

According to a paper by Castro & McIntosh (1994) an optimum concrete mixture was designed for the pre-cast concrete linings of the LHWP tunnels (see Table 7.3.5-1). A concrete mixture having a total cementitious content of 360 kg/m³ and a fly-ash content of 40 % (cement 216 kg/m³ and fly-ash 144 kg/m³) was proposed to ensure a resistance to soft water attack.

The actual concrete mixture Neumann & True (1994) used in the linings has a fly-ash content of only 17 % and a total cementitious content of 420 kg/m³ (see Table 7.3.5-2).

The optimum concrete mixture as design by Castro & McIntosh (1994) will be a mixture with a higher resistance to soft water attack than the mixture used in the linings, because the Ca(OH)₂ content is much lower.

Supervision at the construction stage of concrete structures is of the utmost importance because correct placing, compaction and curing will ensure low permeability and low porosity. These two factors have an influence on the resistance of the concrete mixture to soft water attack.

Research on the use of silica fume in these mixtures is also necessary, because it will decrease the permeability of the concrete and also the Ca(OH)₂ content.

Table 7.3.7-1: Mix proportion for LHWP tunnel linings as proposed by Castro & McIntosh (1994)

Materials	Mix (kg/m ³)
Cement	216
Cement Extender	144
Coarse Aggregate - 25 mm	820
Coarse Aggregate - 12 mm	385
Fine aggregate rivers sand	370
Fine aggregate Crusher dust	370
Admixture - water reducer	1.8
Admixture - air entrainment	0.7
Admixture - Superplasticiser	3.6
water	180

7.4 Conclusions on chemical aspects

Literature, which has been reviewed, indicates that there are two main factors influencing the deterioration of the concrete due to soft water attack namely the aggressiveness of the water and the composition of the concrete under attack.

For the laboratory tests the aggressiveness of the water was kept constant by using de-ionised, distilled water at a mean temperature of 27°C and a natural pH value of about 7. The aggressiveness of the water is severe to ensure accelerated soft water attack on the concrete.

No cement extenders were used in the concrete of the Theewaterskloof Tunnel and it was found that the leaching rate of Ca-ions increase exponentially with an increase in the water operating velocity in the pipe system. The main chemical process taking place in the concrete during soft water attack in the Theewaterskloof Tunnel is the leaching of $\text{Ca}(\text{OH})_2$. A maximum and minimum theoretical concrete deterioration depth rate was determined for the concrete. These rates also increase exponentially with increasing water operating velocity in the pipe system. The permeability of this concrete is very high and therefore the leaching rates are also high.

The concrete mixture of the LHWP linings of the delivery tunnels contains fly-ash as a cement extender. Two different Ca-ion leaching rates for a specific water operating velocity were found for the LHWP concrete. This indicates that as a result of the fly-ash content in the mixture decalcification of the C-S-H gel takes place at a low rate and $\text{Ca}(\text{OH})_2$ leaching at a higher rate. The theoretical concrete deterioration depth rates for the LHWP concrete are much lower than the rates for the Theewaterskloof Tunnel. There are not enough data available at this stage but it seems that the concrete deterioration rate increases with increasing water operating velocity.

Although the soft water attack cannot be stopped with a specific concrete mix design it is possible to decrease it significantly with an optimum design. The optimum concrete mixture as designed by Castro & McIntosh (1994) for the LHWP tunnels will still be the best mixture for resistance to soft water attack because the $\text{Ca}(\text{OH})_2$ content is very low in this design mix.

7.5 Recommendations

Attention to the following aspects is necessary:

- More laboratory tests are necessary on the LHWP test section at higher water operating velocities to determine the relationship between the Ca-ions leaching rates and water operating velocities.
- Tests on the microstructure of the concrete of the test sections are necessary to calibrate the rate of the theoretical concrete deterioration depths and to determine the exact influence of the permeability and porosity on soft water attack.
- Tests on the Theewaterskloof Tunnel core have to be completed to calibrate laboratory tests.
- A core from LHWP is required to calibrate laboratory tests.
- The aggressiveness of the water actually flowing through the tunnels should be determined. Laboratory tests with the water from the field will be necessary to confirm the calibration of the accelerated rate of deterioration in the laboratory with the actual field behaviour.

Further tests on concrete mixtures with silica-fume as cement extender will be valuable to determine a concrete mixture with a higher resistance to soft water attack.

8 REMEDIAL MEASURES

The following remedial measures have been investigated in this report:

- Hydraulic flushing (field and laboratory tests)
- Mechanical cleaning (field tests)
- Chlorination
- Tunnel lining
- Doubling or new tunnels
- *Minimization of secondary losses*

8.1 Hydraulic flushing

8.1.1 Introduction

When tunnels are operated at high flow velocities, sediment deposits can be re-entrained and flushed out of the tunnel, thereby increasing the discharge capacity of a tunnel. In this chapter hydraulic flushing in field and laboratory conditions was investigated, in hardwater and softwater conditions.

8.1.2 Orange-Fish Tunnel experience

a) Tunnel description

The tunnel consists of about 81 km of 5.33 m diameter in situ concrete lined tunnel, constructed during the 1970's and is the longest irrigation tunnel in the world. The tunnel transfers water mainly for irrigation to the Eastern Cape from Gariep Reservoir. A longitudinal profile of the tunnel is shown in Figure 8.1.2-1. The tunnel flow is controlled downstream by 6 large valves (Figure 8.1.2-2), and the tunnel flow is measured at a large Parshall flume at the outlet, as well as at pumps and a small flume at a canal diversion at the outlet at Theebus (Figures 8.1.2-3 and 8.1.2-4).

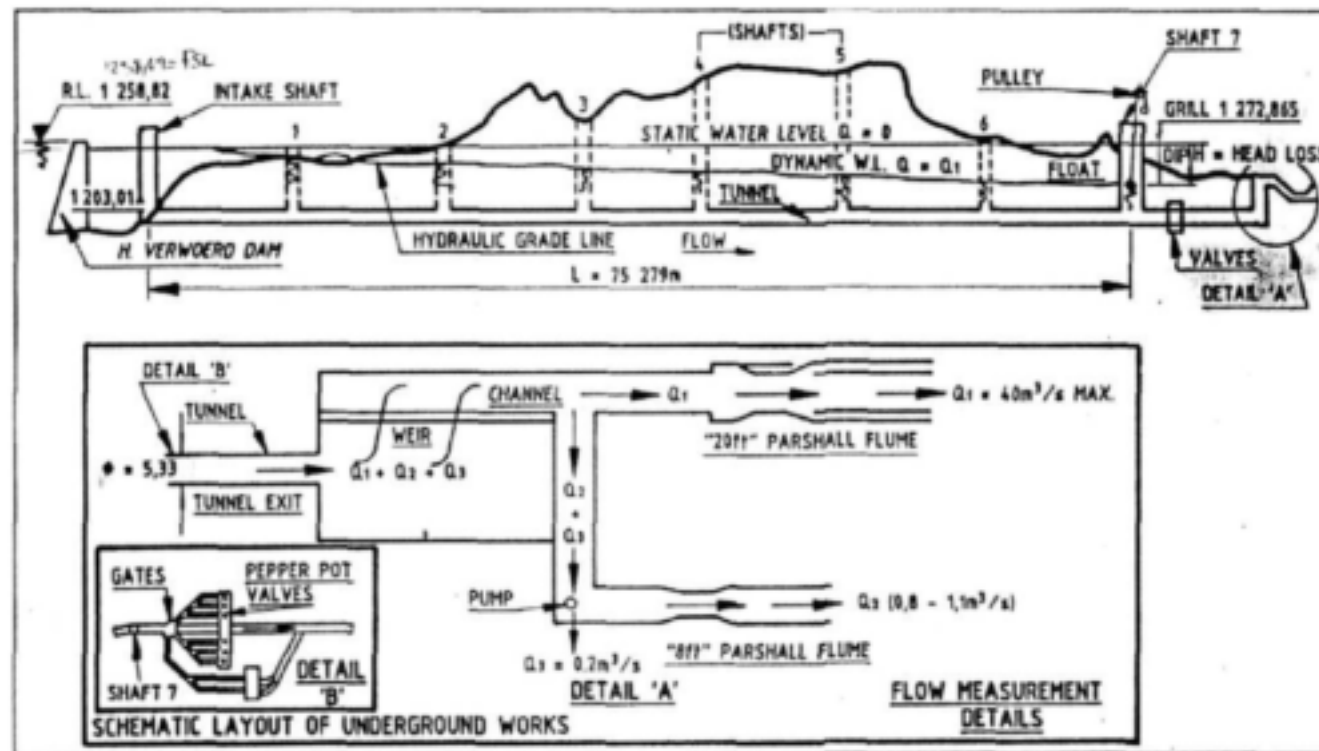


Figure 8.1.2-1 Longitudinal profile of Orange-Fish Tunnel



Figure 8.1.2-2 Orange-Fish Tunnel valve

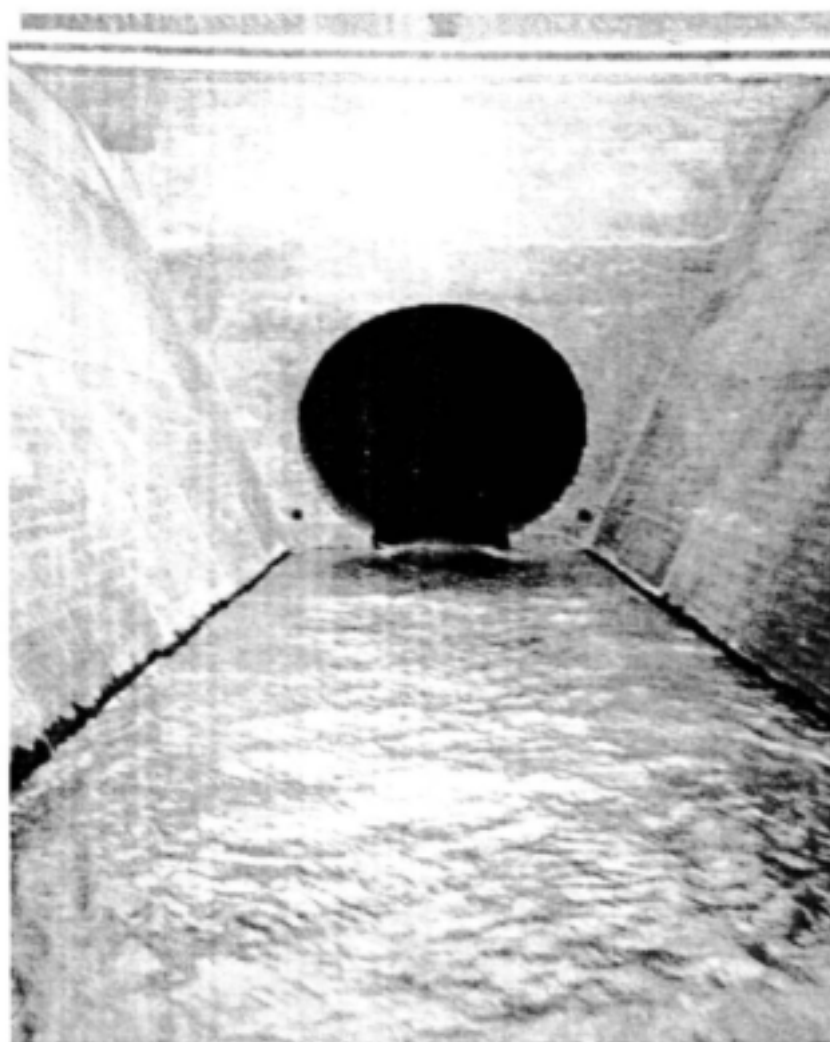


Figure 8.1.2-3 Orange-Fish Tunnel outlet



Figure 8.1.2-4 Orange Fish Tunnel flow measurement gauge

b) Historical tests

Commissioning and other subsequent hydraulic tests were carried out during the 1970s and 1980s. Rapid ageing was observed due to the first 10 years of operation at relatively low flow conditions. This resulted in sediment deposition in the tunnel, consisting of clay fractions with a brown colour. The observed maximum annual flows are shown in Figure 8.1.2-5.

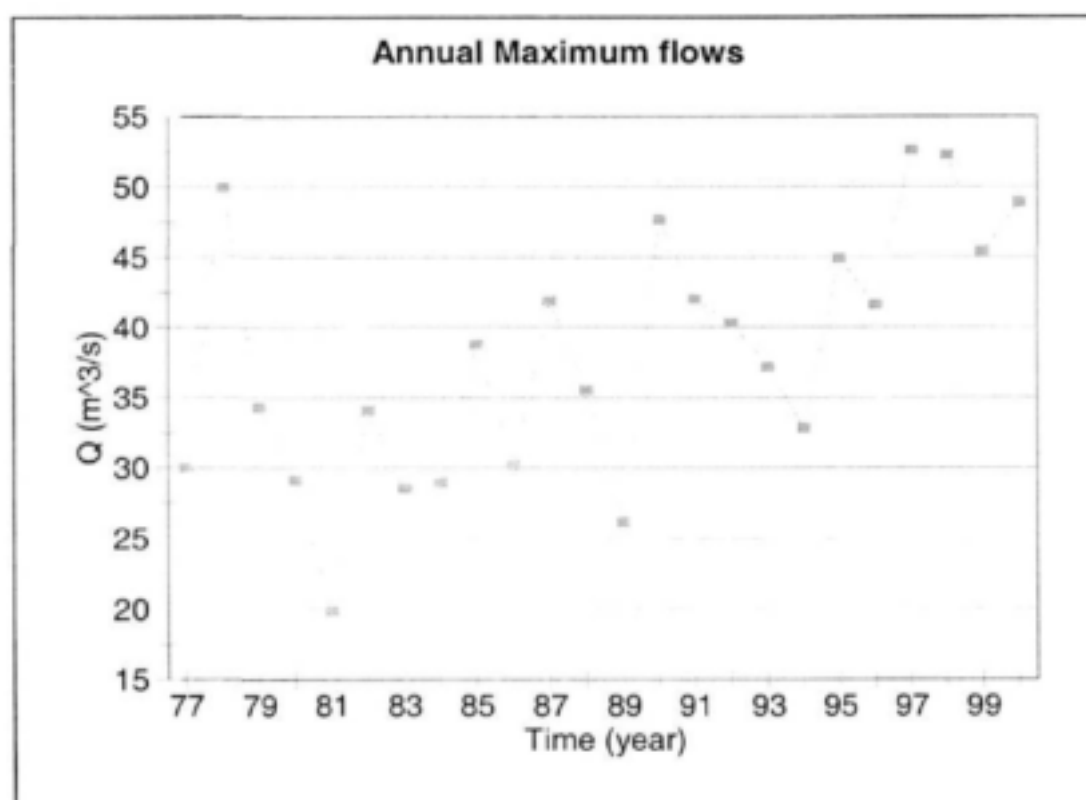


Figure 8.1.2-5 Observed maximum annual Orange-Fish Tunnel flows

c) Tunnel inspection 1999

The tunnel was inspected by driving through it in a day. Almost no sediment was visible. In many places the concrete had the original smooth finish created by the shuttering. The concrete is hard and the water is not aggressive to concrete (Figures 8.1.2-6 and 8.1.2-7).

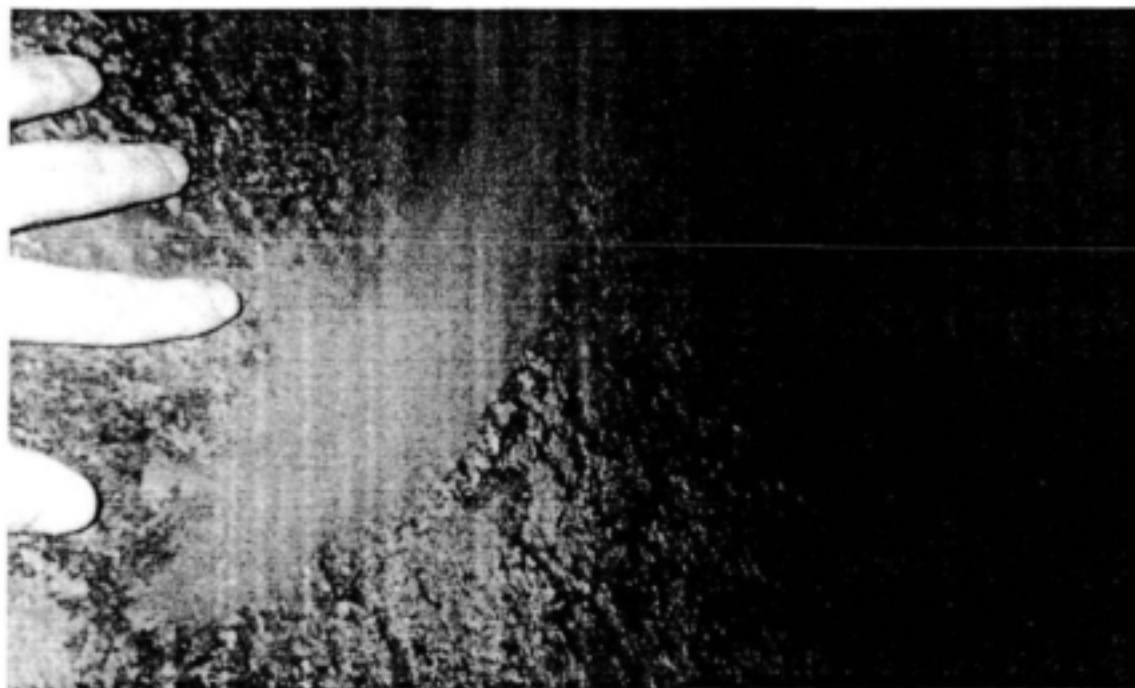


Figure 8.1.2-6 Localized clay deposition in Orange-Fish Tunnel

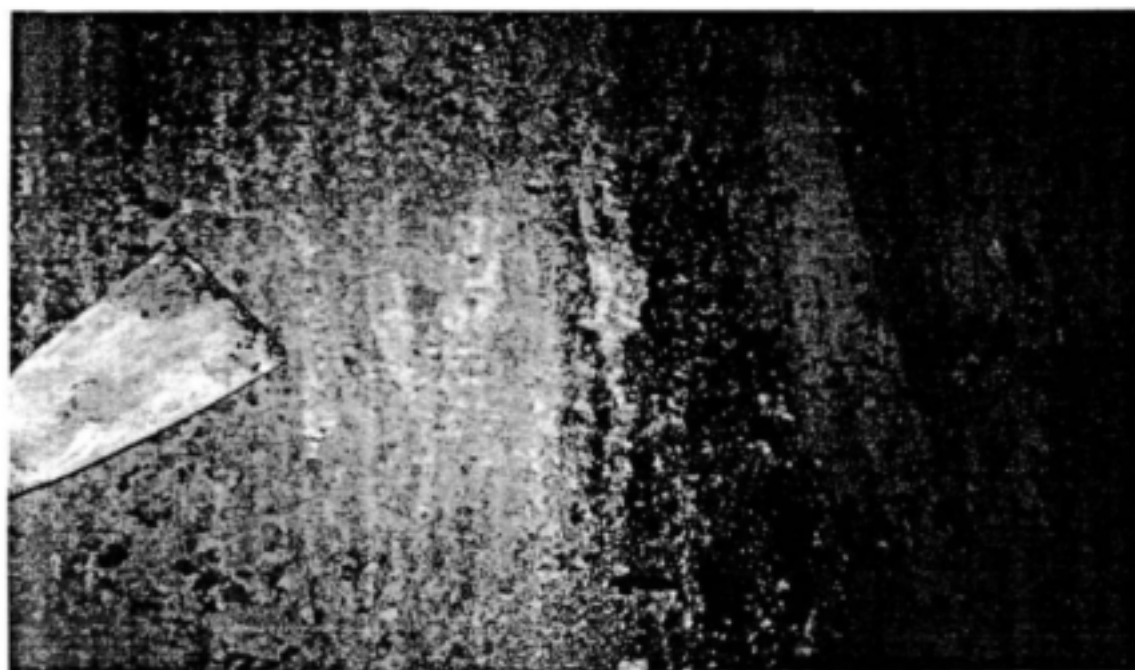


Figure 8.1.2-7 Typical hard smooth concrete surface of Orange-Fish Tunnel

d) Tunnel test 1998

This test involved periods of steady flow, increasing to maximum and then decreasing. Shaft water levels were gauged by borehole type depth gauges. Static water levels were also determined. Figures 8.1.2-8 and 8.1.2-9 show a depth gauge used during the tests. Ott water level recorders were however also installed for continuous monitoring (Figure 8.1.2-10). These recorders worked at most shafts but difficulties were experienced at the deep shafts (400 m) and during discharge adjustments when the surge pulled the float into the tunnel. Figure 8.1.2-11 shows the test results together with other historical data.



Figure 8.1.2-8 Depth gauge at tunnel shaft operated by DWAF personnel



Figure 8.1.2-9 Depth recording at Orange-Fish Tunnel shaft.



Figure 8.1.2-10 Ott water level reader and Thalimedes data logger

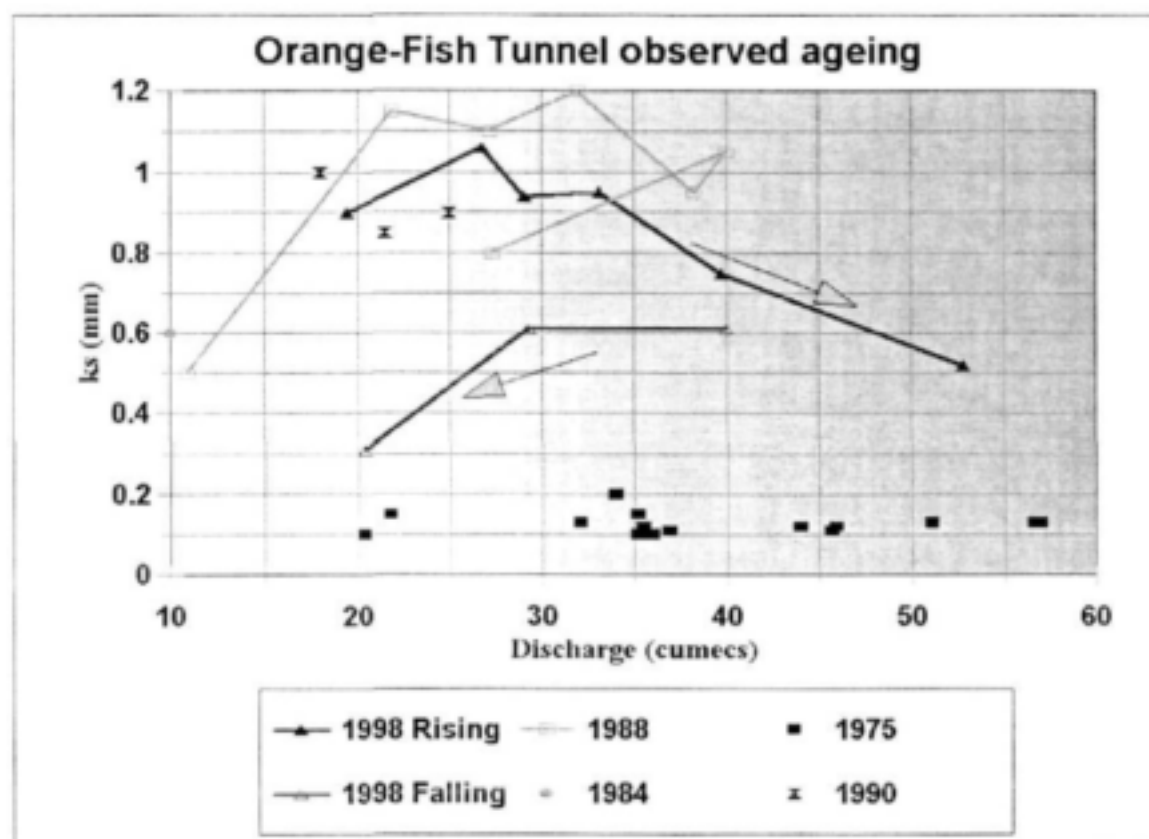


Figure 8.1.5-11 Historically observed hydraulic roughness of Orange-Fish Tunnel

In 1975 the new tunnel roughness was about 0.13 mm for all flows tested up to the maximum discharge of 57 m³/s. By 1984 the roughness had increased to 0.6 mm, and during 1988 the roughness varied between 1.2 mm and 0.8 mm at higher flows. This roughness was confirmed in 1990 tests. During 1998 another comprehensive hydraulic test was carried out, but this time in discharge increments going up right to the maximum capacity. The tests started at the same roughness as for the 1990 test, but as the discharge increased the roughness decreased with $k_s = 0.5$ mm at 53 m³/s. The flushing effect at high flow however became apparent when evaluating the downward steps in flows: at 29 m³/s k_s dropped from 0.94 mm to 0.6 mm, and at 20 m³/s it reduced from 0.9 mm to 0.3 mm.

The flushing changed the long-term ageing rate as indicated in Figure 8.1.2-12.

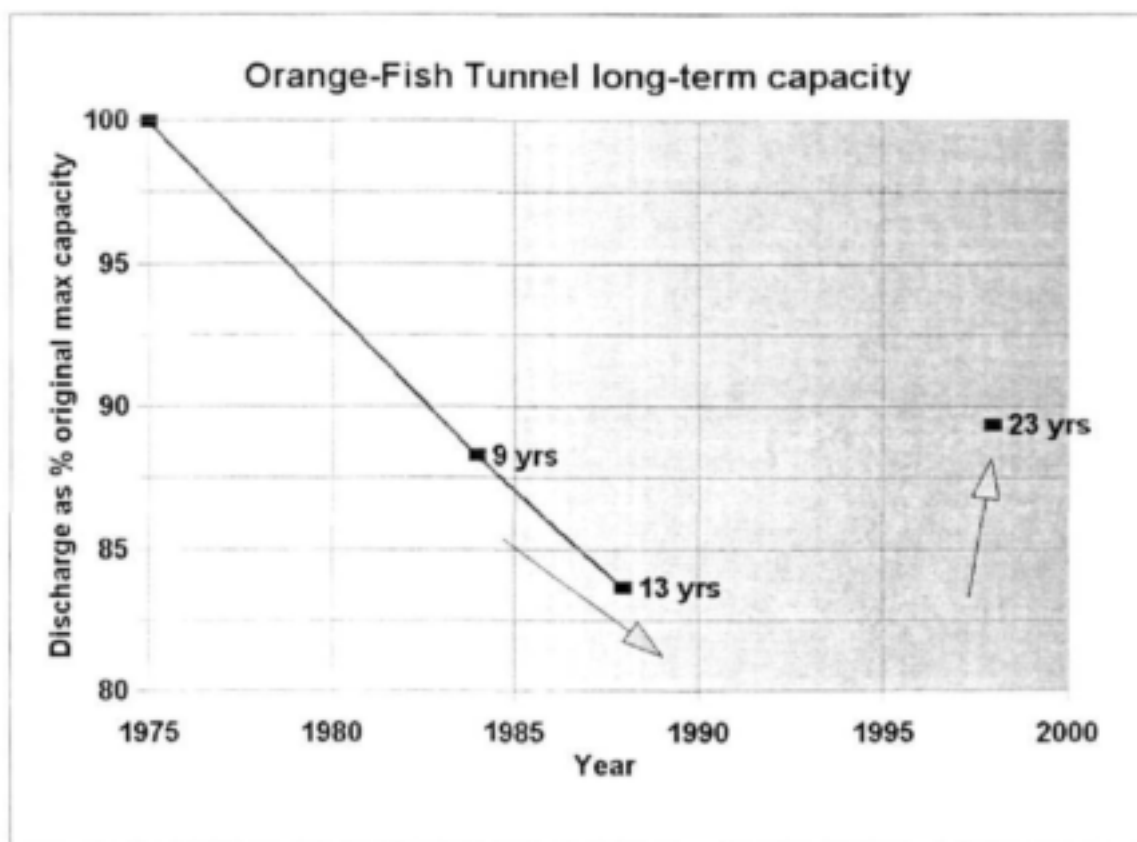


Figure 8.1.2-12 Orange-Fish discharge capacity change over time

e) Tunnel test 2000

HDPE pipes were installed down shaft 7a (near outlet), ending in the tunnel upstream of the shaft at 10 m, 160 m, and 310 m distances. The 150 m closest to the shaft was cleaned by high pressure rotating head water jets by DWAF as part of this study to investigate the effect of mechanical cleaning on roughness. The cleaned section was found rougher than the undisturbed tunnel.

f) Tunnel test 2001

During February 2001 a last flow test was carried out to assess what had happened to the cleaned sections roughness over time, and to flush the tunnel for a longer duration at a maximum flow of about 48 m³/s. The results are shown in Figures 8.1.2-13 and 8.1.2-14.

For the upstream part of the tunnel, there appears to be a definite decrease over time in roughness during the maximum flow testing. For the downstream part of the tunnel downstream of shaft 6, the roughness at high flow is very low.

they are consistently higher than for the rest of the tunnel. The very low roughness values of the 300-150 m section also agrees with the roughness from shaft 6 to 7a.

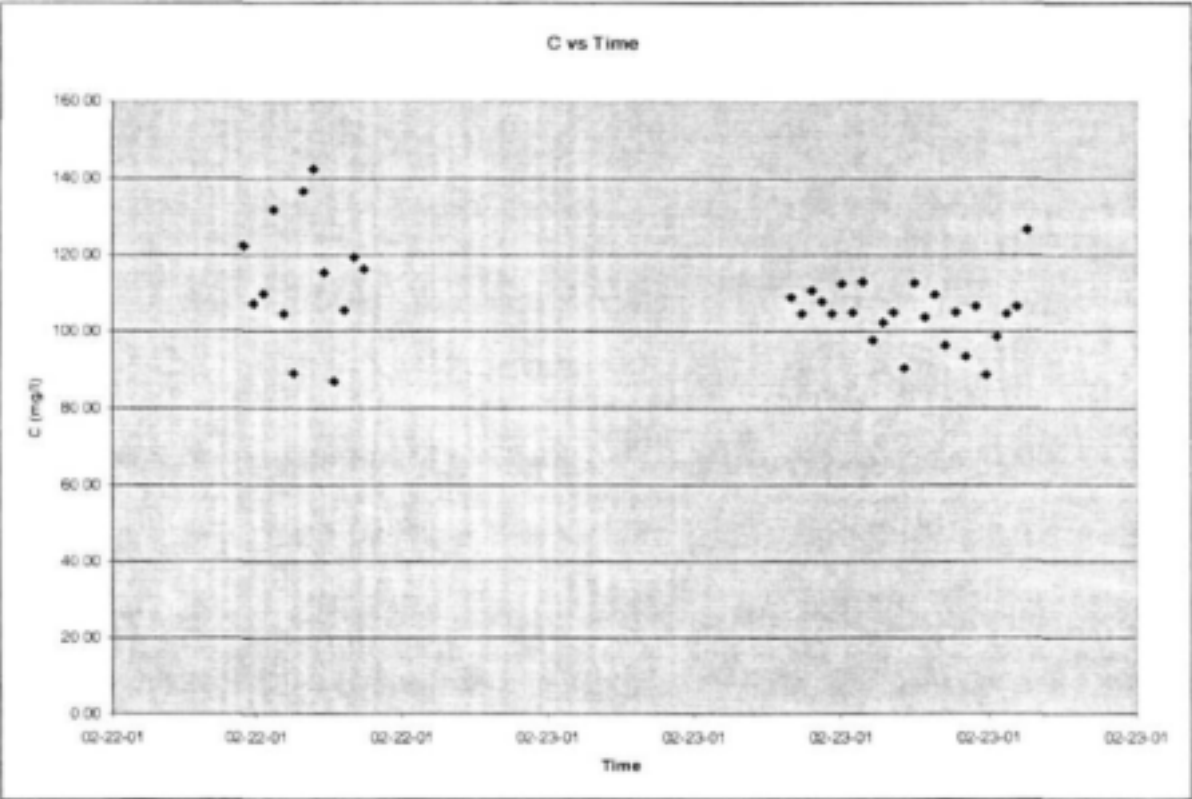


Figure 8.1.2-15 Orange-Fish Tunnel sediment transport 2001

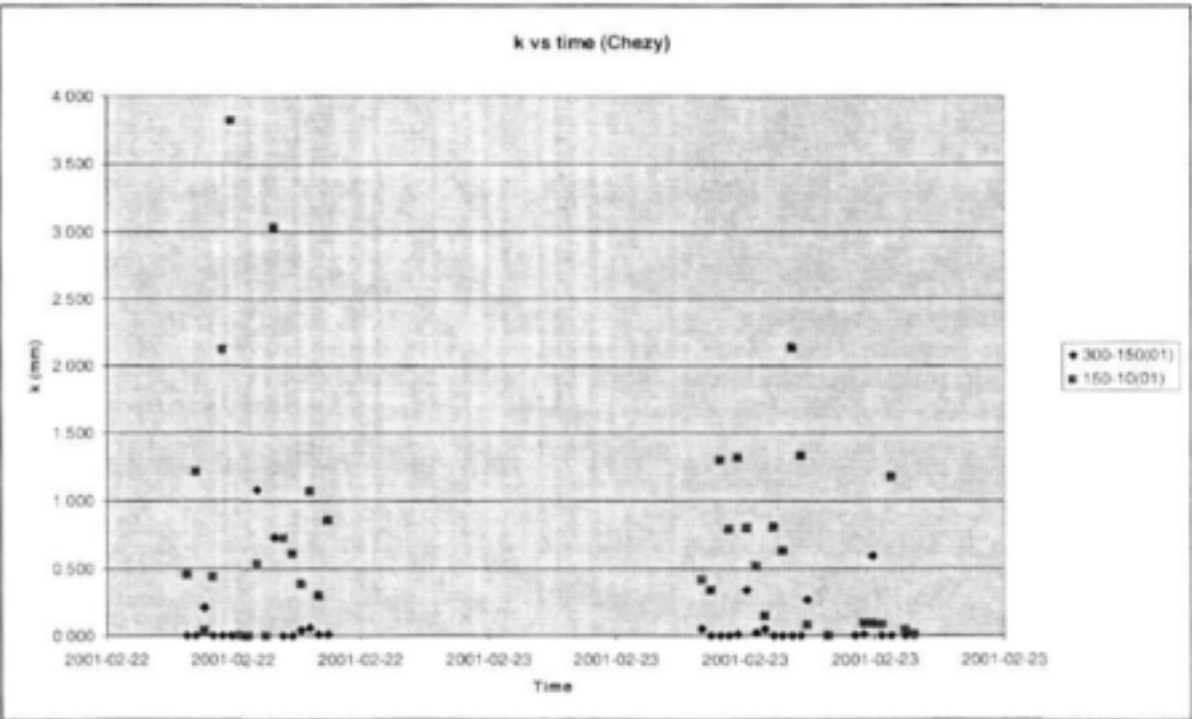


Figure 8.1.2-16 Orange-Fish Tunnel cleaned section roughness 2001

8.1.3 Theewaterskloof Tunnel experience

a) Tunnel system description

The Theewaterskloof Tunnel as referred to in this report, was completed in 1980. It consists of an intake at Theewaterskloof Reservoir near Villiersdorp, a 4.23 m diameter tunnel of about 12 km in length to the Berg River, a steel pipe section underneath the Berg River, and about 12 km tunnel from the Berg River crossing to the Jonkershoek shaft. This shaft controls the flow through 3 automatic 1.2 m needle valves that are regulated by water levels in Kleinplaas Dam, which is about 300 m from the Jonkershoek shaft. Kleinplaas Dam acts as balancing storage from where water is transferred underneath Stellenbosch Mountain through a 4 km tunnel which is connected to a pipeline that conveys the water to Cape Town water treatment plants. The tunnel system is a gravity system and is quite unique in that it transfers water in both directions: In winter the Banghoek and Wolwekloof shaft intakes along the tunnel, transfer Berg River catchment water to Theewaterskloof Reservoir, while in summer when the Berg irrigation and Cape Town water demands are high the water is transferred from Theewaterskloof Dam.

A layout of the tunnel is shown in Figure 8.1.3-1 and figure 8.1.3-2 shows a long section.

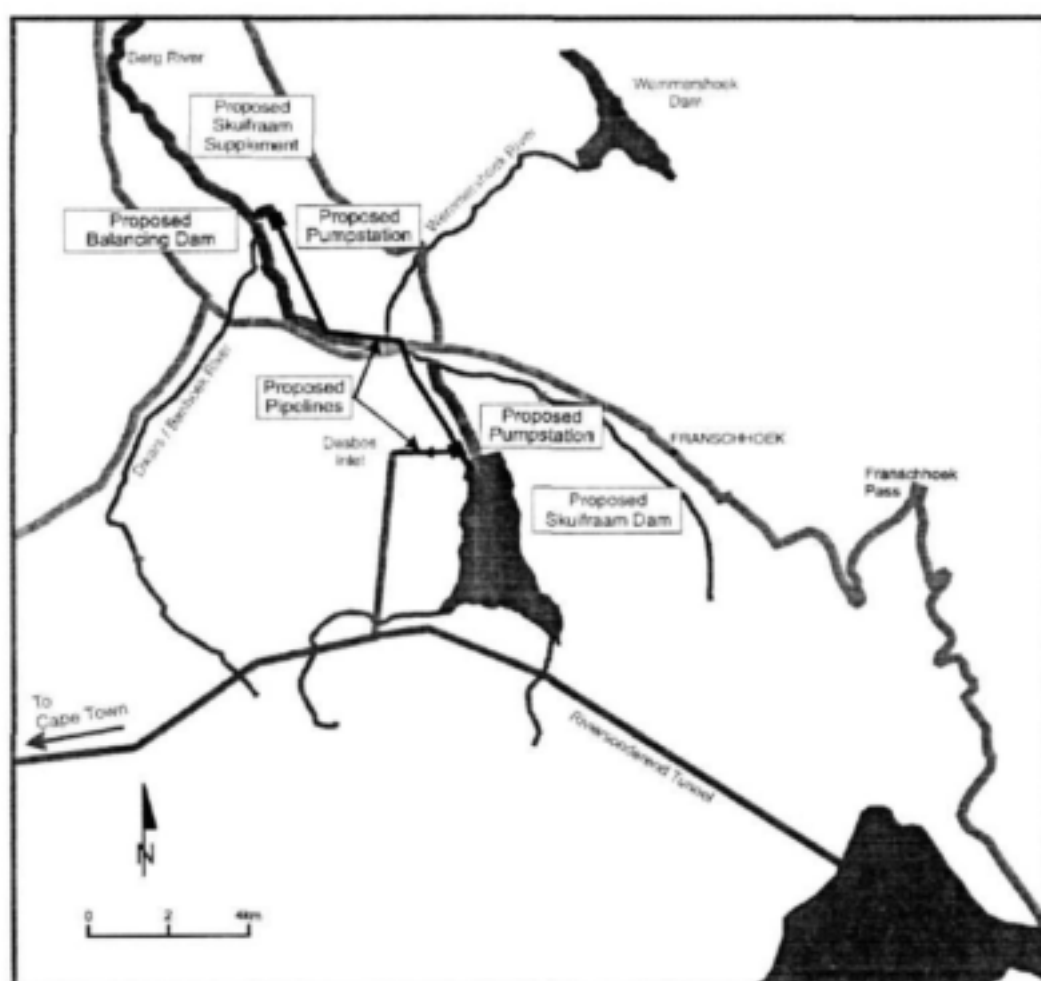


Figure 8.1.3-1 Theewaterskloof Tunnel layout

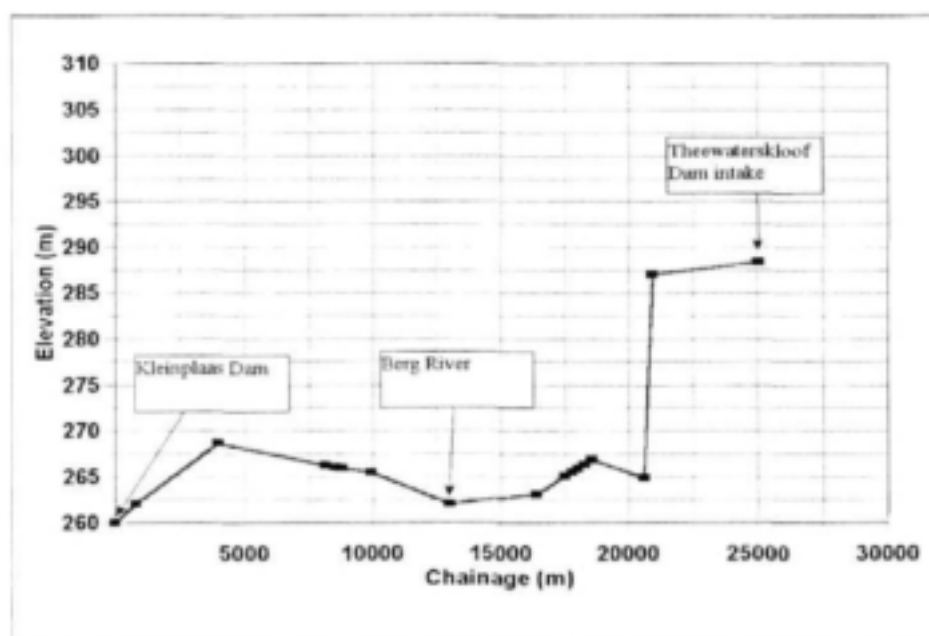


Figure 8.1.3-2 Longitudinal profile of the Theewaterskloof to Kleinplaas Dam tunnel

Between Theewaterskloof Dam and the Berg River East portal, the Charmaine pump station (used for dewatering) and the Charmaine air vent are located, on the Southern side of the Franschhoek mountain. The Berg River releases are made at the Berg River western portal. Between the Berg River and Jonkershoek shaft, we find the Wolwekloof vehicle entrance, the Dasbos tunnel junction, air chamber, Wolwekloof shaft and Banghoek shaft.

b) Tunnel inspection (1999)

During the winter of 1999 the tunnel was dewatered for inspection. The 28 km tunnel was inspected in two days on foot and by boat, with DWAF. An inflatable boat was lowered (Figure 8.1.3-3) at the intake shaft and pulled along the tunnel for 4 km to get past the deep section of the tunnel with standing water.

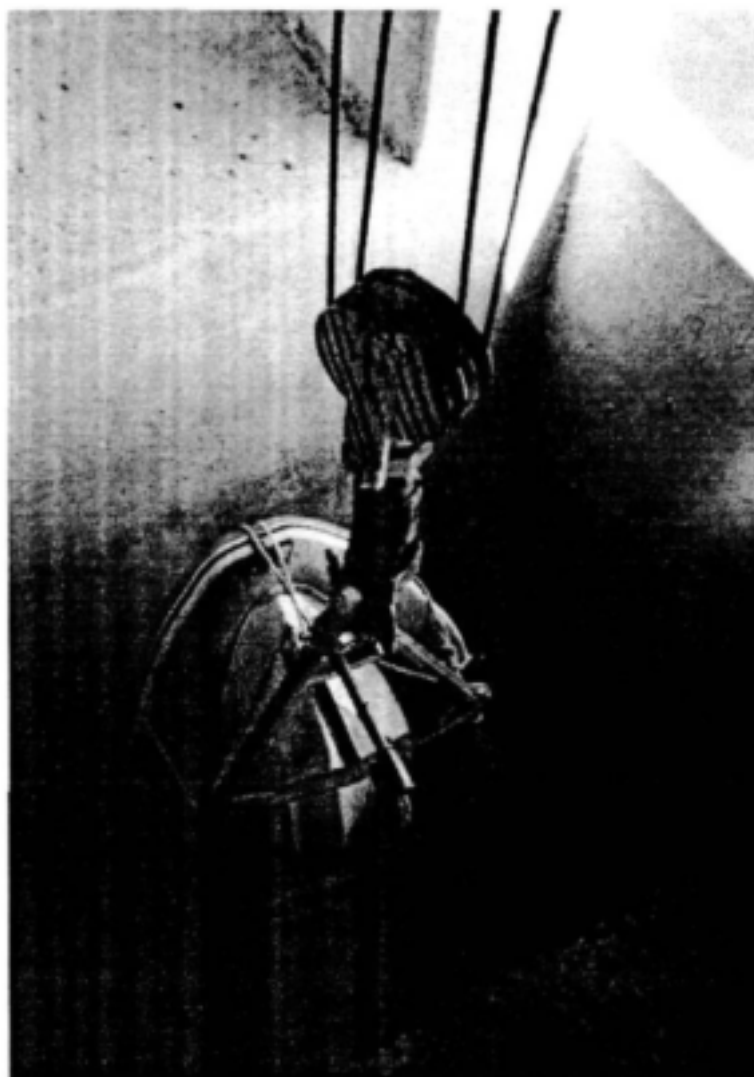


Figure 8.1.3-3 Lowering of the boat at the Theewaterskloof Dam intake shaft

Variable ageing characteristics were found along the tunnel:

- Between Theewaterskloof Dam and the Berg River where the tunnel has its largest diameter, the first part contains a slimy bacterial growth (figure 8.1.3-4) like a mass of worms covering the concrete about 10 mm thick. There are also many patches of lower strength concrete surface where severe erosion has occurred, in some places leaving long etch marks. Some erosion holes are quite deep (1.0 m) (Figures 8.1.3-5 and 8.1.3-6). These patches of poor quality concrete were caused by ground water intrusion problems experienced during construction.

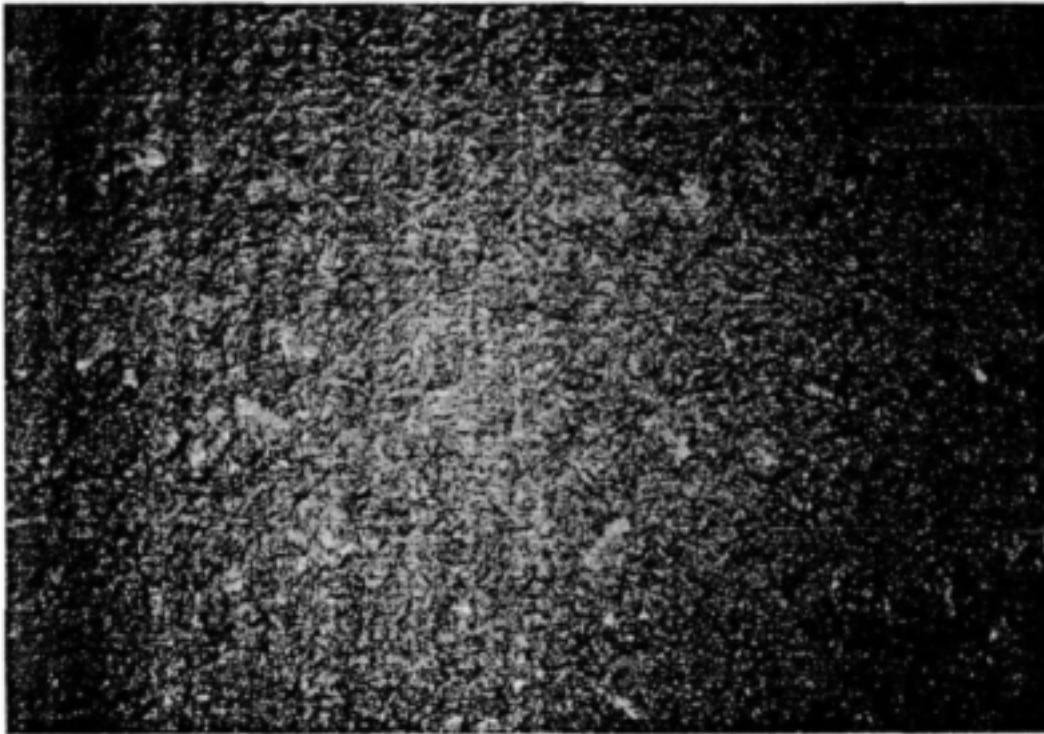


Figure 8.1.3-4 Bacteriological growth between the intake and Berg River, Theewaterskloof Tunnel



Figure 8.1.3-5 Deep scour hole in soft water corroded concrete, intake to Berg River, Theewaterskloof Tunnel



Figure 8.1.3-6 Deep scour in soft water corroded poor quality concrete, intake to Berg River, Theewaterskloof Tunnel

- Between Berg River and Banghoek, the sediment deposits are about 7 mm thick, consisting of fine sediment, with a cohesive nature and black colour. During tunnel emptying the sediment remained intact, similar to manganese deposits. In one part where a sediment strip fell from the wall, it removed the upper 3 mm of concrete with it, exposing coarse aggregate (Figure 8.1.3-7).



Figure 8.1.3-7 Exposed coarse aggregate after sediment removal due to emptying, Banghoek to Jonkershoek, Theewaterskloof Tunnel

Sand deposits up to 1 m deep were found on the tunnel floor, especially at the air chamber, originating from the Berg catchment intake shafts (Figure 8.1.3-8).

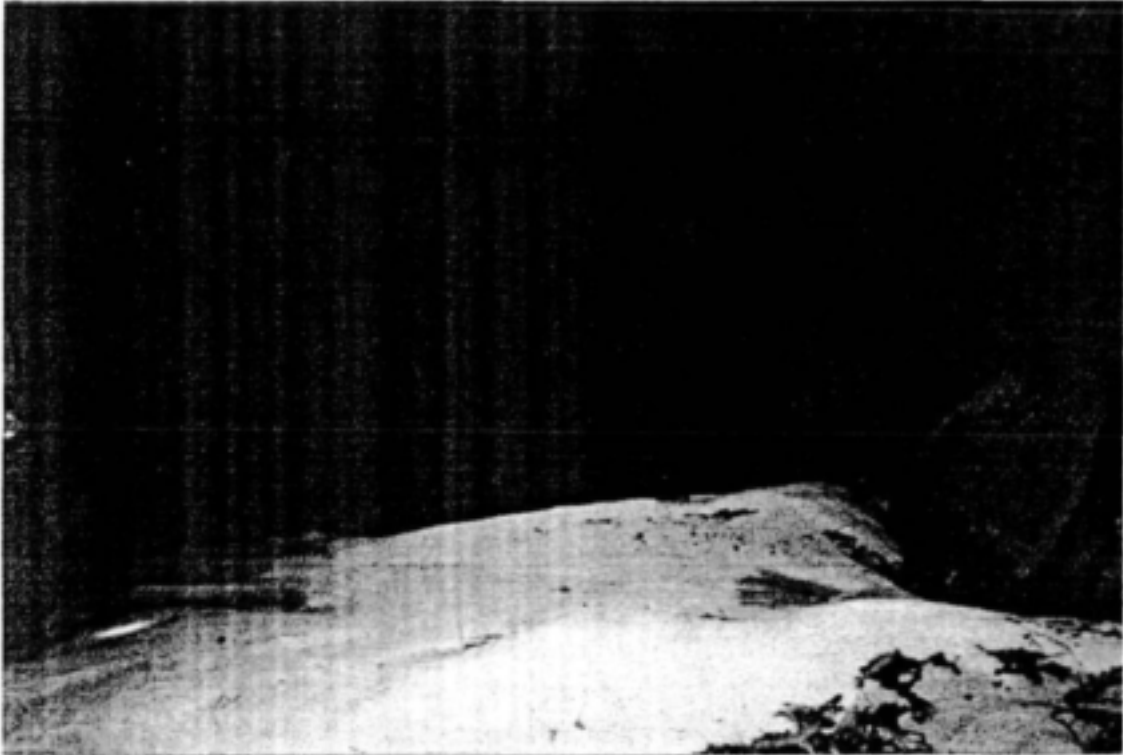


Figure 8.1.3-8 Sand deposition , Wolwekloof to Banghoek, Theewaterskloof Tunnel

- The upper layer of the concrete right through the tunnel was soft due to soft water corrosion and could easily be damaged (Figure 8.1.3-9).

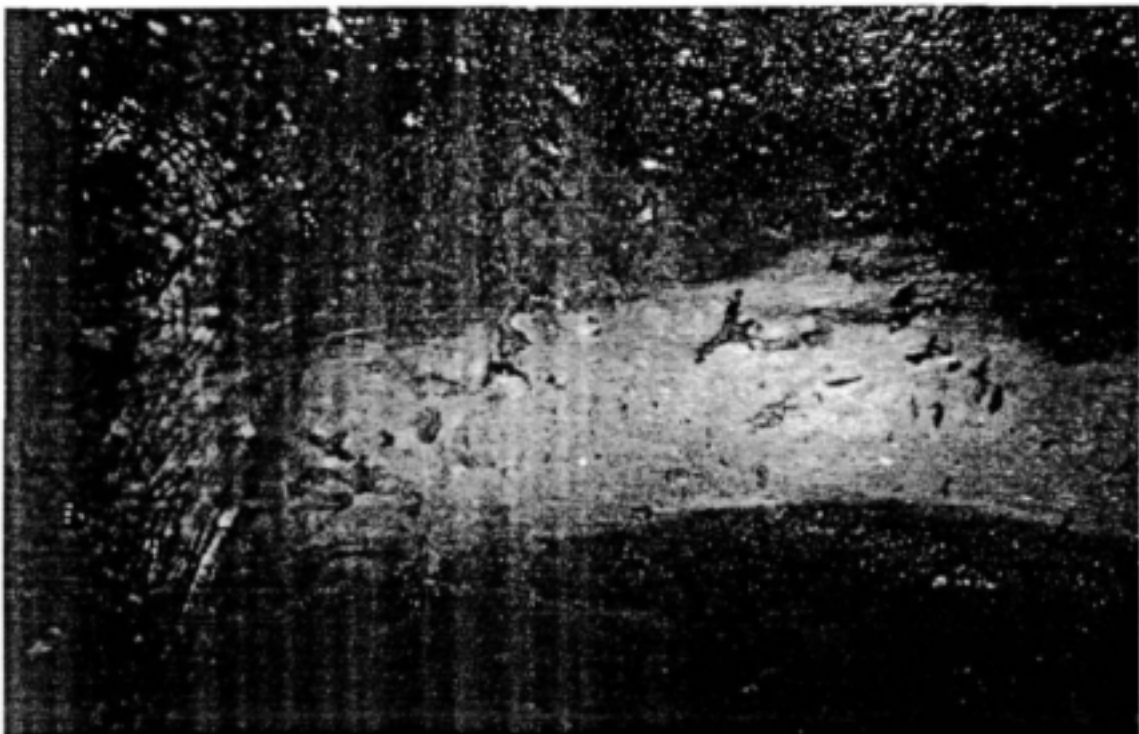


Figure 8.1.3-9 Typical soft water corrosion underneath sediment deposition, Theewaterskloof Tunnel

- There are two contraction sections in the tunnel where the roof of the tunnel has been lowered in the transition, without providing air vents. Patterns against the tunnel top clearly show that air has been trapped.

c) Flow test of 1999

During May 1999, the hydraulic roughness of the tunnel was determined. Tests were carried out at steady flows, increasing in steps to the maximum flow, and then decreasing again. No tests were carried out at commissioning to make ageing comparisons possible.

The test duration was limited due to water restrictions in the Western Cape. Discharge had to be calculated by reservoir routing of Kleinplaas Reservoir, taking into account its inflow, river outflow, Cape Town usage and storage change with time. Flow meters originally installed at the three needle valves at Jonkershoek did not work. About 30 staff members of DWAF were involved in the field work which was coordinated by the regional office.

The tunnel flows as calculated from the Kleinplaas reservoir balance are shown in Figure 8.1.3-10. Figure 8.1.3-10 also indicates sediment concentrations flushed out at Jonkershoek.

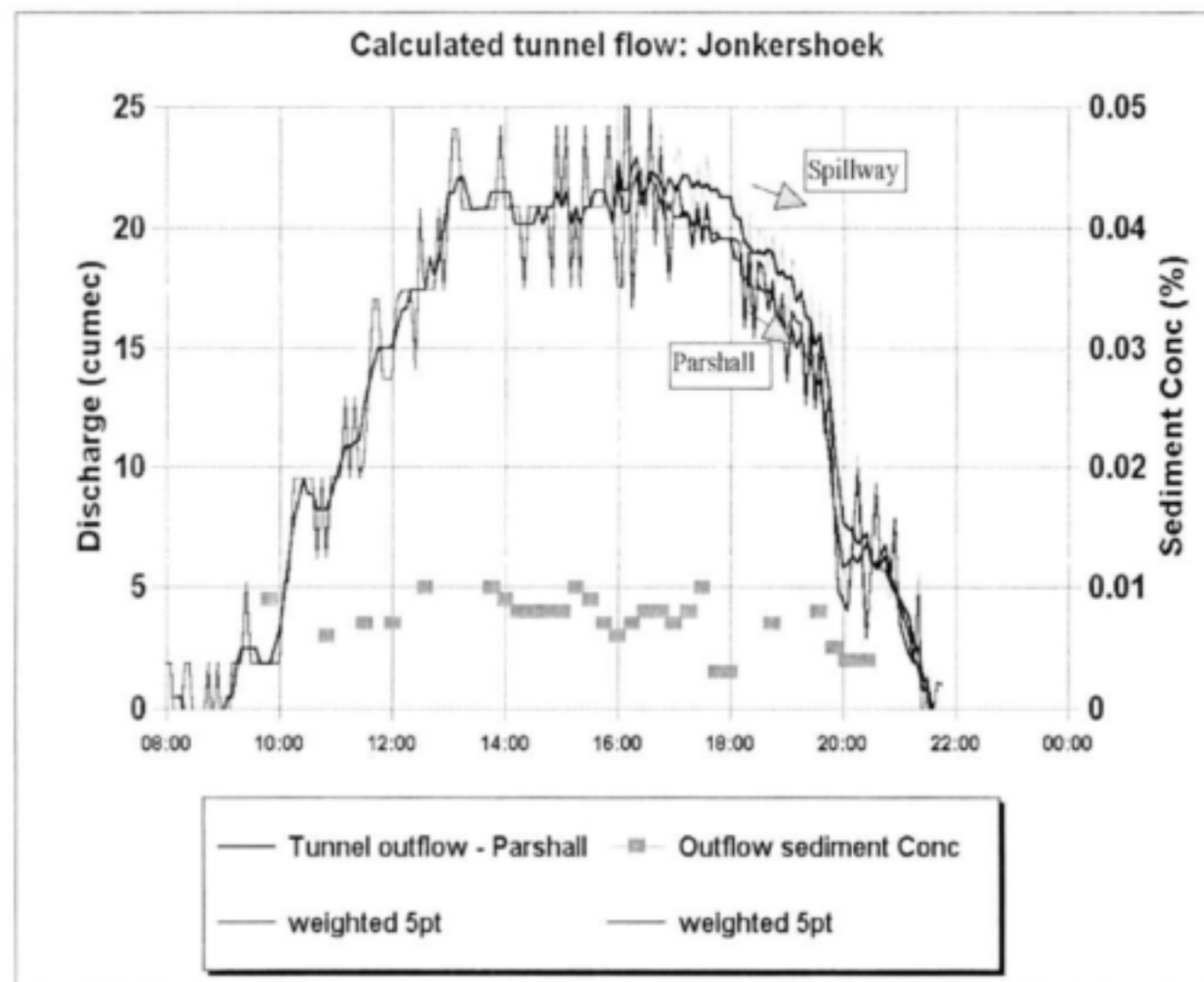


Figure 8.1.3-10 Tunnels flows and sediment concentrations during 1999 tests

The results of the hydraulic tests indicated that the Berg River to Jonkershoek tunnel sections had relatively low roughnesses: 3 mm, 0.9 mm and 0.8 mm for the Berg to Wolwekloof, Wolwekloof to Banghoek, and Banghoek to Jonkershoek Tunnels respectively. The tunnel diameter is a minimum at Jonkershoek.

In the Franschhoek Tunnel, the roughnesses were 20 mm, 10 mm and 40 mm, for the Theewaterskloof intake to Charmaine pump station shaft, to Charmaine air vent, and to Berg River East portal respectively.

d) Flow test of 2001

During 2001 another steady flow test was carried out, this time with acoustic-doppler flow meters mounted to each of the steel pipes at Jonkershoek to determine an accurate discharge. The test was carried out over four days, with the first and last days at about 67 % capacity, and the other two days at maximum flow (at the given Theewaterskloof Dam water level) (Figure 8.1.3-11). The flushing efficiency in terms of sediment being flushed out and possible reduction over time in roughness were investigated.

The tunnel roughnesses determined from the flow tests are shown in Figures 8.1.3-12 to 8.1.3-15. The Banghoek to Jonkershoek section has a roughness in the order of 1 mm, with most values less than 1.5 mm. The third day's roughness is slightly higher than that for day two, which is difficult to explain. The last day's roughness was slightly less than for day one, indicating possible cleaning by flushing. The flushing velocity was 1.4 to 2.0 m/s over the four days.

In the Wolwekloof to Banghoek Tunnel, the day one to day four decrease in roughness was from 1.7 mm to about 0.8 mm.

The Berg River to Wolwekloof readings vary too much to be reliable. The manometer used at the West portal could be the problem with air having been trapped in the pipe. The general roughness is less than 20 mm, which is possible due to the sand deposition, but seems too high compared with Wolwekloof to Banghoek.

From Theewaterskloof to East Portal at Berg River, the roughness varies between 20 and 30 mm, which was expected due to the concrete erosion and slime growth.

The sediment concentrations measured at the three pipes at the Jonkershoek shaft are shown in Figure 8.1.3-16. (OP in the legend indicates observed sediment concentration at Eastern Portal, Berg River.) The central pipe's concentrations are in line with those of the tunnel flows and were the highest. On the first day of tests, 81 tons of sediment were flushed from the 3.5 m diameter tunnel (from Berg River to Jonkershoek). On the next days the sediments flushed were 120 ton, 11 ton and 24 ton respectively. If the sediment had been removed evenly from the 12 km long tunnel, this would be equal to a layer of 2 mm average thickness. The sediment being flushed out was however mostly sand from the deposits at the tunnel invert.

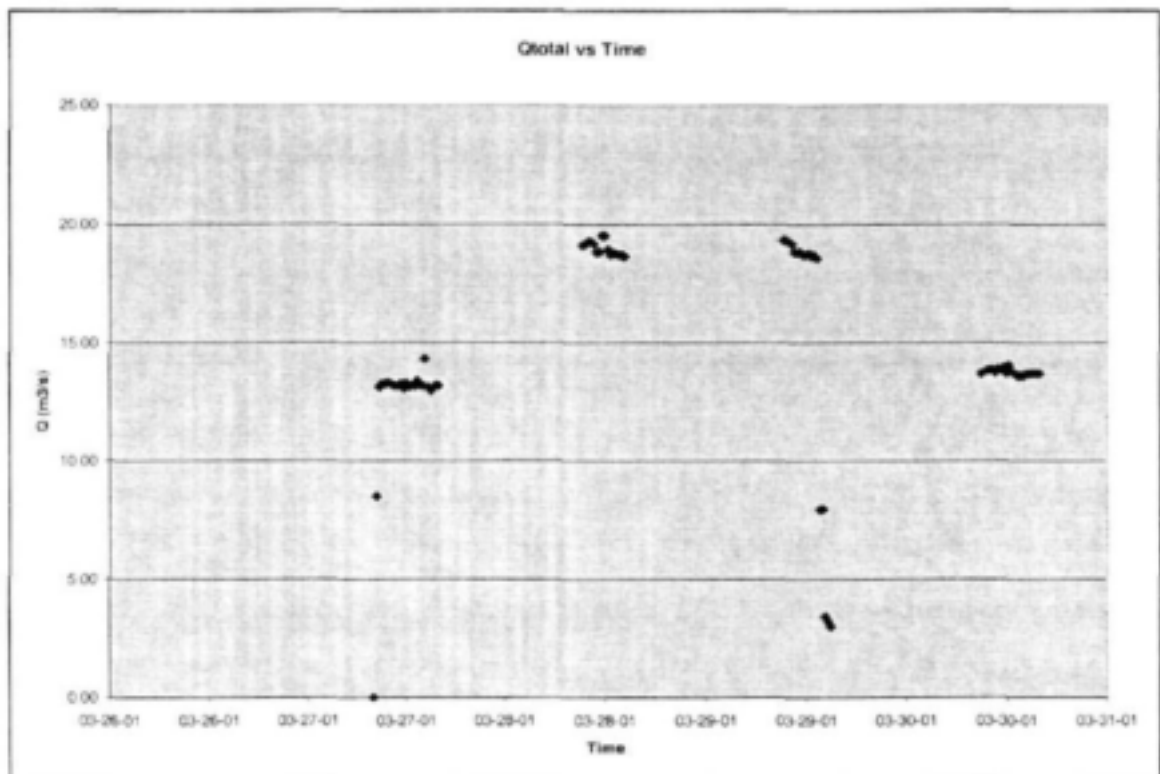


Figure 8.1.3-11 Test flows used

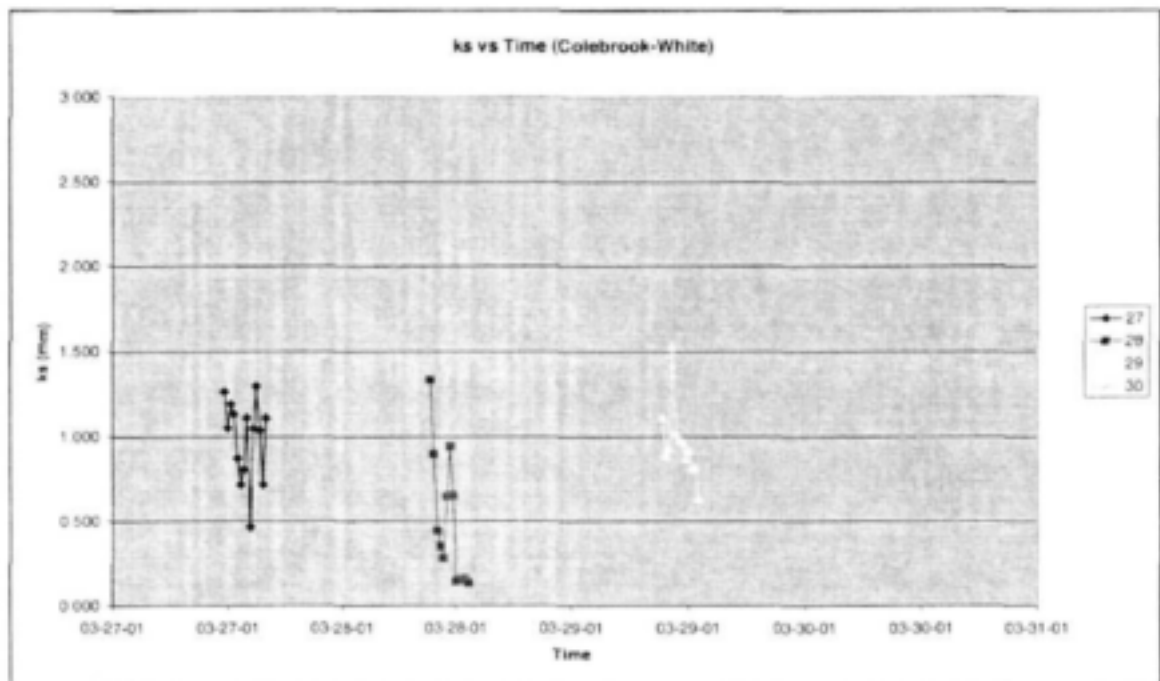


Figure 8.1.3-12 Banghoek to Jonkershoek roughness 2001

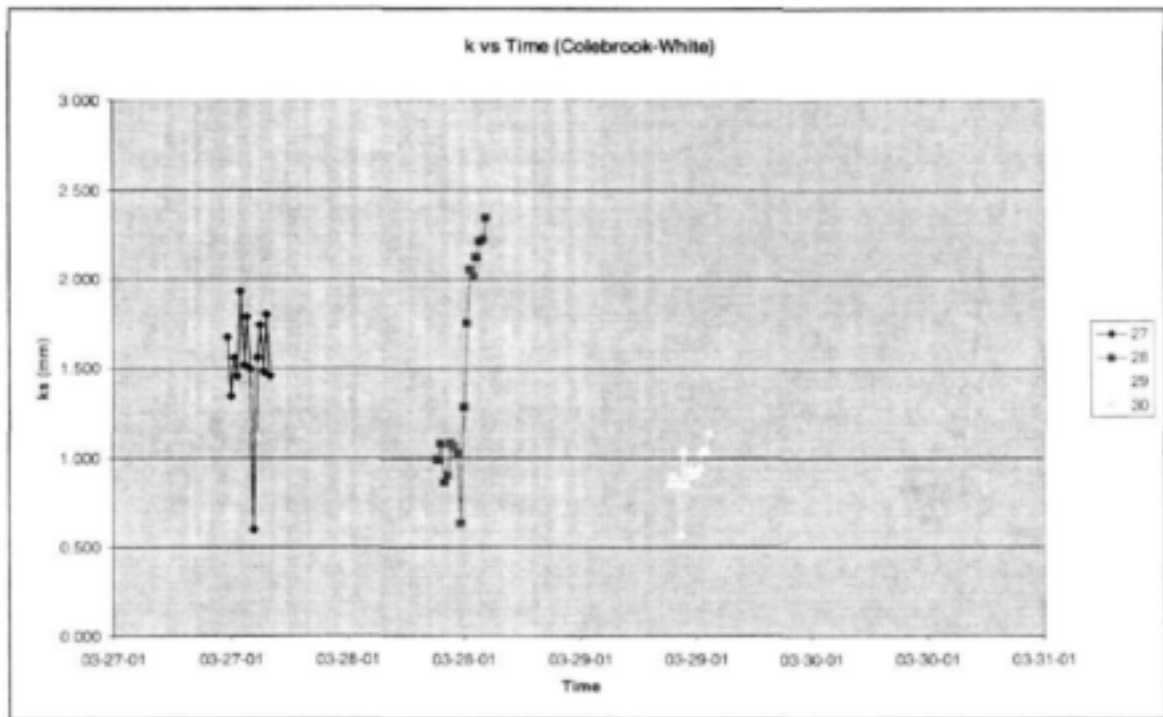


Figure 8.1.3-13 Wolwekloof to Banghoek roughness 2001

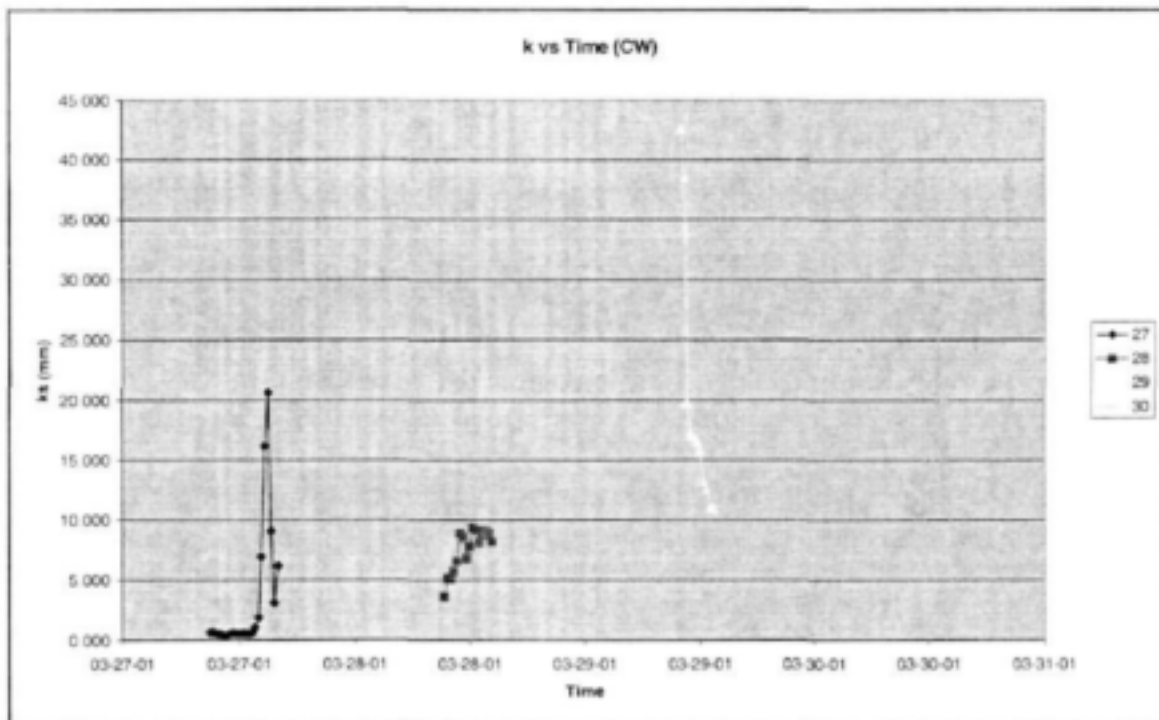


Figure 8.1.3-14 Berg River west to Wolwekloof roughness 2001

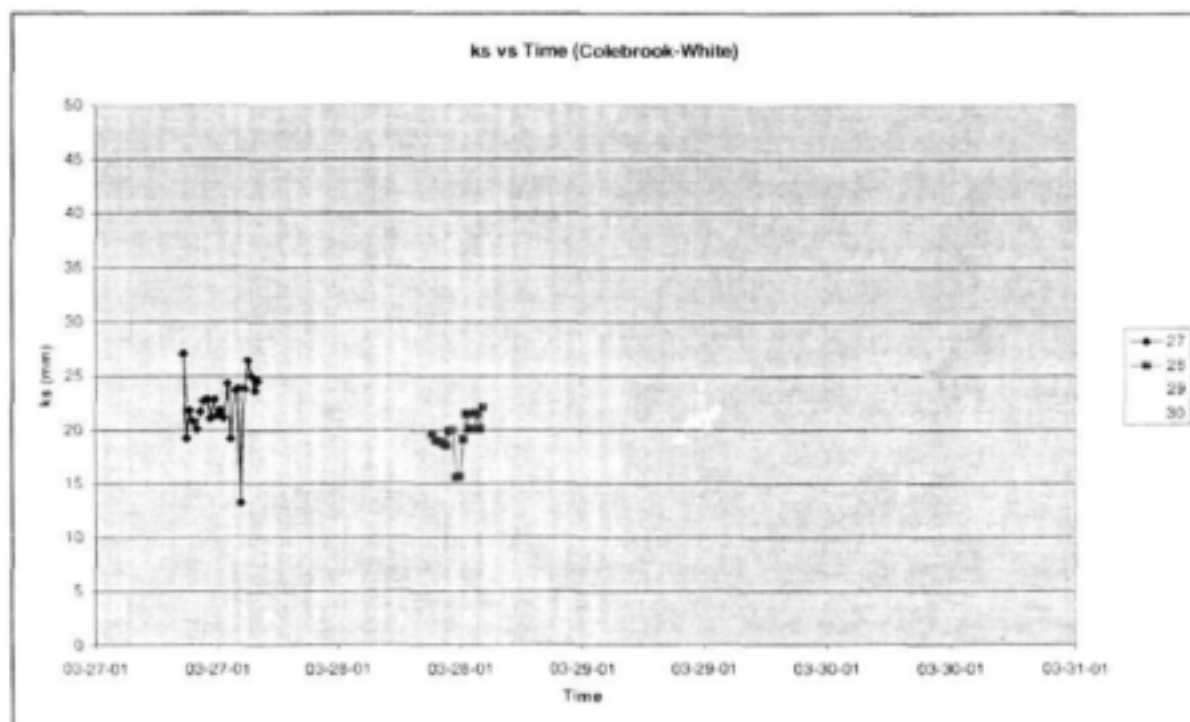


Figure 8.1.3-15 Theewaterskloof Dam to Berg East portal roughness 2001

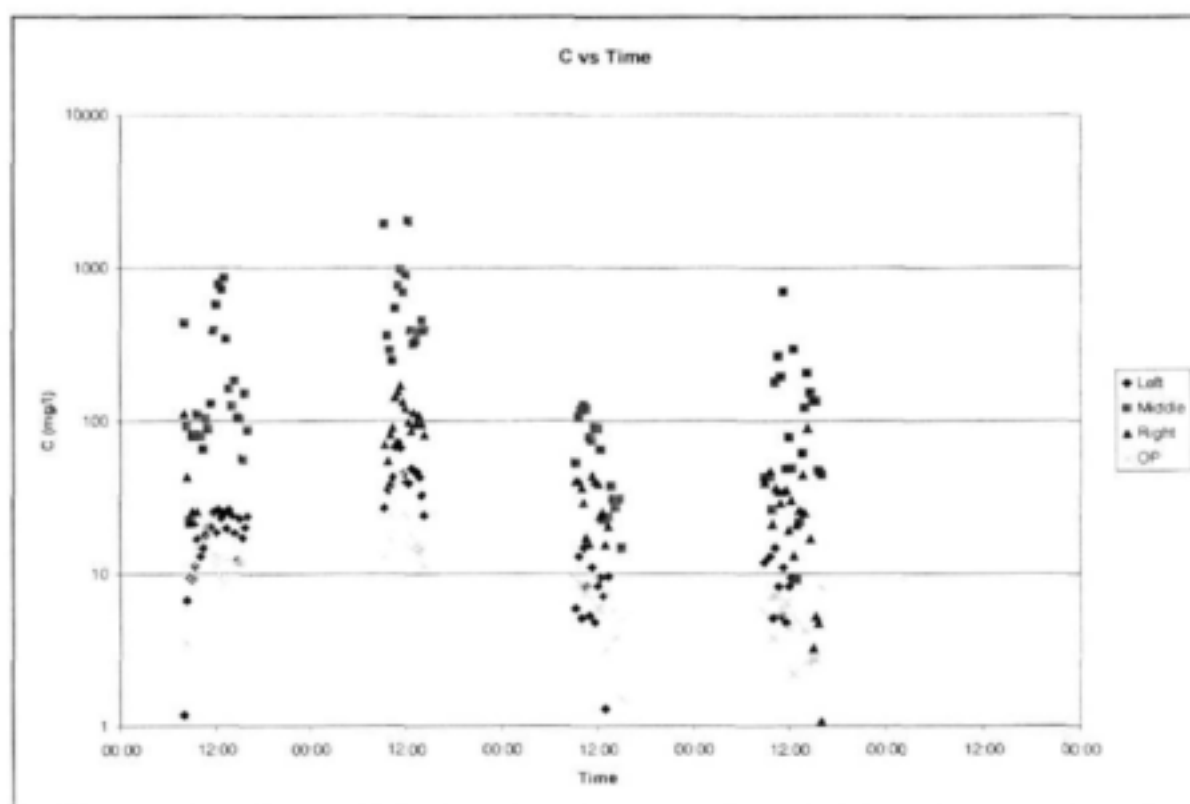


Figure 8.1.3-16 Sediment concentrations flushed out at Jonkershoek

In general the roughness values of the 1999 and 2001 tunnel tests are in agreement.

8.1.4 Roode Elsberg Tunnel experience

This tunnel links Roode Elsberg Dam with the Hex River valley. It is 5.2 km in length and 1.98 m in diameter, and is insitu concrete lined. The diameter is much smaller than typical tunnel diameters of today of the major water transfer schemes, but its capacity is much larger than the maximum water requirement. This tunnel has been in operation since the 1960s. Figure 8.1.4-1 shows Roode Elsberg Dam where the tunnel starts at the left bank, while the tunnel outfall at the Hex River Valley is shown in Figure 8.1.4-2.

The tunnel was inspected during 1999 by members of the project team and DWAF (Figure 8.1.4-3). The tunnel is relatively small, especially at the intake section (Figure 8.1.4-4). Prior to shutdown of the tunnel for inspection, the emergency gate had to be lowered which required special equipment (Figure 8.1.4-5).

Inspection during 1999 showed that due to relatively low operating velocities, a slimy soft sediment deposit had developed, about 10 mm thick. The surface of the sediment has a waviness which could be due to bedforms (Figure 8.1.4-6). Unfortunately, within the inner 3 km of the tunnel a lot of the construction equipment such as pipes, cables and railway tracks, had been left behind which would lead to high effective roughness (Figure 8.1.4-7).



Figure 8.1.4-1 Roode Elsberg Dam

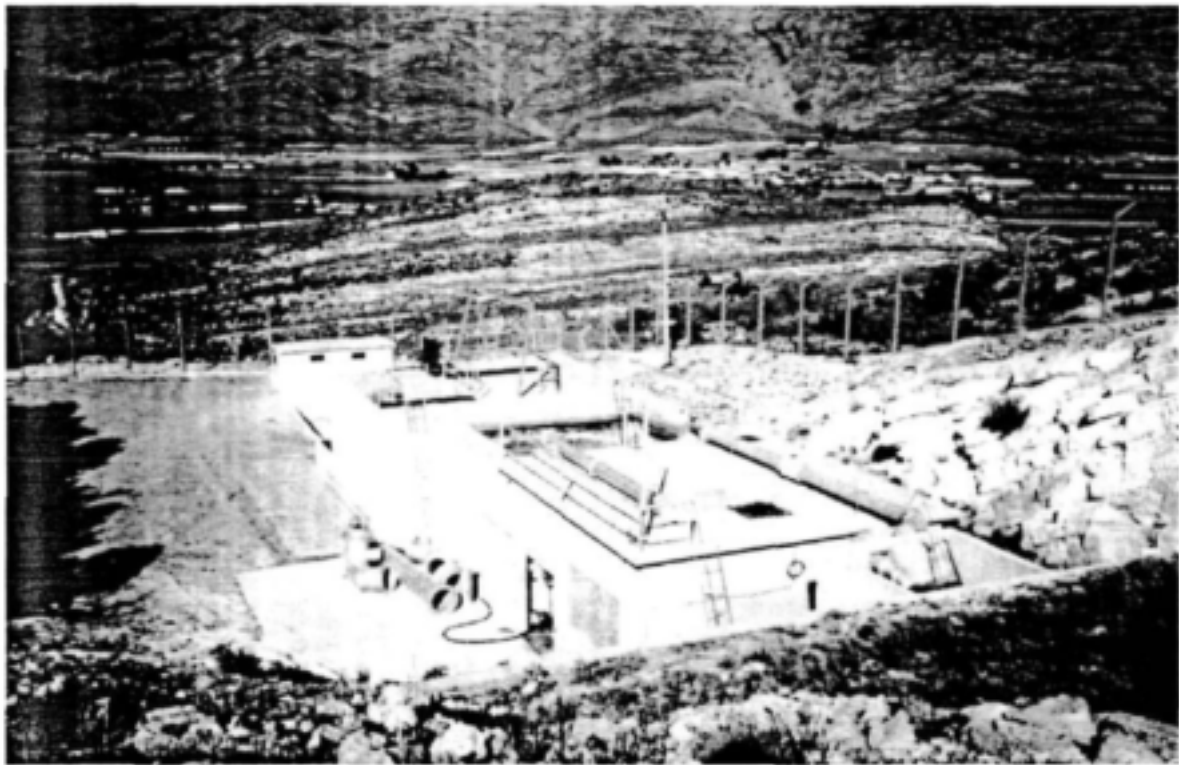


Figure 8.1.4-2 Roode Elsberg Tunnel outfall



Figure 8.1.4-3 Roode Elsberg Tunnel inspection team



Figure 8.1.4-4 Roode Elsberg Tunnel at intake



Figure 8.1.4-5 Lowering of the emergency gate at Roode Elsberg Tunnel

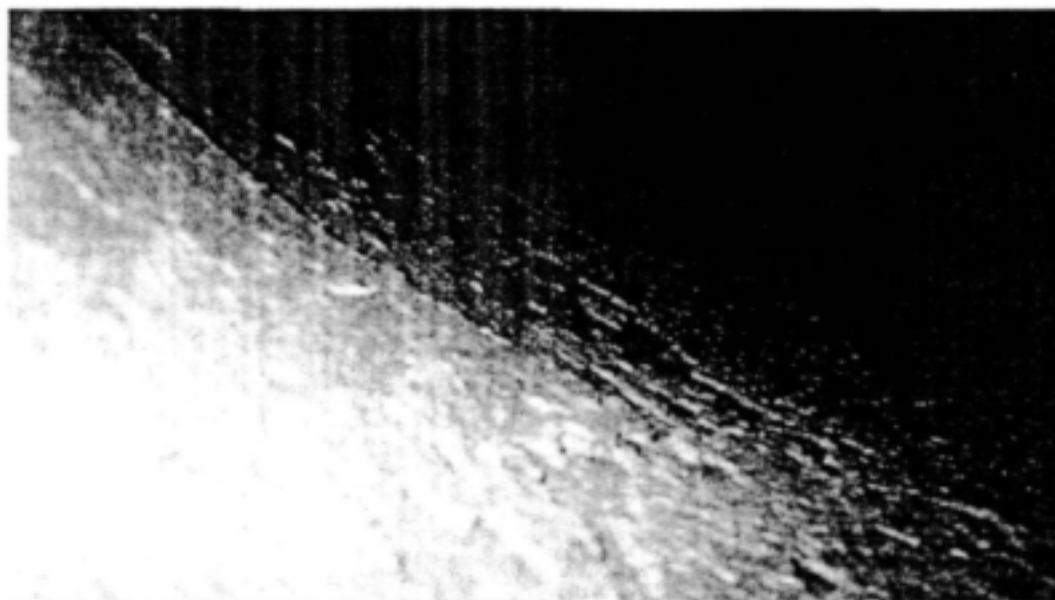


Figure 8.1.4-6 Sediment deposits in Roode Elsberg Tunnel (1999)



Figure 8.1.4-7 Construction equipment left in Roode Elsberg Tunnel

Steady flow tests were carried out on 20 September 1999 to determine the hydraulic roughness of the tunnel. The tunnel had been operated at relatively low flow velocities in the past at about 0.5 m/s maximum.

An HDPE pipeline was installed at the downstream end of the tunnel, connected to a tap at the tee of the bypass pipeline. The static water level was recorded and three steady discharges were tested: 1.0, 1.47 and 1.77 m³/s, with flows measured at the three electro-magnetic flow meters. A fourth discharge of about 2.4 m³/s was also tested by releasing water through the bypass at the outlet (Figure 8.1.4-8). The highest flow velocity tested was quite small at 0.76 m/s. The tunnel is however linked directly to the irrigation distribution system and it is therefore not possible to test higher flows. Spillage through the bypass is also a loss from the system and the test could therefore only be carried out when Roode Elsberg Dam was spilling and for a short duration.



Figure 8.1.4-8 Bypass pipe at Roode Elsberg Tunnel

The head loss in the tunnel could be measured when the bypass pipeline was not in operation and the observed data are indicated in Table 8.1.4-1.

Table 8.1.4-1 Roode Elsberg Tunnel field test data

Discharge (m^3/s)	Velocity (m/s)	Total head loss (m)	Hydraulic roughness k_s (mm)
1.0	0.32	0.29	1.4
1.47	0.48	3.15	200
1.77	0.57	6.85	390

While the Reynolds number of all the above tests were high and in the turbulent zone, the first steady flow at $1.0 \text{ m}^3/\text{s}$ was in the transitional zone and laminar boundary effects could have masked the boundary roughness effects. The low observed roughness could perhaps also be explained by the bedforms that are in equilibrium at this low typical discharge, minimizing the stream power. At the higher discharges the flow is fully rough turbulent and the hydraulic roughness dominates, giving values of 200 to 390 mm which is considerably higher than found at any of the other tunnels found in South Africa or internationally. Secondary losses at the intake and outlet of the tunnel, as well as $0.5x$ the kinetic energy head for construction debris in the tunnel, were taken into account in the determination of the friction head loss.

At higher discharges than the ones tested, it is expected that the hydraulic roughness will decrease as the sediment is scoured. The flushing test did scour sediment as indicated by the observed sediment concentrations, but at the discharge of $1.77 \text{ m}^3/\text{s}$, the sediment availability of the cohesive sediment became limited due to the high shear stress required to re-entrain the consolidated sediment (Figure 8.1.4-9). At $2.4 \text{ m}^3/\text{s}$ the shear stress at the wall of the tunnel was about 11.8 Pa. This value agrees with a

critical shear stress for re-entrainment of 12.2 Pa found in rivers with colloidal alluvial silt (Nalluri and Featherstone, 2001).

During the flow test of 4 h duration, about 5 ton sediment was flushed through the tunnel. The test duration was relatively too short to identify with reliability changes in hydraulic roughness associated with the flushing event, but the data seem to indicate that at 1.77 m³/s, the hydraulic roughness decreased from about 400 mm to 375 mm over a steady flow period of 45 minutes.

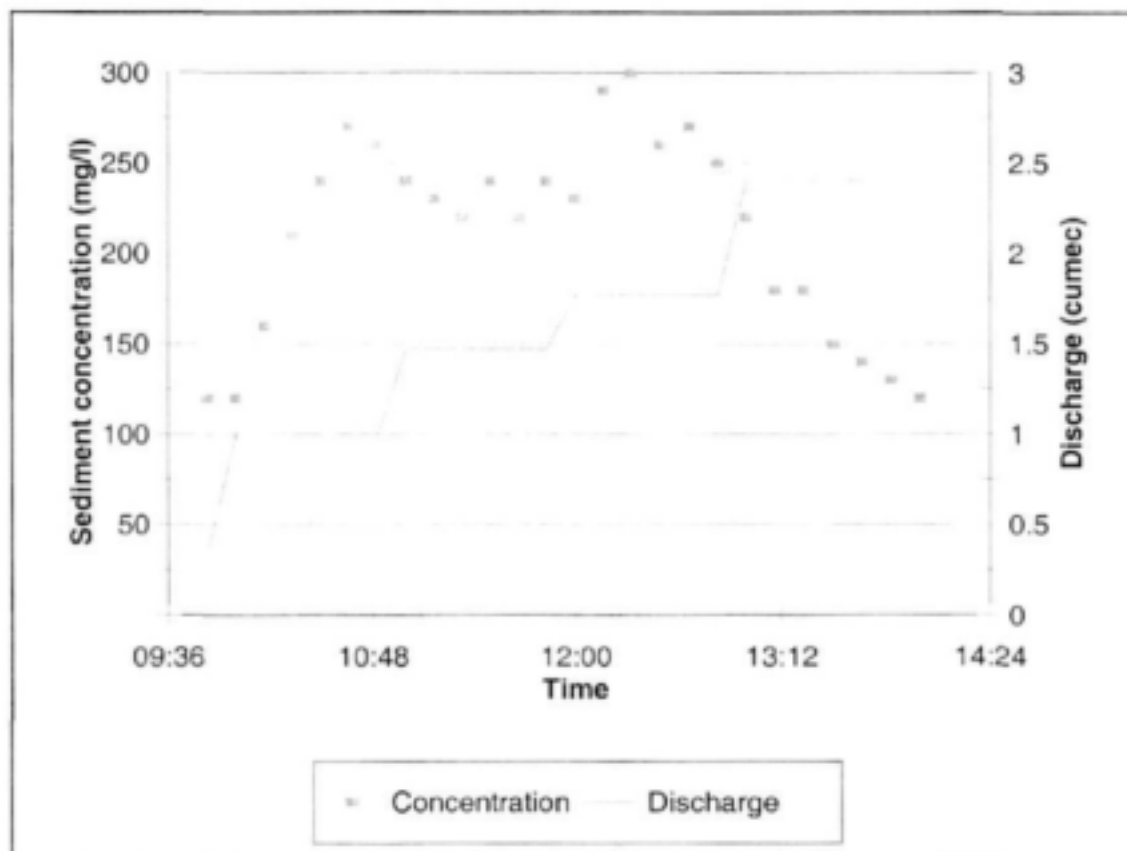


Figure 8.1.4-9 Sediment transport through Roode Elsberg Tunnel (1999)

8.1.5 Dasbos pipe laboratory test

a) Background

Flushing tests to try and remove sediment deposits in a laboratory are much easier to control and monitor than in the field. It is however not possible to rapidly age a pipe under laboratory conditions by sediment deposition when water quality, sediment characteristics and possible bacterial growth play a role. In the laboratory higher flow velocities and longer duration tests can be carried out than in the field, without the risk of damage to the prototype tunnel.

An aged pipeline connected to the Theewaterskloof Tunnel system was selected for flushing tests in the Hydraulic Laboratory of the University of Stellenbosch. This pipe is a 100 mm diameter AC pipe which has been in operation for 20 years since commissioning of the tunnel. The pipe was connected to the Dasbos Tunnel portal near Franschhoek, and supplied water for irrigation to local farmers.

Four 4 m lengths of pipe were carefully removed from the field, with the pipe ends sealed to maintain the moisture content inside, and transported by truck by DWAF to

Stellenbosch during 2001. Assembly of the pipe in the laboratory was carried out on the same day of delivery and flow tests started the next day.

The laboratory setup consisted of the 4 pipes placed horizontally on the floor, linked upstream to the laboratory clear water system by a bend, valve and vertical steel pipe. After slow filling of the pipeline, flow control was by means of a valve at the downstream end of the pipe, where the water discharges into a large container with a V-notch weir for flow gauging. The head loss was measured by means of a manometer. The upstream gauge point was located 5 m downstream of the pipe bend to ensure uniform flow conditions, with the second one 12 m downstream.

The test programme consisted of a series of steady flows, starting at low flow velocities and increasing to the maximum velocities that could be generated with the available 12 m laboratory head. After the maximum flow test, the same steps were repeated in reverse order. Each test was run until equilibrium hydraulic roughness was obtained. Following these tests some more high flow tests were carried out at longer durations.

b) Characteristics of pipe ageing prior to testing

The pipe had a uniform layer of black deposit, about 3 mm thick, with small bedform ripples closely spaced. This material had a sticky oily feeling and character, and it was difficult to damage its surface by hand. When water was applied it however became easier to remove the sediment layer.

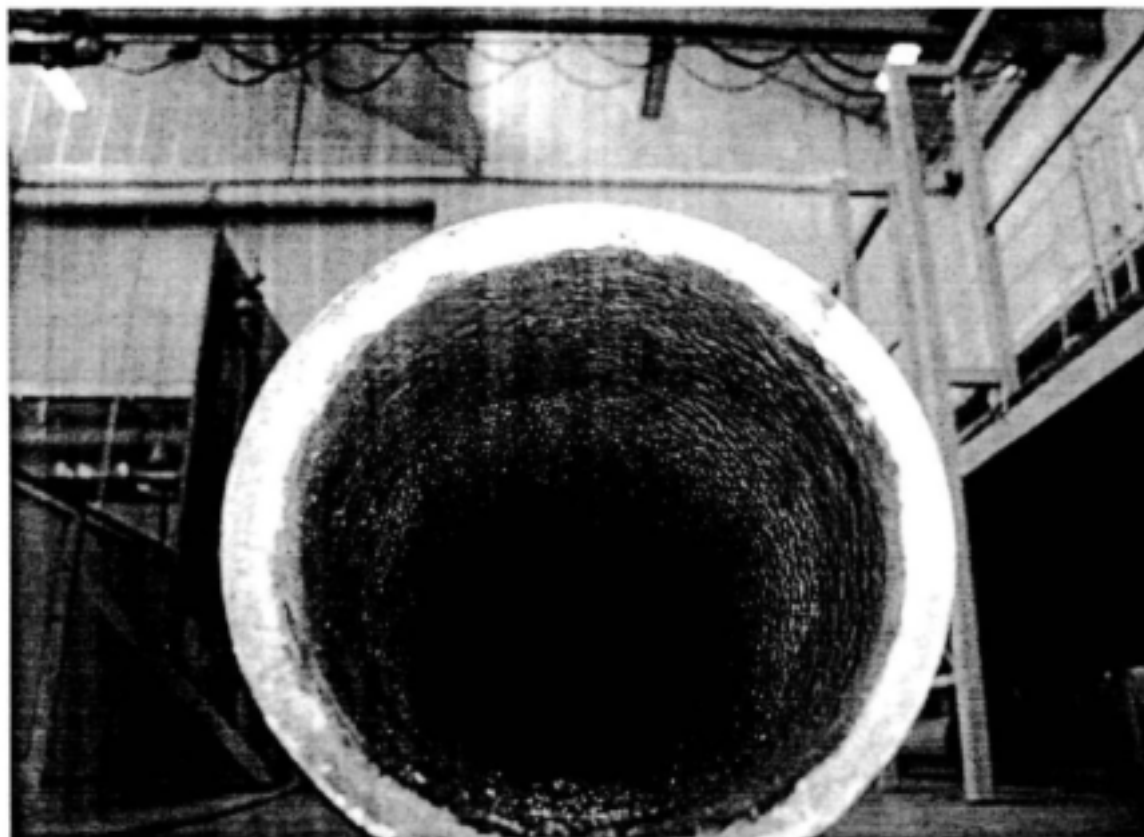


Figure 8.1.5-1 Dasbos pipeline before tests started

The nature of the sediment is similar to that found in the 5.5 m diameter Apalachia Tunnel, UK (Colebrook, 1958). "...became coated with a 5/16-in. thick layer of black mucilaginous material in a period of 10 years. This coating was uniform in depth and surface roughness from end to end of the tunnel. An analysis of the material showed

that it is primarily manganese and is considered that the deposition was probably due to the absorption of the manganese by alga polysaccharides."

A small amount of vegetable binding material was also found in the pipe sediment. Manganese deposits are common in Western Cape pipes. Manganese bacterial slimes have also been observed in many Tasmanian hydroelectric conduits (Brett, 1980).

The Dasbos tunnel normally has no flow, apart from the small irrigation release through the pipeline. Coarse sediment and even silt can therefore not be transported through the tunnel, and it is probably only organic and colloidal sediments that reached the test pipe. Bacterial action caused the manganese deposits, mixed with fine sediment. In the Berg to Jonkershoek Tunnel, similar black deposits have been observed, but with a more spongy softer character probably due to more fine sediment in the deposit, and it was not as sticky and difficult to damage as in the pipe. Field flow tests at 2 m/s on the main Berg-Jonkershoek Tunnel could not remove the cohesive deposited tunnel wall sediment, but the non-cohesive sand was flushed out.

c) Test results

The observed hydraulic roughness changes with time are shown in Figure 8.1.5-2. The key finding was that it was not possible to decrease the hydraulic roughness by high flow flushing, and in fact it increased slightly. At 0.9 m/s flow velocity at the start of the tests, $k_s = 1.6$ mm, while later after a maximum velocity of 3.2 m/s, the roughness k_s at 0.9 m/s was 1.8 mm. At 2.1 m/s, $k_s = 1.75$ mm and 2.1 mm before and after hydraulic flushing.

During the rising velocity phase of the tests at steady flows of 2.7 m/s and then 3.0 m/s, the roughness varied considerably, and this can possibly be ascribed to roughness adjustment of the sediment, or air in the manometer pipes.

The long duration test at 2.7 m/s indicated no further change in roughness from about 2.1 mm. Lower flow tests at 0.6, 0.9 and 1.2 m/s also indicated no change.

A chlorination test was carried out next, after testing it first on a small sediment sample from the pipe which seemed to indicate that the sediment would fluidise with some mechanical disturbance. The pipe was first filled with the HTH mixture and left standing overnight, before the test commenced. A high flow at 3.6 m/s for a duration of 3.5 h, however, did not change the roughness. The same procedure was also repeated with Jayes fluid, running it at 3.4 m/s for 4 h, and then at 4.3 m/s for 4 h. The Jayes fluid definitely decreased the roughness from about 2 mm to 1.8 mm. The decrease was however small and the visible effect on the sediment was negligible. The final roughness achieved was the same as at the start of the tests during the rising steady flow tests, at a velocity of 2.1 m/s.

After several months the sediment inside the pipe dried out and fell from the wall of the pipe on its own, as a powdery deposit.

The tests indicate that under certain conditions, ageing by sediment deposition where the cohesive sediment is well consolidated, consisting of manganese deposits, with bacterial involvement, cannot be restored by hydraulic flushing. One should therefore not operate the tunnel at first below its design capacity for long durations, since it would lead to a higher ageing rate and ultimate roughness, and if the sediment consists of manganese deposits and especially if bacterial growth is involved, high flow flushing or higher operating velocities later in the life of the tunnel might not be able to decrease the roughness. From the field inspections bacterial growth and manganese like deposits were observed in the Theewaterskloof Tunnel. More research is required to link the biological character of the deposited sediment to the water quality.

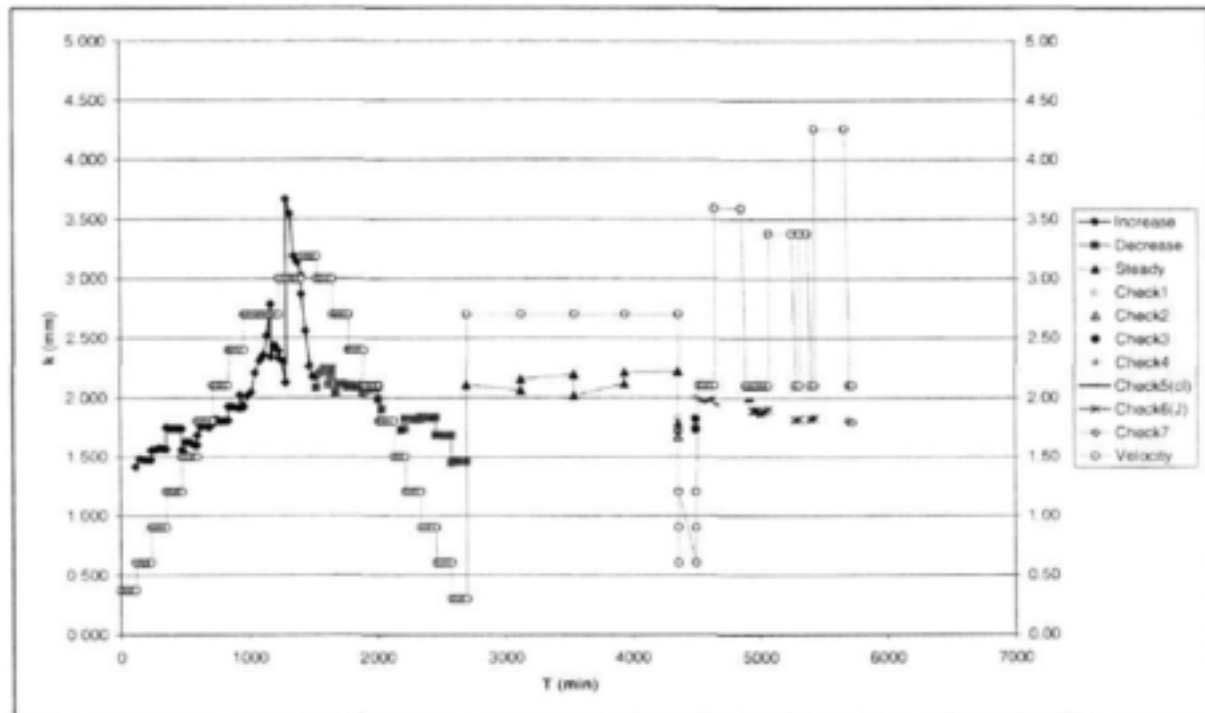


Figure 8.1.5-2 Dasbos pipe flushing hydraulic roughness test results

Finally, a 0.3 m section of pipe was cut in half for inspection after testing and to clean one half mechanically by hand using a brush, as shown in Figure 8.1.5-3. The cleaned surface (pipe on the left hand side in figure) was found to be relatively rough and did not resemble the original smooth finish of the pipe. Even if hydraulic flushing is able to re-entrain the sediment, it is doubtful whether a final roughness of below 0.6 mm can be achieved.

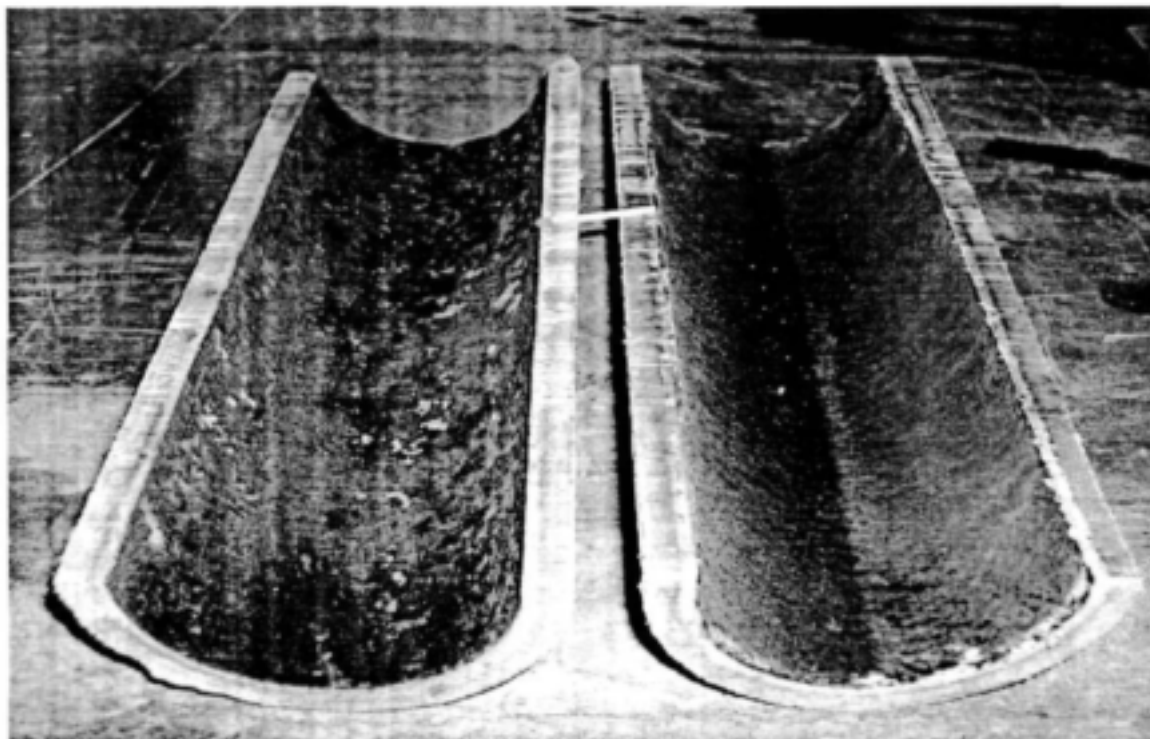


Figure 8.1.5-3 Dasbos pipe after testing and cleaning

8.2 Mechanical cleaning

8.2.1 USA experience

The Metropolitan Water District of Southern California (MWD), USA, operates the 400 km long Colorado River Aqueduct. Construction on the concrete lined aqueduct was completed in 1941. The tunnels have the shape of an inverted horseshoe, 4.9 m high, and are designed for free surface flow conditions. The water is not aggressive to concrete.

According to Kelly (1998), MWD periodically uses a tunnel cleaning machine to remove calcium build-up on the tunnel walls. The tunnel cleaner is an old front loader from which the bucket was removed and on which a framework and brushes were installed. The bristles of the brushes are spring steel and were found quite effective in removing the calcium buildup. No recent specific hydraulic measurements have been carried out to verify the cleaning efficiency, but a visual observation was that the before and after conditions were similar to the difference between 40 and 150 grit sandpaper. This corresponds to a water level drop of 0.25 m in the tunnel after cleaning.

Horowitz and Lee (1971) published an article on their findings of tunnel cleaning efficiency based on the MWD experience. Manning n-values were found to vary between 0.0127 to 0.013 before cleaning, while after three passes of tunnel cleaning, the hydraulic roughness measured as Manning n improved to 0.0113 to 0.0119. This relates to an average discharge capacity increase of 11 % (5 m³/s) from 46 m³/s to 51 m³/s for the Colorado tunnels.

Without cleaning of the MWD tunnels, hydraulic roughness measured as Manning n values quickly increased to 0.015. Photographs of the tunnel cleaning machine are shown in Figures 8.2.1-1 to 8.2.1-4.

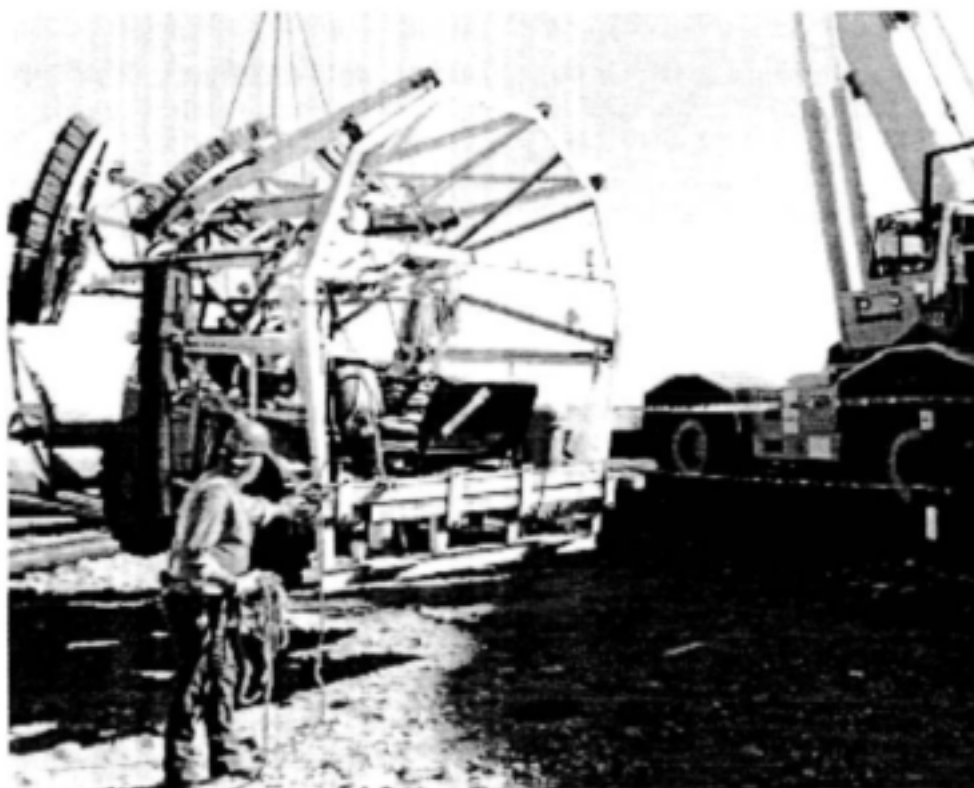


Figure 8.2.1-1 Tunnel cleaning machine before lowered down shaft into tunnel

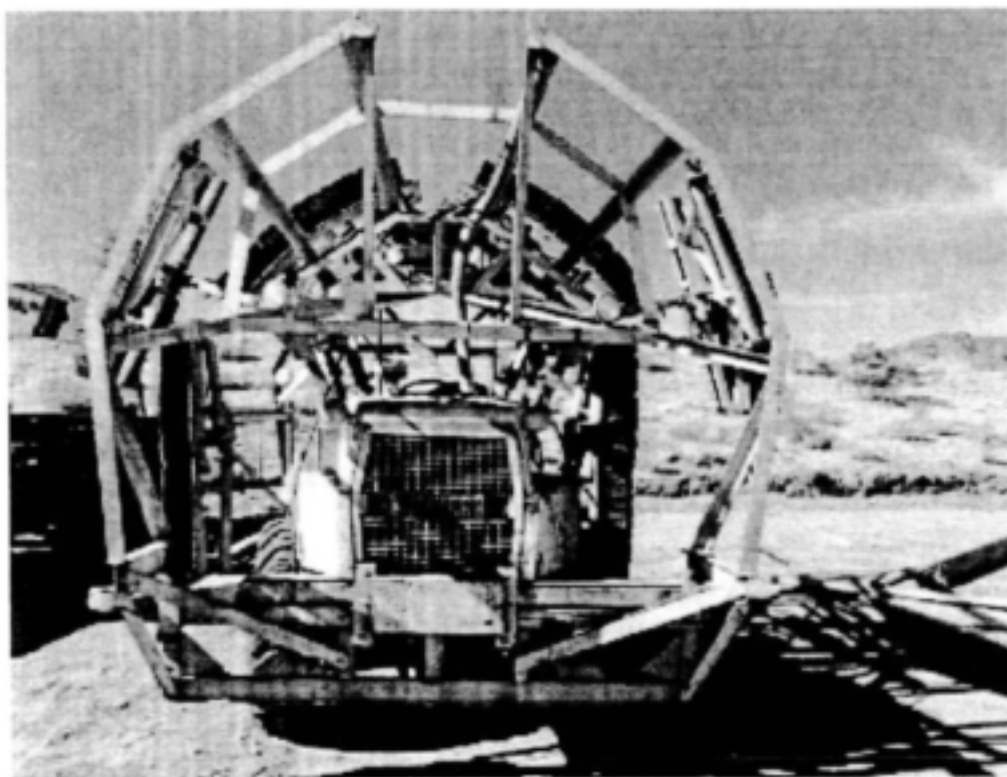


Figure 8.2.1-2 Rear view of tunnel cleaner

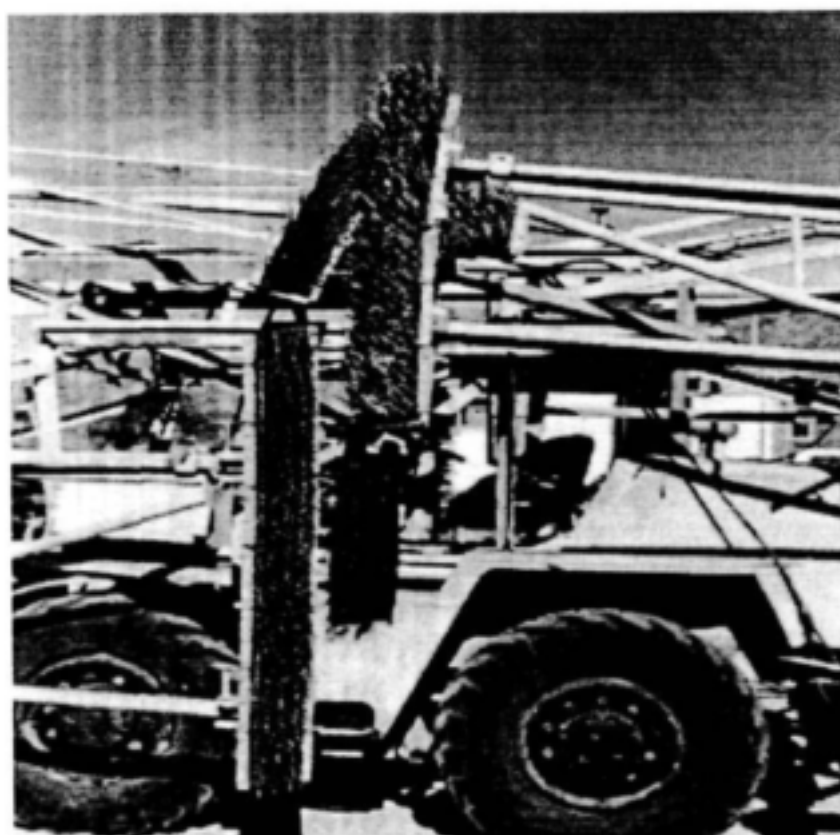


Figure 8.2.1-3 Side view of cleaner with steel brushes

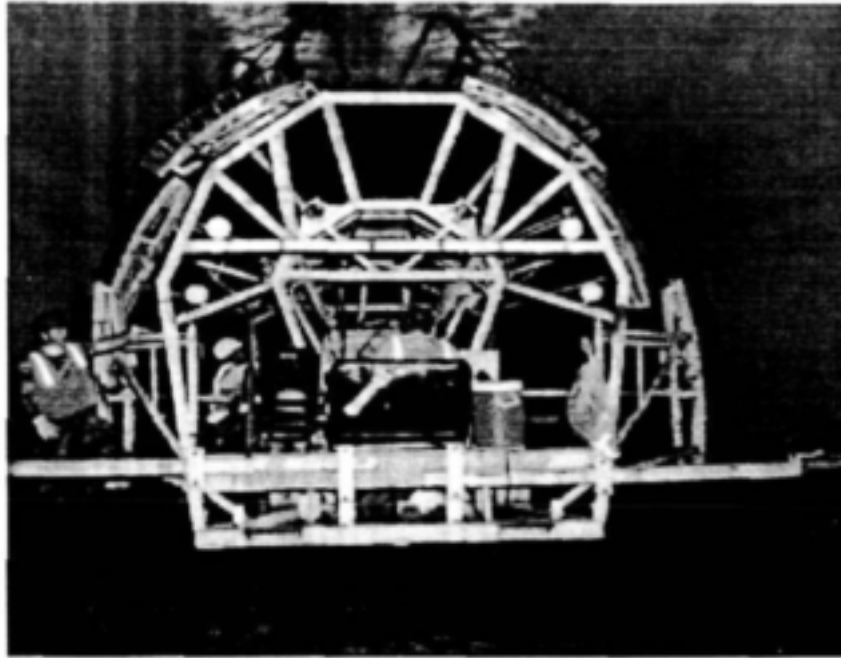


Figure 8.2.1-4 Tunnel cleaning machine inside Colorado MWD tunnel

8.2.2 Reidol Tunnel, Wales

At commissioning the Manning n value of the concrete tunnel was assumed to be 0.0125. Twelve years later, after tunnel cleaning, the n value was measured as 0.0134. Eighteen months later it was measured at 0.0141, indicating the effects of renewed deposition.

8.2.3 Tarraleah siphon, Tasmania

Brett (1979) reported on the cleaning of two of the 1.93 m siphons after 5 years of operation and associated change in roughness. The pipe consists of steel and mostly woodstave. The roughness dropped from 0.46 mm before cleaning, to 0.11 mm after cleaning in 1964. Within 6 months, however, the roughness increase linearly to 0.33 mm, and over the next 6 months at a slower rate during winter, to reach a roughness of 0.46 mm at the end of 1965 and the ageing probably continued after that to reach the pre-cleaning roughness within two years after cleaning.

8.2.4 Tarraleah No. 1 canal valley crossing, Tasmania

The valley is crossed by twin steel pipes 2.26 m in diameter and each 364 m in length. They were originally coated with horizontal retort tar in 1938. Some cleaning and partial reconditioning was done in 1952 and 1953. Late in 1966 the first measurements showed No.1 pipe to have a roughness $k_s = 1.88$ mm, while No. 2 pipe had a roughness of 1.08 mm. No. 2 pipe was then closed for cleaning and painting with a coal-tar epoxy. No. 1 pipe was not cleaned or painted until 1974. Head loss measurements taken during since 1966 have yielded the results shown in Figure 8.2.4-1. High and low k_s values indicated in the figure are the highest and lowest readings during each year and the fluctuations are due to seasonal growth variation. There is no explanation why the loss in No. 1 pipe is higher than in No. 2 pipe (Brett, 1979).

Where canal sections of the Tarraleah transfer system were cleaned, deterioration of the concrete surface was found with loss of cement and fines leaving the dolerite aggregate proud after the growth is cleaned off. Brett (1979) also states: "It has been

found economic to manually scrape the surface of the canal to remove growth on occasions to restore capacity, but this has been only temporarily effective because of the rapid regrowth".

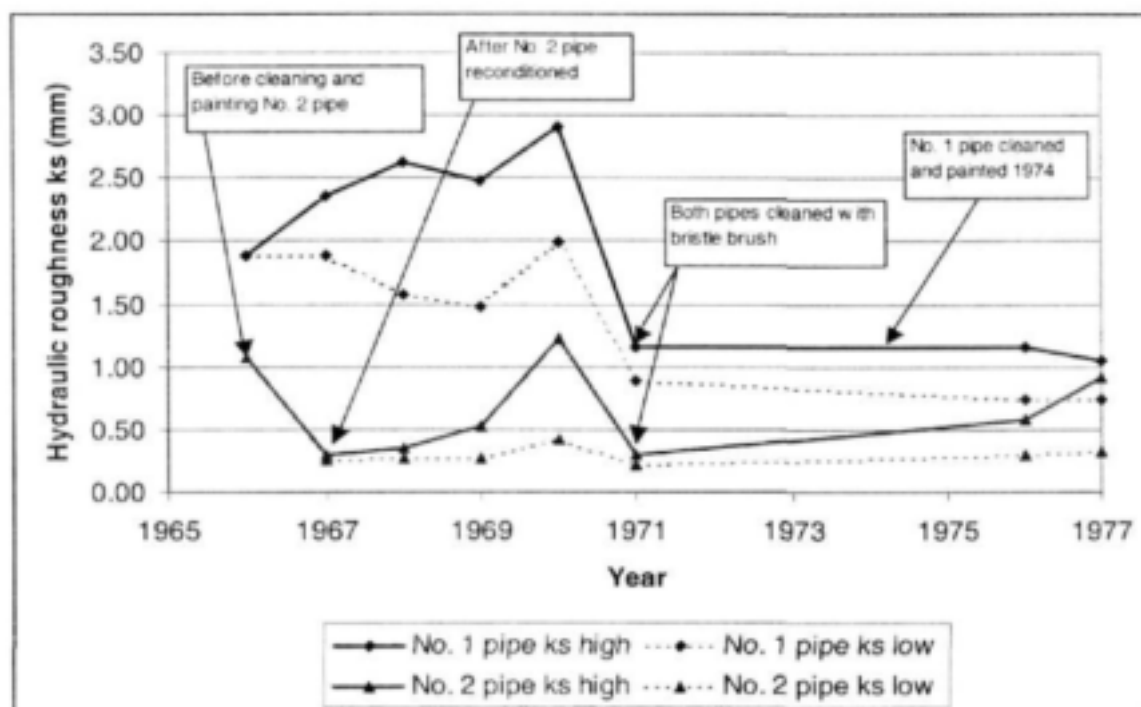


Figure 8.2.4-1 Tarraleah pipes ageing after cleaning

8.2.5 Orange-Fish Tunnel

During 1999 a 150 m section of the tunnel was cleaned by using pressurized water jets with rotating heads. This tunnel section was located 10 m upstream of shaft 7a (surge shaft at downstream end of tunnel). Although there was no visible sedimentation in the tunnel, the test was carried out to find out whether this technique is suitable for hard water conditions and whether the tunnel concrete will not be damaged. The nozzles were held about 1 m from the concrete which gave a spray diameter of about 0.5 m on the concrete surface (Figure 8.2.5-1). The cleaning was completed in about one week. Due to the large diameter of the tunnel, a truck was used as platform to reach the roof of the tunnel (Figure 8.2.5-2). A control tunnel section of 150 m length was used to record and compare head loss over time in the cleaned and not cleaned tunnel sections. The data indicate that the cleaned tunnel is rougher and damaged, and did not improve with time due to possible sediment deposition. Figure 8.2.5-3 shows the test pipes at shaft 7a and field measurement during 2001.

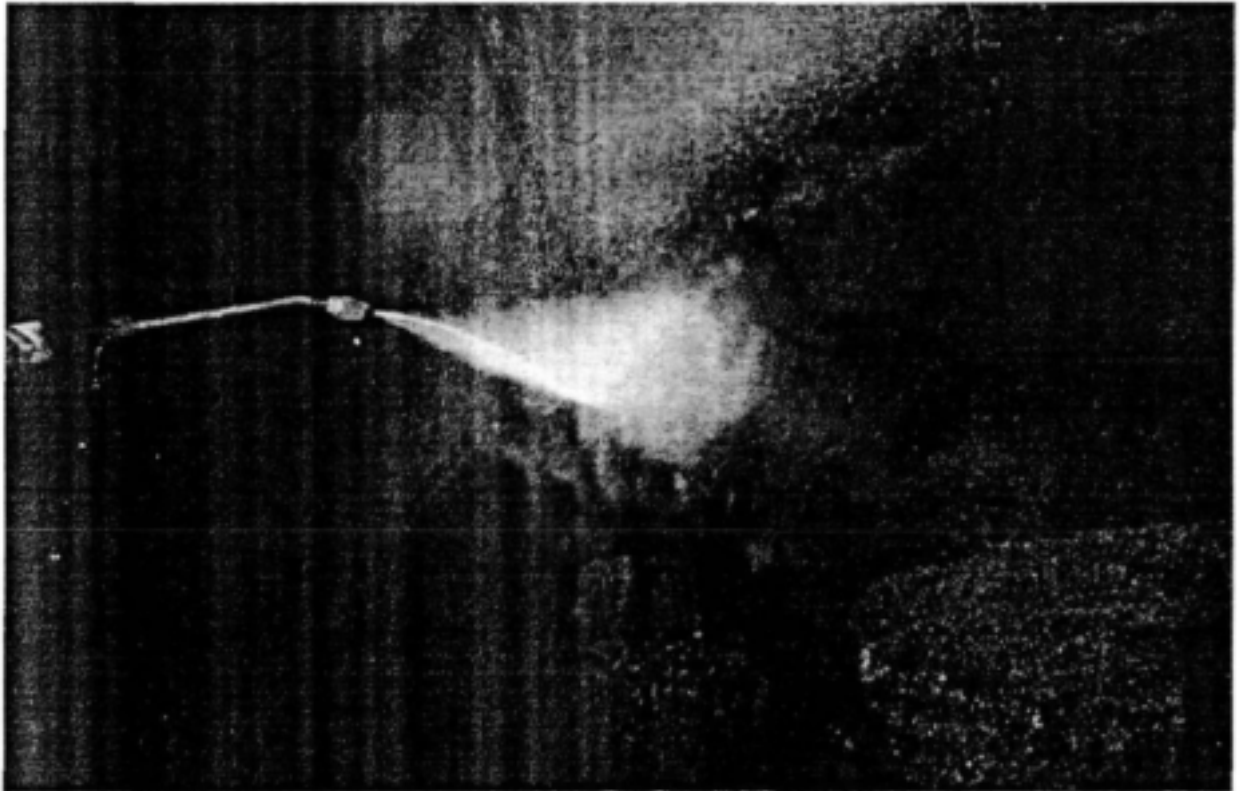


Figure 8.2.5-1 Water jet cleaning of Orange-Fish Tunnel (1999)



Figure 8.2.5-2 Cleaning team on truck in Orange-Fish Tunnel



Figure 8.2.5-3 Field measurement at shaft 7a during 2001

8.2.6 Theewaterskloof Tunnel

This soft water corroded tunnel was cleaned by using brooms, wetted with the water at the tunnel invert, to remove the sediment deposits over a 150 m section upstream of the Wolwekloof vehicle entrance (Figure 8.2.6-1). This operation was carried out by DWAF, and HDPE pipes were installed upstream and downstream of the cleaned and a control section, and taken out horizontally about 300 m and then up the mountain to an elevation above the Theewaterskloof Reservoir water level. After installation the static water level was recorded and then steady flows were tested to determine the hydraulic roughness of the tunnel sections. The cleaned section showed consistently higher roughness than the control section. It therefore seems that no matter how careful you are when cleaning a tunnel exposed to soft water conditions, damage to the concrete surface is highly likely to occur. Figure 8.2.6-2 shows a cleaned section (left hand side), next to the sediment deposition. Figure 8.2.6-3 shows the tunnel after cleaning with coarse aggregate on the surface of the concrete.



Figure 8.2.6-1 Theewaterskloof Tunnel cleaning during 1999

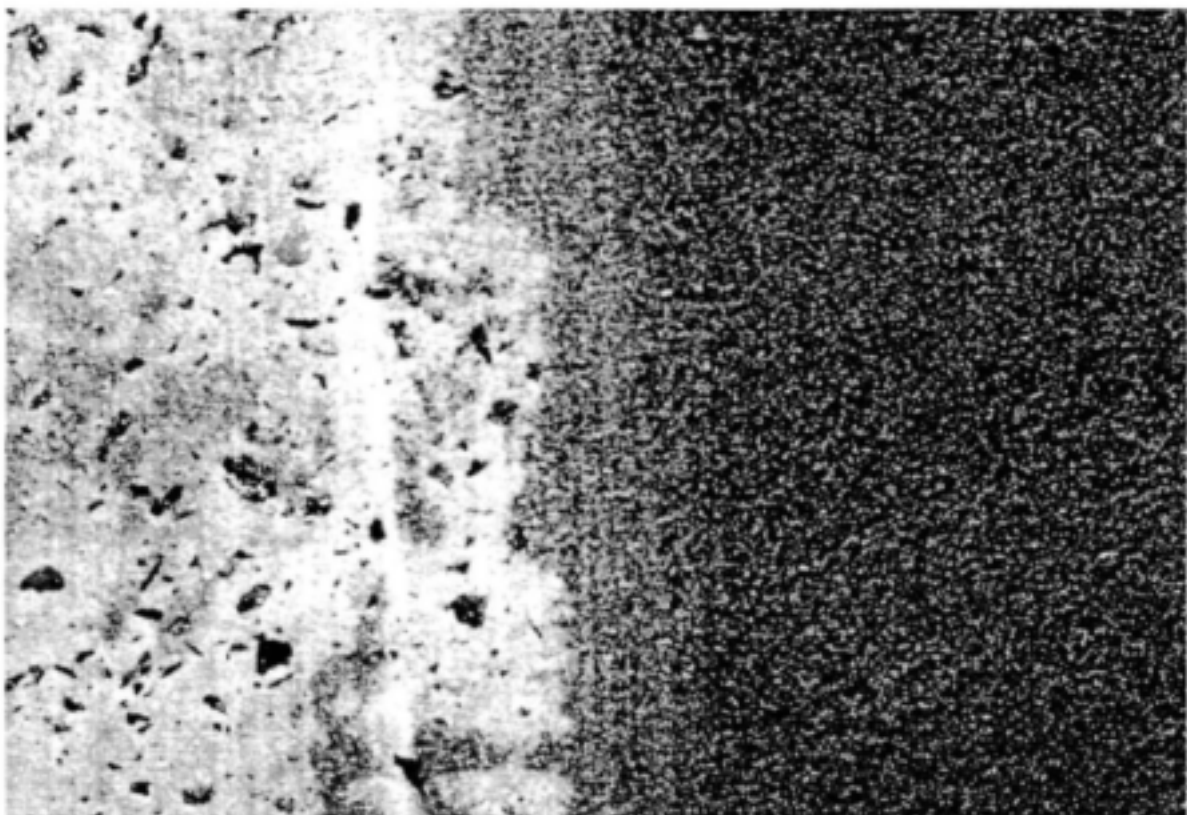


Figure 8.2.6-2 Theewaterskloof Tunnel with locally cleaned section next to sediment deposits



Figure 8.2.6-3 Theewaterskloof Tunnel after cleaning

8.2.7 Roode Elsberg Tunnel

A tunnel section close to the intake was cleaned by the Hex River Irrigation Board. As in the case of the Theewaterskloof Tunnel, brooms were used wetted with the invert water. The cleaning operation is not an easy task in the wet, dark and cold conditions (Figure 8.2.7-1). HDPE pipes were installed and taken up the shaft, but tests on the cleaned section could not be performed due to relatively dry winters with little spillage and excess water available for high enough flow in the tunnel and because the HDPE pipe connection had to be broken when the emergency sluice gate was raised. Divers would have to carry out the reconnection by swimming 30 m into the tunnel from the intake in the reservoir. The water depth in the reservoir was however too deep to use standard diving equipment and the exercise was abandoned.



Figure 8.2.7-1 Tunnel cleaning at Roode Elsberg Tunnel

During the cleaning operation, however, it was quite clear that the concrete was damaged by the cleaning operation. Aggregate located on the concrete surface with 20 to 30 mm diameter fell from the tunnel wall, leaving holes.

8.3 Chlorination

8.3.1 USA experience

At the MWD tunnels chlorination is carried out on an annual basis, and seems to be effective if implemented with mechanical cleaning. The system is however closed. More details are provided in Chapter 9.5.

8.3.2 Tasmanian tunnels

Flow tests carried out during 1964 and 1965 in siphons of the Tarraleah No. 2 Canal, Tasmania, revealed some very interesting information with regards to the hydraulic roughness changes. The hydraulic roughness data are indicated in Figure 8.3.2-1 (Brett, 1979). The project was completed in 1959. The No. 1 siphon is 1.93 m in diameter with 66 m of steel pipe and 745 m of woodstave pipe. No. 3 Creek siphon has the same diameter, with 158 m of steel pipe and 594 m of woodstave pipe. The steel pipe was coated with horizontal retort tar. In one of these conduits experimental injections of chlorine have been used to prevent slime growth.

The 1964 injections of chlorine did not seem to be as effective in reducing the friction in a slimed conduit as did the 1965 injections, which prevented slime regrowth after cleaning and kept Manning's n down to 0.0104 ($k_s = 0.11$ mm) in No. 3 Creek siphon, whereas in No. 1 Creek siphon Manning's n grew from 0.010 ($k_s = 0.07$ mm) to 0.012 ($k_s = 0.46$ mm). Chlorination seemed to have been effective only after cleaning.

In Tasmania, chlorination of tunnels has not been adopted because of possible adverse effects on the environment (Brett, 1979).

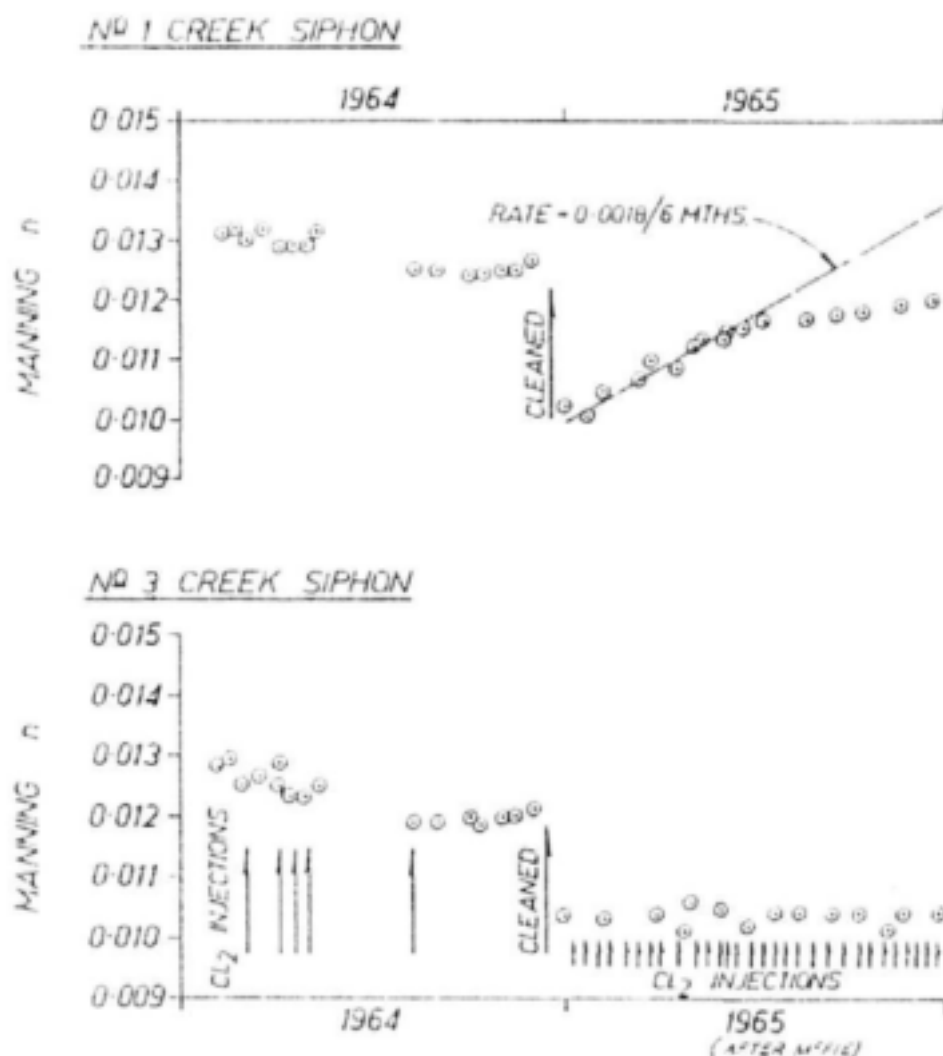


Figure 8.3.2-1 Hydraulic roughness change with chlorination at Tarraleah siphons (Brett, 1980)

8.4 Concrete design

The first and foremost line of defence against aggressive waters is to use a sound, dense and impervious concrete, made from the appropriate materials, and properly compacted and cured (Basson, 1989).

The gradual erosion of the leached mortar can be minimized by use of special cements, addition of pozzolan to mixes, or use of a variety of protective coatings and sealants applied to the concrete surface (Graham, 1998).

No portland cement concrete, regardless of its other ingredients, will withstand attack from water of high acid concentrations. Where strong acid corrosion is indicated, other construction materials or an appropriate surface covering or treatment should be used. This may include applications of sulfur-concrete toppings, epoxy coatings, polymer impregnation, linseed-oil treatments, or other processes, each of which affects acid resistance differently. Replacement of a portion of the Portland cement by a suitable amount of pozzolan selected for that property can improve the resistance of concrete to weak acid attack (Graham, 1998).

The addition of an appropriate amount of silica-fume and a high-range water-reducing admixture to a concrete mixture will greatly increase compressive strength and abrasion resistance.

8.5 Tunnel linings & other materials

8.5.1 General considerations

A variety of materials are used for repair of concrete, some are better suited for certain repairs, and judgement should be exercised in the selection of the proper material. Consideration should also be given to the time available to make repairs, access points, logistics in material supply, ventilation, nature of the work, available equipment, and skill and experience of the local labour force. For example if a surface coating has to be applied to a concrete tunnel after commissioning, sufficient downtime is limited and one has to allow for dewatering, drying, surface preparation, applying the coating, and filling. Such downtime for general maintenance which has been allowed for at the LHWP is about one month per year. Longer periods would only be available during the early life of this tunnel when the system yield exceeds the water demand.

8.5.2 Acrylics and other concrete polymer systems

There are three main ways in which polymers have been incorporated into concrete to produce a material with improved properties as compared to conventional Portland cement concrete. These are (Graham et al. 1998):

- polymer-impregnated concrete (PIC),
- polymer-portland cement concrete (PPCC), and
- polymer concrete (PC).

PIC is a hydrated portland-cement concrete that has been impregnated with a monomer which is subsequently polymerised in situ. By effectively case hardening the concrete surface, impregnation protects structures against the forces of cavitation and abrasion. The depth of monomer penetration depends on the porosity of the concrete and the process and pressure under which the monomer is applied. These materials are quite costly and some monomer systems could be hazardous to work with if unskilled workmen are used.

PPCC is made by the addition of water dispersible polymers directly into the wet concrete mix. PPCC, compared to conventional concrete, has higher strength, increased flexibility, improved adhesion, superior abrasion and impact resistance, and improved durability. The most commonly used PPCC is a dispersion of organic polymer particles in water. Typically, the fine aggregate and cement factors are higher for PPCC than for normal concrete, and therefore it would have lower resistance to soft water corrosion.

PC is a mixture of fine and coarse aggregate with a polymer used as the binder. This results in a rapid-setting material with good chemical resistance and exceptional

bonding characteristics. So far, polymer concrete has had limited use in large-scale repair of hydraulic structures because of the expense of large volumes of polymer for binder.

Polymer concretes are finding application as concrete repair materials for patches and as precast elements for repair of damaged surfaces, and have been used at several hydraulic structures in the USA such as at Grand Coulee Dam.

8.5.3 Preformed polymeric coatings

Tunnels and conduits that have suffered surface damage due to chemical attack or bacterial action can be protected from further damage with a non-bonded mechanically attached PVC lining. Such a lining at the Berg River crossing of the Franschoek-Berg-Jonkershoek Tunnel scheme was unsuccessful, since it was broken up in pieces over time and some pieces blocked the Berg River outlet valves, and were very difficult to remove without dewatering the tunnel.

Polyurethane and neoprene coatings exhibited good resistance to abrasion during high-head testing (Graham et al., 1998) and offer good protection against chemical corrosion. The problem with flexible coatings like these is their bonding to the concrete surfaces. Once an edge or a portion of the coating is torn from the surface, the entire coating can be peeled off rather quickly by hydraulic force, and their use in tunnels is therefore not recommended.

8.5.4 Organic coatings applied as liquids

This category covers a wide variety of materials, ranging from low-performance bituminous varnishes to highly durable catalysed coatings. Table 8.5.4-1 gives the most commonly used types, but is by no means an exhaustive list of all possible types (Basson, 1989).

Table 8.5.4-1 Coatings for concretes in aggressive water (Basson, 1989)

Category	Typical examples	Minimum coating thickness required	Remarks
Inorganic	Extra thickness of base concrete as sacrificial allowance Plaster coats	20 to 50 mm	Use under mild to moderate leaching conditions only; typically used in sewers
Preformed polymeric liners	Polyethylene Polyvinyl chloride Polychloroprene	0.2 to 1 mm	
Emulsion-based waterborne organics	Polyacrylics Polyacrylonitriles	150 µm	Tolerant of damp surfaces
Solvent-based organics	Chlorinated rubbers Vinyls Vinylidenes	150 µm	High-build, thixotropic types available
Catalyzed organics	Epoxy tars Solventless and water-based epoxies Polyurethanes	150 to 300 µm	Accurate mixing and time scheduling required

Bitumens were widely used in the past but have been omitted here due to their relatively high permeability and inferior ageing characteristics, as have hot-applied, high-build, coal-tar enamels as a result of the difficulty of application requiring highly skilled workers. Underwater materials are specially formulated and should be selected with the manufacturers technical personnel. If the water that is to be in contact with the coating is intended to be potable, the manufacturer should guarantee that the material is free from any toxic substances and has been approved for the purpose.

Preparing concrete surfaces for coating should be carried out carefully to ensure adequate adhesion. The basic function of any method of surface preparation is to remove any deleterious contaminants that may be present. Unless otherwise indicated by the coatings manufacturer, one of the following two procedures should be employed (Basson, 1989):

- Brush-off blastcleaning. Over-enthusiastic blasting could lead to a rough abrasive surface, as found at the Orange-Fish Tunnel during this study, when rotating head, high pressure jets were used to clean a 150 m length tunnel.
- Acid etching is a chemical process by which unwanted contaminants are removed by means of an acid treatment

Surfaces that have been wetted during surface preparation should be allowed to dry thoroughly before organic coatings (other than water-borne emulsions), are applied. Trapped moisture reduce adhesion and leads to blistering. The moisture content of the concrete before application should be below 5 to 8 % (Basson, 1989).

Cement-based mortars may be trowelled or applied by pneumatic projection (shotcreting), while liquid organic coatings may be applied by brush, roller, conventional spray or airless spray. In all cases the manufacturer's directions for thinning, spreading rate and where applicable, mixing proportions should be carefully followed (Basson, 1989).

Basson (1989) recommended the use of organic coatings in conditions where his aggressiveness index exceeds 750: highly aggressive (See Chapter 6 classification).

8.5.5 Catalysed organic coatings: Epoxy resins

Resins are natural or synthetic, solid or semisolid organic materials of high molecular weight. Epoxies are one type of resin. These materials are typically used in preparation of special coatings or adhesives or as binders in epoxy-resin mortars or concretes. Epoxies will bond to most building materials and epoxy formulations have been developed recently which will bond to damp concrete and even bond to concrete under water. There are case histories of the successful uses of these materials in hydraulic structures, in the USA as well as the Drakensberg pump storage scheme tunnel where it was used for patching.

If epoxy is used for patching a very smooth finished surface is obtained, highly resistant to damage, with the result that the abutting original material erodes away, leaving an abrupt change in surface geometry and developing a condition worse than the original damage. This was observed in the Franschhoek-Berg River Tunnel where patching of poor original concrete that was eroded away was done with high strength concrete. Within a number of years deep eroded holes developed next to the patched areas, some measuring 2 m by 1 m in area, and 1 m deep.

8.5.6 Preplaced-aggregate concrete (PAC)

PAC also known as "prepacked concrete", is used in the repair of large cavities and inaccessible areas. Well graded aggregate is placed in a form and neat cement grout or sanded grout is then pumped into the aggregate matrix through openings. The grout is placed under pressure until initial set and the concrete has a low volume change.

8.5.7 Pipe inserts

This method is commonly used for the repair of small diameter pipes to obtain a jointless, structurally sound pipe inside an existing pipe without excavating the existing unsound pipe. One such method is to place a plastic pipe inside the deteriorated concrete pipe and then to fill the annular space between the concrete and plastic liner with grout. With the proper selection of material for the plastic liner pipe insert, this repair method can provide a sound, chemically resistant lining.

Another popular method is the installation of a resin saturated fibreglass "hose" into the pipeline using water pressure. After installation, the hose is filled with hot water to initiate the chemical reaction of the resin. The hardened resin forms a rigid pipe lining.

8.6 Other measures

Measures to increase the discharge capacity such as doubling or partial doubling, construction of a new tunnel, minimization of secondary energy losses and pumps could be considered with other measures listed in this chapter, to provide an economical and sustainable design solution.

8.7 Tunnel costs

Historical tunnel costs are shown in Table 8.7-1. The unit costs (Figure 8.7-1) are extremely expensive and everything possible should be done to prolong the life of a tunnel by limiting the rate of ageing and ultimate roughness, especially if this is possible through flow control during operation.

Table 8.7-1 Historical tunnel costs

HISTORICAL TUNNEL COSTS											
	Project	Phase	Start point	End point	Length (km)	Diameter (m)	Lining	Construction Method	Actual cost (mil Rand)	Year of completion	Significant cost implications
1	Lesotho Highlands Water Project	Transfer Tunnel	Malibamatso Intake	Muela Hydro Power Station	45.00	4.35	In situ	TBM	^(*) 1461	1998	None
2		Delivery Tunnel South	Muela Intake	Vent Shaft 5	15.00	5.10 bored 4.5 lined	In situ	TBM	^(*) 397	1998	Partial lining only 30% of length. Steel lining at river crossings
3		Delivery Tunnel North	Vent Shaft 5	North of Clarens	22.00	5.10 bored 4.6 lined	Pre-cast	TBM + One-pass lining	554	1997	Underpassed two major rivers
4		Matsoku	Ha Tsehla	Kutha Kutha	5.60	4.5 Horse shoe shape	Floor - In situ Wall - Fibre reinforced	Drill and Blast	34	2001	None
5		Mohale	Mohale Reservoir	Katse Reservoir	32.00	5.30 bored 4.8 lined	Pre-cast	TBM ₁	1000	2004	Hard Rock Water Fissure
6						6.90 bored 6.40 lined		TBM ₂			
7	Midmar Tunnel	-	Merryvale	Pieter Maritzburg	6.60	3.8 bored 3.5 lined	In & Out lets lined, 2.5km. Reinforced shotcrete and rock anchors	TBM	91	1996	None

^(*) April 1994 Rates

TBM - Tunnel Boring Machine

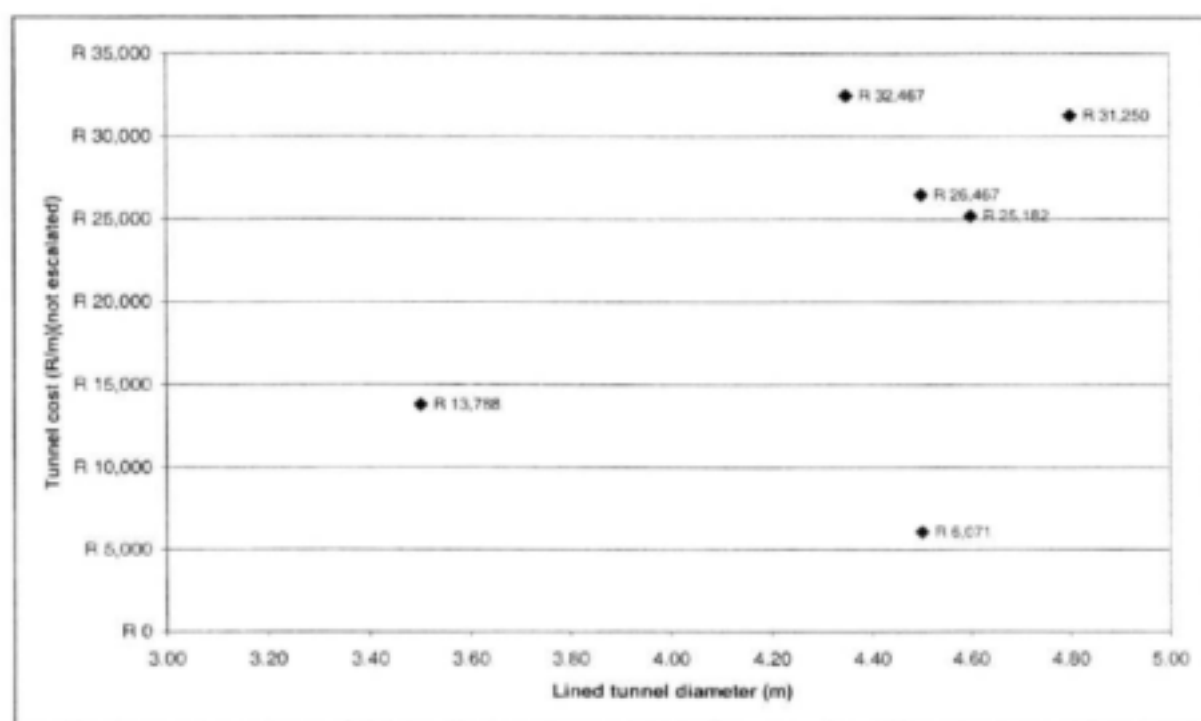


Figure 8.7-1 Unit costs of recently completed tunnels

9 LHPW TUNNEL RATE OF AGEING AND ULTIMATE ROUGHNESS PREDICTION

9.1 Introduction

The first phase of the LHWP was completed recently. A layout of the LHWP system is shown in Figure 9.1-1, while Figures 9.1-2 to 9.1-4 show more detailed profiles of the Transfer Tunnel, Delivery Tunnel and Intake Tower at Katse Dam respectively. The Transfer Tunnel conveys water from Katse Dam to the Muela hydropower scheme in Lesotho, while the Delivery Tunnel transfers water from Muela Dam to the Ash River in South Africa. The Ash River outfall (end of the Delivery Tunnel North) is shown in Figure 9.1-5.

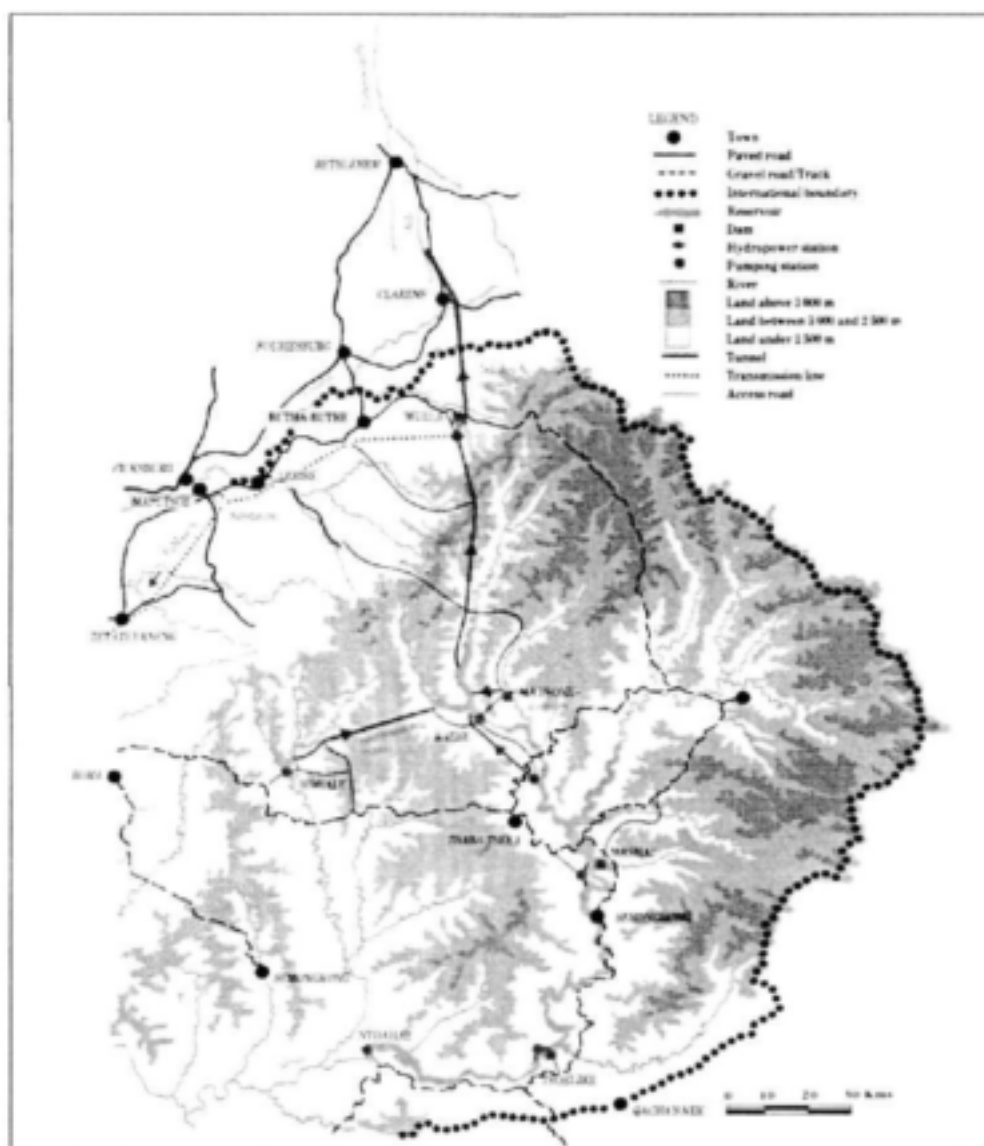


Figure 9.1-1 Layout of LHWP tunnels

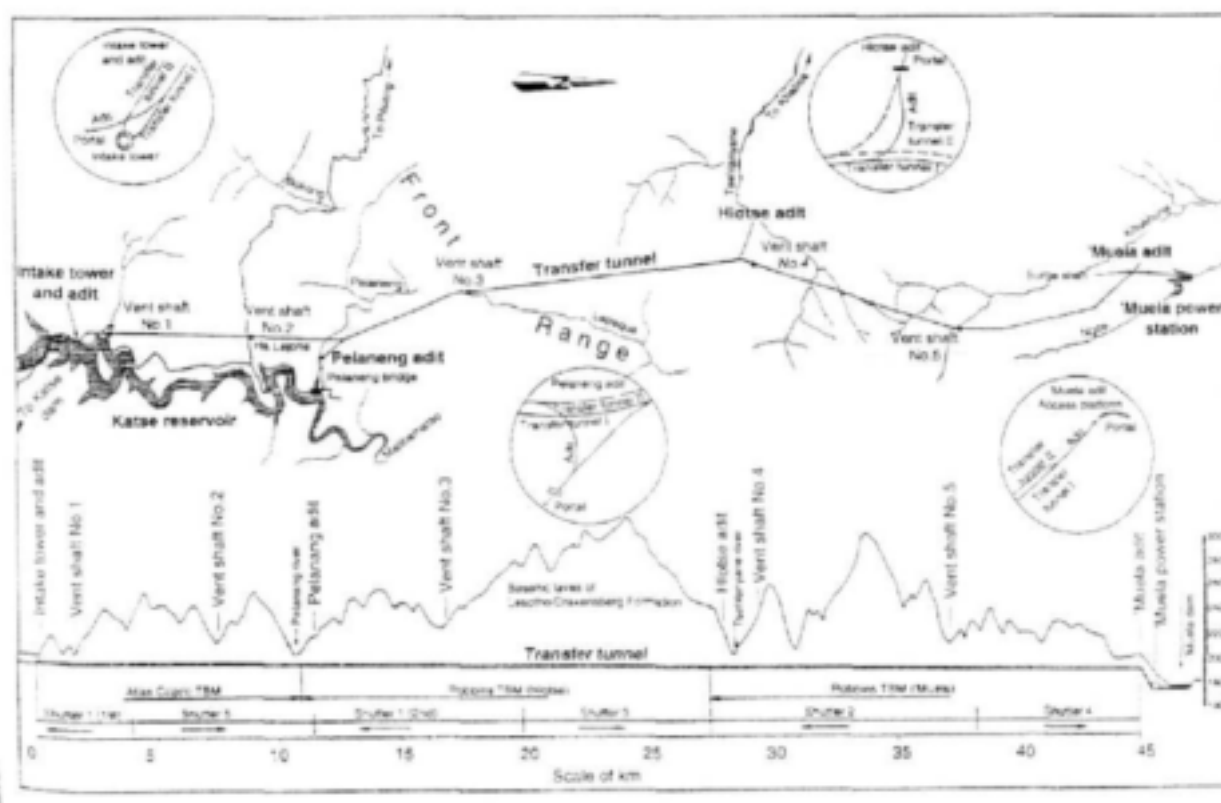


Figure 9.1-2 Layout of the LHWP Transfer Tunnel

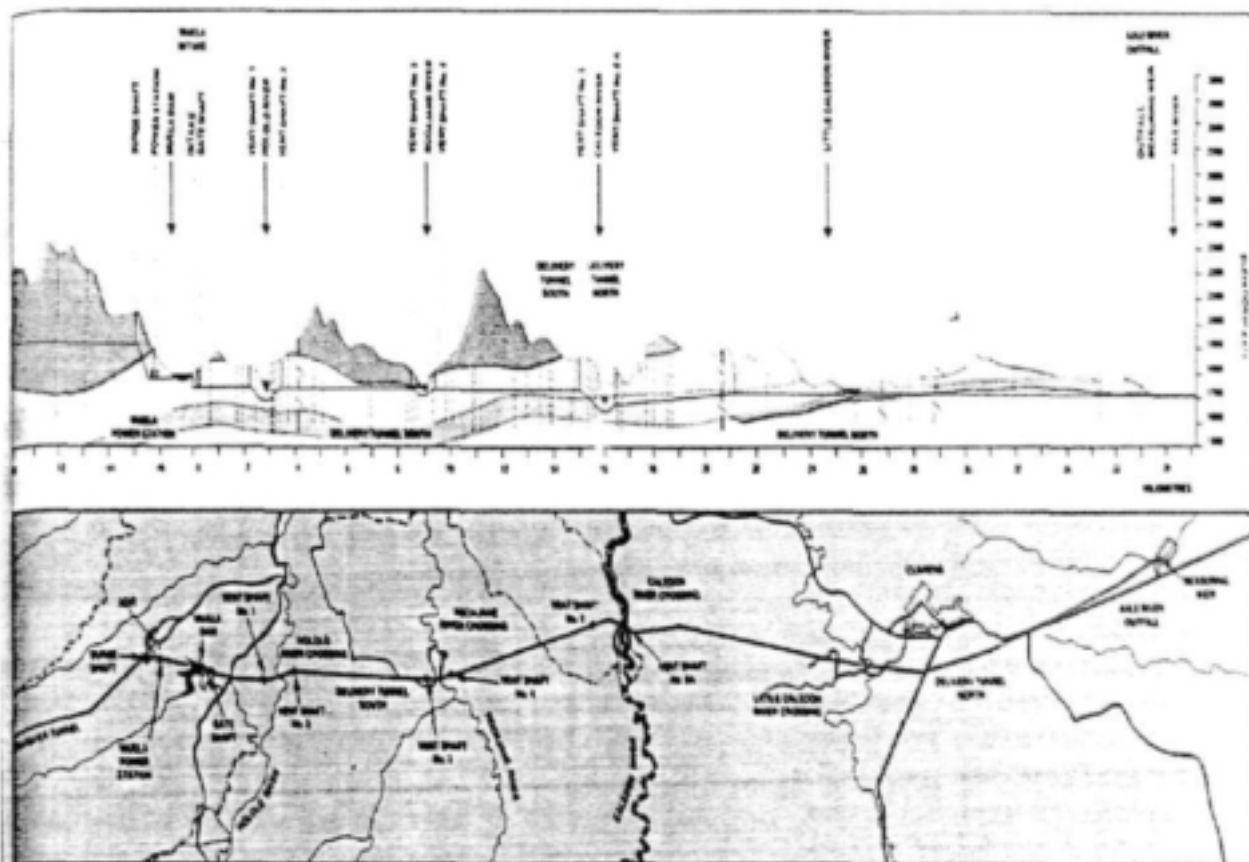


Figure 9.1-3 Layout of the LHWP Delivery Tunnel

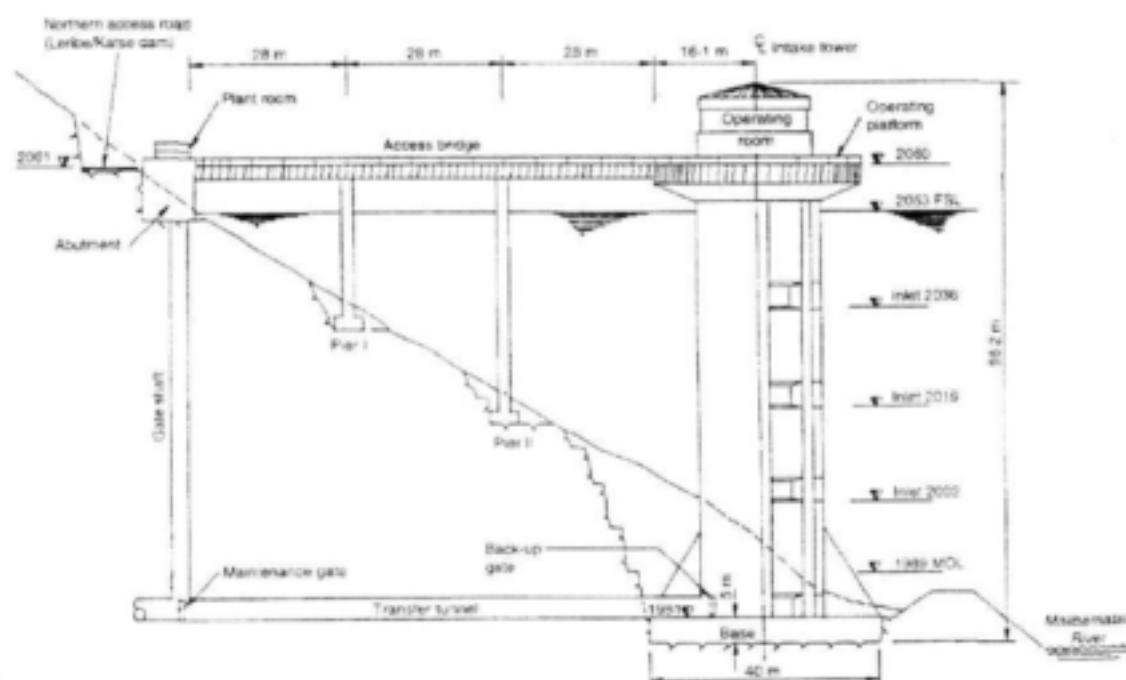


Figure 9.1-4 Intake tower of the LHWP Transfer Tunnel



Figure 9.1-5 Delivery Tunnel North at Ash River

9.2 Ageing prediction (DWAf, 1999)

DWAf commissioned a study during 1999 to investigate the possible future ageing and hydraulic capacity of the LHWP tunnels. Some typical ultimate tunnel roughness values being used during the design of the tunnels are indicated in Table 9.2-1.

Table 9.2-1 Design tunnel roughness for aged condition

Description	Design new	Observed new	Aged ultimate design
LHWP Transfer Tunnel	$n = 0.0123$	$n = 0.0111$ ($k_s = 0.03$ mm)	$n = 0.016$
LHWP Delivery Tunnel	$n = 0.012$	$n = 0.0116$	$n = 0.016$
Orange-Fish Tunnel (5.33 m diameter)	$n = 0.013$	$n = 0.0118$ ($k_s = 0.15$ mm)	design: $n = 0.013$; observed 1990: $n = 0.0138$ ($k_s = 0.9$); 1998: $k_s = 0.6$ mm

Colebrook (1958) expressed the ultimate tunnel roughness as function of hydraulic radius and mean flow velocity. Based on this, the long-term ageing rate during the earlier years of operation of the Orange-Fish Tunnel at low flows is estimated at 0.1 mm/year (0.07 mm/year observed by 1990), with an ultimate roughness of 5.0 mm as worst scenario. The same equation however indicates that the ultimate roughness could be as low as 0.2 mm for the Orange-Fish Tunnel if operated at close to 2 m/s flow velocity. The 1998 tests carried out at similar flow velocities, indicated an Orange -Fish Tunnel roughness of 0.6 mm, which could possibly still decrease with longer duration flushing.

During the design phase of the LHWP absolute maximum and minimum possible rates of roughness increase in the Delivery Tunnel were set at 0.03 mm/year to 0.5 mm/year, based on an equation by Colebrook (1958), which relates change in roughness to the pH of the water. The phenomena observed by Colebrook were that acid waters encouraged algal growth. While the 1988 design assumptions used pH values of 7 to 5.5, CSIR sampling (23 tests) in 1997 indicates pH values exceeding 8. Sampling by DWAF at the Ash River tunnel outlet (12/11/97 to 12/01/99) also indicates a median pH = 7.9 (standard deviation = 0.3), based on 47 samples. Even with a maximum pH of 7 the expected rate of tunnel roughening would be no more than 0.03 mm/year based on the equation of Colebrook. The new ageing equation (DWAF, 1999) predicts an ageing rate of 0.04 mm/year at the median pH of 7.9.

The CSIR analysis also shows that, notwithstanding the high ("alkaline") pH's, the Katse water is also potentially aggressive to concrete and some roughening of lining surfaces over time could be expected. Unfortunately no methodology is available to predict with accuracy what the degree of deterioration of the concrete lined portions of the tunnel will be. The Orange-Fish Tunnel does however provide invaluable information in this regard and based on the 1988 and 1998 tunnel roughness evaluations, it is possible to adapt similar rates of roughening for the LHWP tunnels, in the order of 0.07 mm/year, and 0.022 mm/year based on the 1998 tests.

For the DWAF(1999) study, a probable ageing rate of 0.1 mm/year (conservatively high) has been used and a minimum rate of 0.03 mm/year. The economic analysis was based on an ageing rate of 0.1 mm/year.

While these values indicate that relatively low final roughness values are possible if the LHWP tunnels are operated at design flow capacity (or even higher), an assumption has been made for the study DWAF (1999) that $k_s = 4$ mm (without cleaning), at an ageing rate of 0.1 mm/year as the most probable scenario (DWAF, 1999).

9.3 Transfer Tunnel

Based on the tender assumptions, the minimum, mean and maximum head losses over the full length of the tunnel were estimated to be 32.3 m, 36.6 m and 45.3 m respectively, for a tunnel of 4.95 m I.D., only lined for 6,7 km at 4.35 m I.D. During construction it was decided to concrete line the complete tunnel at 4.35 m diameter, which calculations showed should give similar head losses as the tender proposal.

The design flow of the tunnel for water transfer purposes is 32.6 m³/s, while the design normal maximum discharge (turbines at rated load) is 35.9 m³/s. A review indicated that medium and long-term friction losses in the concrete lined Transfer Tunnel could be expected to be 32.8 m and 45.2 m respectively for the medium and long-term friction losses. These correspond to between 12 and 16 percent of the gross available head with the Katse Reservoir at full supply level.

Details of the commissioning tests carried out at the Transfer Tunnel have been obtained from LHC. The new Transfer Tunnel roughness is 0.03 mm (Manning $n = 0.0111$), which is slightly less than the mean roughness of the Delivery Tunnel of 0.15 mm (Manning $n = 0.0116$). With the current tunnel roughness which is slightly less than the "mean Phase 1A typical" Manning $n = 0.0123$ used during the design of the tunnel, the discharge capacity is now 11 percent higher than the design discharge at a design head loss of 36.6 m. Compared to hydraulic tests carried out at other tunnels, the new tunnel friction loss is relatively small.

The tunnel capacities at various head losses and ageing rates are indicated in Table 9.3-1 and Figure 9.3-1 as determined in a recent study by DWAF (1999). For the purpose of the assessment the same rate of tunnel roughening as with the Delivery Tunnel has been assumed.

The analyses indicate that the Transfer Tunnel could, under certain operating conditions, have sufficient long-term capacity, even at Katse minimum operating level (MOL), to transfer the Phase II yields.

Table 9.3-1 Hydraulic capacity of the Transfer Tunnel with ageing (m³/s) (DWAF, 1999)

Capacity at head loss of	Current condition	Minimum ageing at 0.03 mm/year	Estimated probable ageing at 0.1 mm/year	
Year:	1998	2020	2020	2050
39 m (Katse at MOL ^(a))	42.2	34.4	30.5	28.6
103 m (Katse at FSL)	69.8	56.0	49.6	46.6

Note: (a) The available energy head is limited by the minimum design operating level (1950 m) in the surge shaft upstream of the Muela hydro power station.

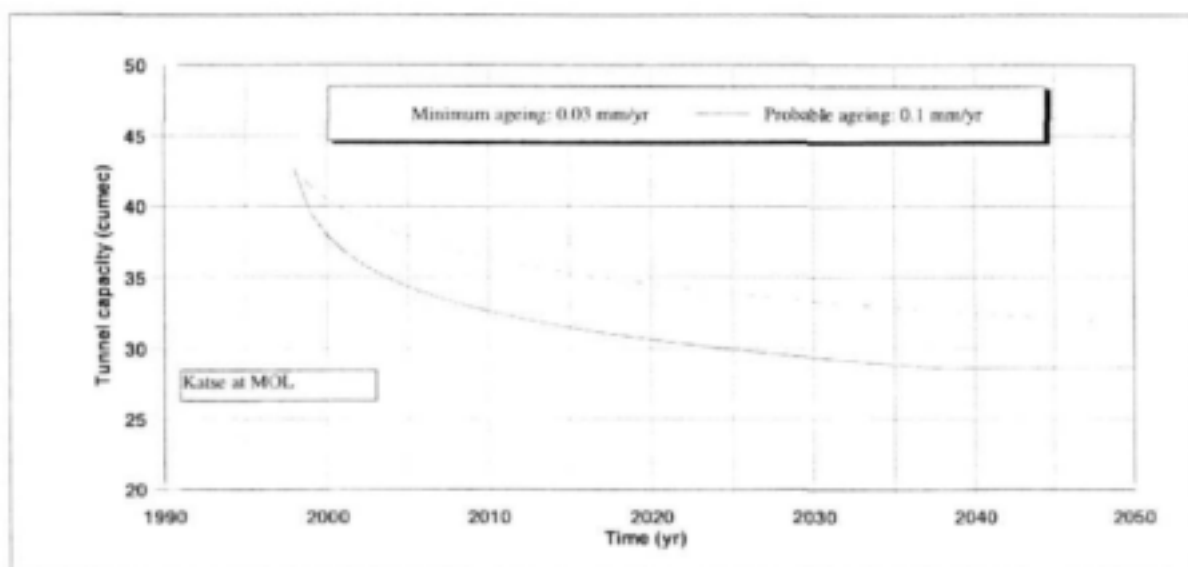


Figure 9.3-1 Simulated hydraulic capacity loss of the Transfer Tunnel due to ageing (DWAf, 1999).

9.4 Delivery Tunnel

Various ageing scenarios of the Delivery Tunnel have been investigated, based on the commissioning tests and a calibrated numerical hydraulic model (DWAf, 1999). The results are indicated in Table 9.4-1 and Figure 9.4-1.

Allowance has to be made for tunnel maintenance on an annual basis in the order of 10 % of the time. The discharges indicated in Table 9.4-1 are absolute values and do not include down time.

Table 9.4-1 Hydraulic capacity of the Delivery Tunnel with ageing (m^3/s)(DWAf, 1999)

Description	Current condition	Minimum ageing at 0.03 mm/year	Orange-Fish tunnel ageing at 0.07 mm/year*	Estimated probable ageing at 0.1 mm/year
Year:	1998	2020	2020	2020
Muela Dam at FSL (1775 m)	48	42	39	37
Muela Dam at MOL (1759.5 m)	38	33	30	29
Muela Dam at AOL (1773.5 m)	47	41	38	37

*Based on 1988/89 tests at the Orange-Fish Tunnel; the recent 1998 tests indicated ageing at 0.022 mm/year.

NOTE: FSL Full Supply Level,
MOL Minimum Operating Level
AOL Average Operating Level

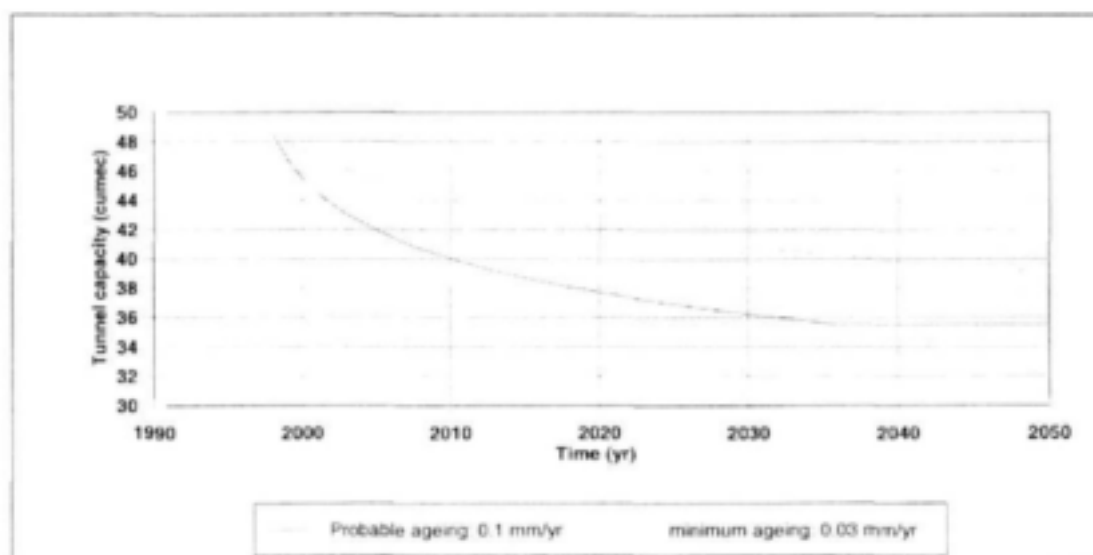


Figure 9.4-1 Simulated hydraulic capacity loss of the Delivery Tunnel due to ageing (DWAf, 1999)

9.5 Proposed operation to minimize rate of ageing

9.5.1 General

Various operational and structural modifications have been investigated for the Transfer and Delivery Tunnels in order to maximize the hydraulic capacities, as counter measures to limit the impacts of tunnel ageing and hydraulic constraints in the system (DWAf, 1999).

9.5.2 Transfer Tunnel

The options available for increasing the capacity of the Transfer Tunnel are operating Katse Reservoir at a higher MOL and/or lower the MOL at the surge shaft (upstream of Muela).

Operation of the Transfer Tunnel at variable flows is possible, and allows water transfers at high Katse reservoir levels equal to the yield, while at low reservoir levels, the tunnel discharge capacity (less than the yield), limits the transfer of water, and would therefore impact on the system yield. This option has been investigated in the yield analysis. Table 9.5.2-1 shows the required MOL's at Katse Reservoir for three phase II yields. It is clear from Table 9.5.2-1 that the highest yield would decrease the live storage in Katse Reservoir considerably.

Table 9.5.2-1 Transfer Tunnel ageing and related MOL(m) required at Katse Reservoir for various possible yields (Surge shaft MOL = 1950 m) (DWAF, 1999)

Year	Discharge: 38.4 m ³ /s	Discharge: 44.7 m ³ /s	Discharge:47.9 m ³ /s
yr 2010	2005m	2025 m	2035 m
yr 2020	2012 m	2028 m	2046 m
yr 2030	2017 m	2034 m	(2054 m)**
yr 2037	2020 m	2045 m	(2059 m)**
yr 2050*	2020 m	2045 m	(2059 m)**

Notes: * MOL constant from yr2037 due to ageing equilibrium

** Katse FSL = 2053 m

Katse MOL = 1989 m (design)

Table 9.5.2-1 is based on a design surge shaft MOL = 1950 m, and rate of ageing of the Transfer Tunnel = 0.1 mm/year.

Table 9.5.2-2 provides additional information in the case of a combination of operating Katse Reservoir at a higher MOL, as well as operating at a lower MOL in the surge shaft (DWAF, 1999).

Table 9.5.2-2 Transfer Tunnel ageing and related MOL(m) required at Katse Reservoir for various possible yields, with doubled surge shaft (Surge shaft MOL = 1919 m)

Year	Discharge: 38.4 m ³ /s	Discharge: 44. m ³ /s	Discharge:47.9 m ³ /s
yr2010	1989 m	1994 m	2004 m
yr2020	1989 m	2003 m	2015 m
yr2030	1989 m	2010 m	2023 m
yr2037	1989 m	2014 m	2028 m
yr2050*	1989 m	2014 m	2028 m

Note: * MOL constant from year 2037 due to ageing equilibrium

The design minimum operating water level in the surge shaft at the end of the Transfer Tunnel is at 1950 m, well above the tunnel. The purpose of such a seemingly high MOL could be to eliminate negative pressures in the tunnel during startup of the hydro power plant, or during rapid shutdown. The consequences on the Transfer Tunnel capacity are however dramatic with the MOL at 1950 m and could be the limiting capacity for the whole transfer scheme at Katse Dam MOL when phase II flows are considered. The benefit of this scenario is indicated in Table 9.5.2-3.

Table 9.5.2-3 Hydraulic capacity of the Transfer Tunnel with ageing with reduced surge shaft MOL (m³/s) (DWAF, 1999)

Capacity at head loss of	Current condition	Minimum ageing at 0.03 mm/year	Estimated probable ageing at 0.1 mm/year	
Year:	1998	2020	2020	2050
39 m (Katse at MOL) ^(a)	42.2	34.4	30.5	28.6
103 m (Katse at FSL)	69.8	56.0	49.6	46.6
70 m (Katse at MOL and surge shaft MOL at 1919m) ^(b)	57.2	46.1	40.8	38.3

Notes: (a) The available energy head is limited by the design minimum operating level in the surge shaft upstream of the Muela hydro power station of 1950 m.

(b) Maximum possible if surge shaft MOL could be lowered from 1950 m (design) to 1919 m (proposed) to maintain positive head on tunnel.

9.5.3 Delivery Tunnel

Several options are available to increase the hydraulic capacity of the Delivery Tunnel (DWAF, 1999).

a) Raised Muela Dam

The raising of Muela Dam to increase the available hydraulic head of the Delivery Tunnel could be considered. Any higher tailwater level would of course result in decreased hydro power generation and efficiency. The long-term Delivery Tunnel capacity with a raised Muela Dam by 5 m is indicated in Figure 9.4.3-1. The raising of Muela Dam would however result in a higher hydraulic gradient line at the low cover Little Caledon River section, exceeding design pressures.

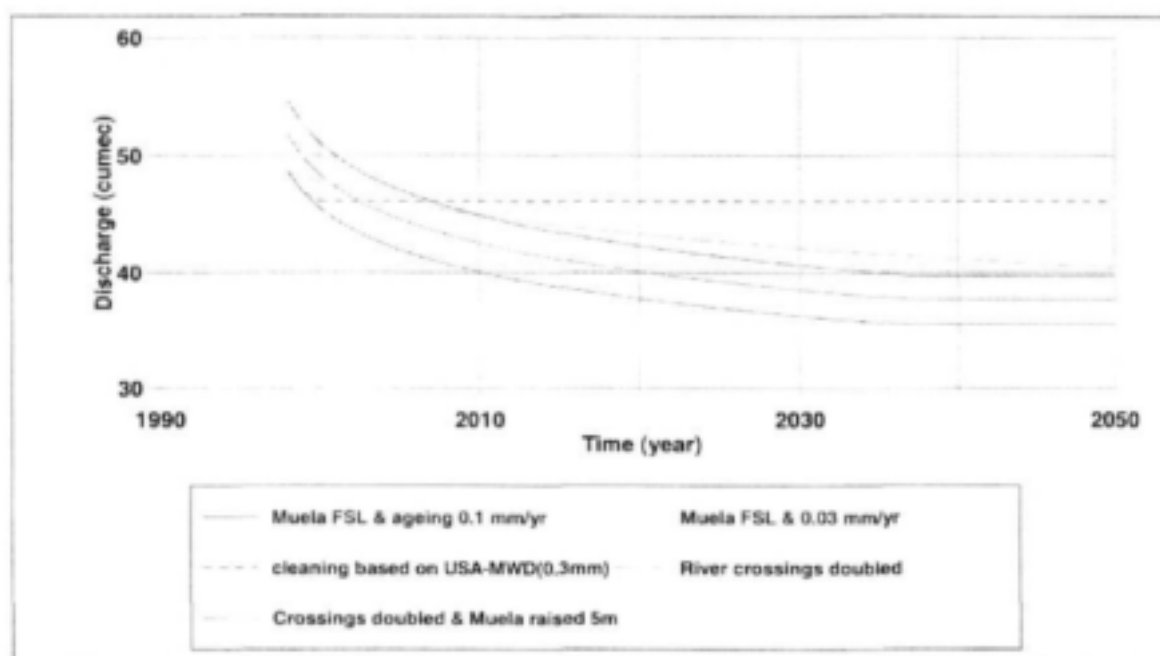


Figure 9.5.3-1 Delivery Tunnel capacity with raised Muela Dam

b) Increase Delivery Tunnel river crossing capacities

The hydraulic gradient line of the LHWP commissioning tests shows clearly that the river crossings, especially at high flows, create relatively high energy losses (HDTC, 1998). By doubling these river crossings, it would be possible to increase the hydraulic capacity of the Delivery Tunnel (See figure 9.5.3-1).

c) Mechanical cleaning : LHWP application

The DWAF (1999) study proposed:

"Regular cleaning of the LHWP tunnels with the aid of mechanical equipment would probably limit the rate of ageing, and while this should probably be tested in the field by pilot project, it is expected that the long-term Delivery Tunnel capacity could be as illustrated in Figure 9.5.3-1. This is based on the USA-MWD experience with roughness coefficients before and after cleaning of $k_s = 0.43$ mm and 0.14 mm respectively, giving a mean roughness of 0.29 mm (Horowitz and Lee, 1971).

With the current LHWP tunnels, the tunnel capacity is relatively high compared to the water yield. As future water requirement increase, however, with the construction of a possible Mashai Dam (after year 2010), the aged tunnel capacity could be limiting the transfer of water. If regular cleaning is considered, annual shut-down of say one week, would result in about 2 percent reduction in yield transferred. It would, however, not be impossible to design an in line automatic cleaning machine which could operate without shutdown of the tunnel."

However, based on the findings of this WRC study, mechanical cleaning in aggressive water conditions is not recommended due to possible damage to the concrete.

d) Hydraulic flushing

At high flow velocities, preferably above 2 m/s, it would be possible over time to recover some of the lost hydraulic capacity due to ageing by flushing out some of the previously deposited sediments in the tunnel. Metcalf and Jordaan (1990), indicated the impact of hydraulic flushing during the Orange-Fish Tunnel testing in 1988:

"Whilst there is a large degree of scatter at lower flows, the trend towards $k_s = 1.0$ mm ($n = 0.0139$) at the higher flows is clearly discernable. The fact that roughness tended to fall as flow was increased, and stay at the lower value ($k_s = 1.0$ to 1.1 mm) when the lower flows were re-established after the tests, suggests a scouring action," (although maximum flow velocity was 1.8 m/s). During the 1998 hydraulic tests carried out at the Orange -Fish Tunnel, a longer duration, higher flow test has been carried out specifically to test the effectiveness of hydraulic flushing as possible remedial measure.

During tests carried out in 1989, Metcalf and Jordaan (1990), observed: " The value of n (Manning) obtained at the start of the test for the working flow of $21.3 \text{ m}^3/\text{s}$ was at $n = 0.0142$, about 2 % higher than that recorded at the end (i.e. post-scour) of the previous years testing, $n = 0.0139$. On resuming working flows of $21.3 \text{ m}^3/\text{s}$ after the tests 'n' showed a decrease to 0.0139, i.e., the same as the previous year."

"At the higher test flow of $34.6 \text{ m}^3/\text{s}$ an initial value of $n = 0.0141$ was obtained which showed a slow fall during the test." This friction factor at the Orange-Fish Tunnel was estimated to be about $n = 0.0137$ ($k_s = 0.8$ mm), once again implying a scouring action by the above normal flow, and an increased discharge capacity of 3 %. During even higher flows in the tests carried out during 1998, the tunnel capacity improved by about 5 % within 20 hours. This was achieved after short duration, high flows, and even higher flushing efficiency would be possible at higher flows ($v > 2$ m/s), typical of the LHWP tunnels. (The Colorado River tunnels typically operated at a flow velocity of 2.6 m/s).

Colebrook (1958) also found that pipes operated at higher hydraulic shear stresses (higher flow velocities), aged slower than tunnels operated at low flow velocities. To limit the rate of ageing it is therefore important that the LHWP tunnels are operated at maximum system yield, with perhaps certain periods on an annual basis, running at maximum tunnel capacity.

e) Chlorination

MWD (USA) uses monthly chlorination during the summer months with sodium hypochlorite to remove algae build-up in the Colorado Tunnel (Kelly, 1998). They have about 12 sites where 4500 litres of 5.5 % sodium hypochlorite is applied. The applications are in early evening and at the inlets to inverted siphons. This maximizes mixing. In years past, liquid chlorine was used, but MWD found that they could attain and maintain a higher chlorine residual with the sodium hypochlorite.

Chlorination of the LHWP tunnels would probably not be practical considering the fact that they discharge into rivers. It would therefore only be an option to consider if the proposed Rand Water pipelines are constructed, linking the tunnel outfall to the Rand Water purification plants in a closed system.

f) Relining

Relining of the Transfer and Delivery Tunnels is possible. Relining could be an epoxy type/poly-urethane coating to be applied in future only on concrete lined sections of the tunnel. An ultimate roughness of the relined tunnel of 1.5 mm have been assumed (DWAF, 1999), which would give the Delivery Tunnel a hydraulic capacity of about 43 m³/s. An ultimate roughness of as low as 0.6 mm would however also be realistic to use, but one has to take into account the possibility of abrasion and siltation (DWAF, 1999). The assumption that a relined tunnel would have a different ultimate roughness than a concrete tunnel used in the DWAF (1999) study should however be modified to be the same.

g) Phased doubling of the Delivery Tunnel

Doubling of the Delivery Tunnel could be considered (or even phased doubling), after other cheaper options have been considered. Complete doubling of the tunnel would give the capacities indicated in Table 9.5.3-1.

Table 9.5.3-1 Hydraulic capacity of a doubled Delivery Tunnel

Year	Hydraulic capacity (m ³ /s)
2020	75
2050	71

Note: Table 9.5.3-1 is based on a rate of ageing of 0.1 mm/year.

Phased doubling is possible and doubling has been considered from the Delivery Tunnel north outfall first, in order to lower the hydraulic gradient line at the Little Caledon low cover section (DWAF, 1999).

9.6 LHWP Tunnel roughness prediction based on findings of this WRC study

a) Delivery Tunnel North

During the 1999 tunnel inspection of the Delivery Tunnel North, about 2 years after operation started, a thin uniform layer of fine sediment was observed in the tunnel. At the outlet the tunnel dried and the sediment layer was not visible (Figure 9.6-1), but deeper into the tunnel it completely covered the tunnel wall (Figures 9.6-2 & 9.6-3). The LHWP water is aggressive to concrete and the upper layer of the concrete (under the sediment) was found to be soft and could easily be damaged, exposing the coarse aggregate (Figure 9.6-4).

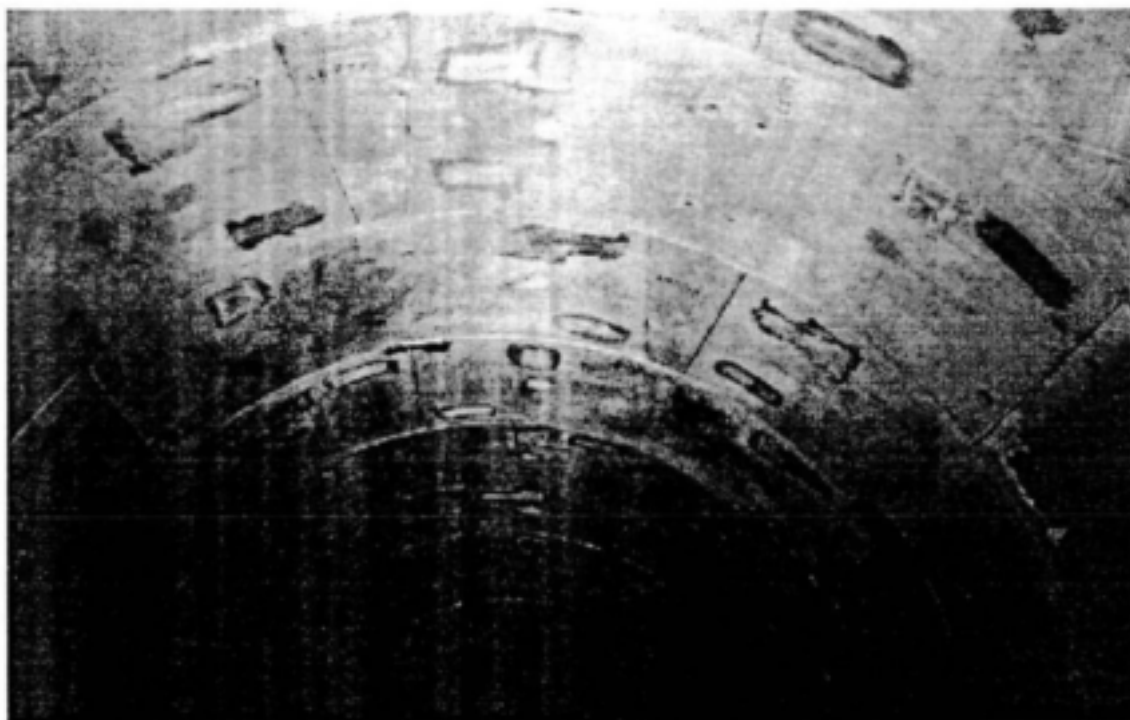


Figure 9.6-1 Segmentally lined Delivery Tunnel North



Figure 9.6-2 Sediment deposition in the DTN (1999)



Figure 9.6-3 Sediment deposition partially covering the joints in the DTN (1999)

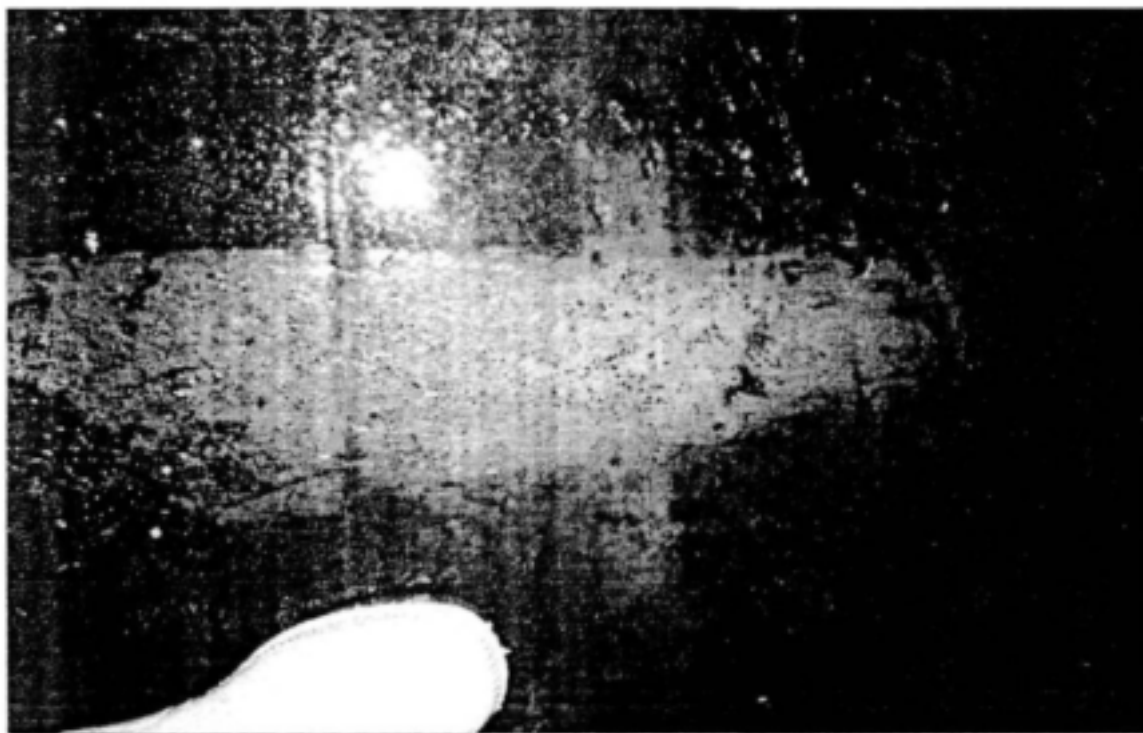


Figure 9.6-4 Soft water corrosion in the DTN (1999)

The following calculation procedure is proposed to determine the long term roughness and discharge capacity of the Delivery Tunnel:

- Determine dominant discharge. This could be based on say the discharge that is exceeded 20 % of the time. The DTN dominant discharge was taken as 35 m³/s at this stage to be conservative. Typical daily flow vary from 15 to 40 m³/s due to hydropower generation.
- Obtain pH data and determine the mean, the range and standard deviation, and select minimum for sensitivity analysis. For DTN the mean is 7.9 with 0.3 standard deviation, measured at the Ash outfall (Calculation based on data in 1999, but pH however subsequently deceased). A minimum value of 7.5 was selected.
- Obtain initial roughness of tunnel
- Apply the rate of ageing equation (eq. 5.7.3-6) for the tunnel at mean pH and selected minimum pH, using the energy slope of the dominant discharge, and determine k_s with time
- Use the ageing k_s values determined above and determine the tunnel discharge capacity by using the maximum available energy slope
- Determine the ultimate roughness for the probable (eq. 5.8.5-2) and maximum probable roughness (eq. 5.8.5-3), based on the dominant discharge and associated energy slope, also considering mean and minimum pH values. Then calculate the ultimate discharge capacity based on the calculated k_s values and the maximum available energy slope, using the Colbebrook White-Darcy Weissbach equation.

The predicted ageing of DTN is shown in Figure 9.6-5, based on a dominant discharge of 35 m³/s to calculate the ageing. The ultimate roughness of the three lines indicated varies between 0.25 mm and 1.31 mm which is much less than the 4 mm used in the original design of the tunnel and in the DWAF (1999) study where 4 mm was also considered as the ultimate roughness, corresponding to about 34 m³/s ultimate discharge capacity. The predicted ultimate discharge capacity is now 41.7 m³/s (probable k_s) with a low value of 38.9 m³/s (maximum probable k_s).

- In order to be conservative, it is recommended that the ultimate roughness minimum should be taken as 0.6 mm due to possible concrete deterioration. The two lower lines in the graph (Figure 9.6-5) should therefore be used.

The probable ageing is given by the central line, with an ultimate discharge capacity of 85 % of the original capacity. The predicted maximum ageing discharge capacity is given by the lower line on the graph, with a long term capacity of 80 % of the original capacity.

The rate of ageing predicted is 0.017 mm/year and 0.032 mm/year for the mean and minimum pH values respectively. This is now considerably less than the estimates of the DWAF (1999) report of probable and maximum probable values of 0.03 mm/year and 0.1 mm/year.

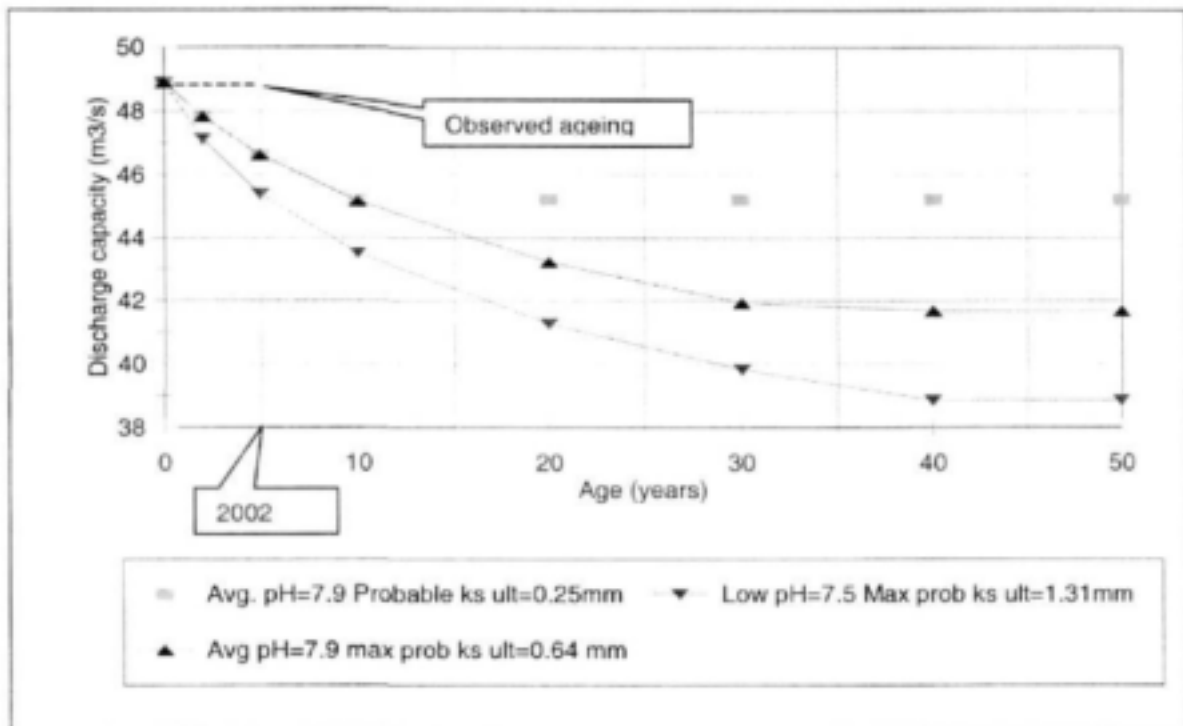


Figure 9.6-5 Predicted ageing of the Delivery Tunnel North (this study)

- The critical conditions for re-entrainment (eq. 5.5.1-1) and transport of non-cohesive sediment transport (eq. 5.6-1) should be determined to transport say 0.5 mm sediment, the typical median size of sand bedded rivers in South Africa.
- The maximum shear stress should not exceed the maximum value (19 Pa) tested in the laboratory (Table 5.5.2-1) where no concrete deterioration associated with an increased roughness occur. The maximum bed shear stress is less at DTN.

The predicted rate of ageing and ultimate roughness has to be refined with more field data, and the ageing prediction of the Delivery Tunnel and Transfer Tunnel should be carried out. At this stage, however, these findings indicate that perhaps a next phase of the LHWP can be implemented without doubling of the tunnels.

Field testing (2002) at the Delivery Tunnel indicates no ageing after 5 years of operation, with head loss changes of less than 50 mm from the commissioning tests. Initially the tunnel was operated at regular peak discharges of 35 to 40 m³/s. Since September 2001 the operation changed to typical peak discharge of 27 m³/s during day time, dropping to 12 m³/s at night, due to hydropower generation requirements and system constraints (Figure 9.6-6).

During the first 5 years of operation, the pH of the water discharging from the DTN has slowly decreased from about 8.0 to about 7.6 (Figure 9.6-6). Since September 2001, when the peak flows reduced, a large scatter in pH data has been observed, which is difficult to explain. The predictions of tunnel ageing shown in Figure 9.6-5 are however based on a range of pH values between 7.5 and 7.9, as observed in the field.

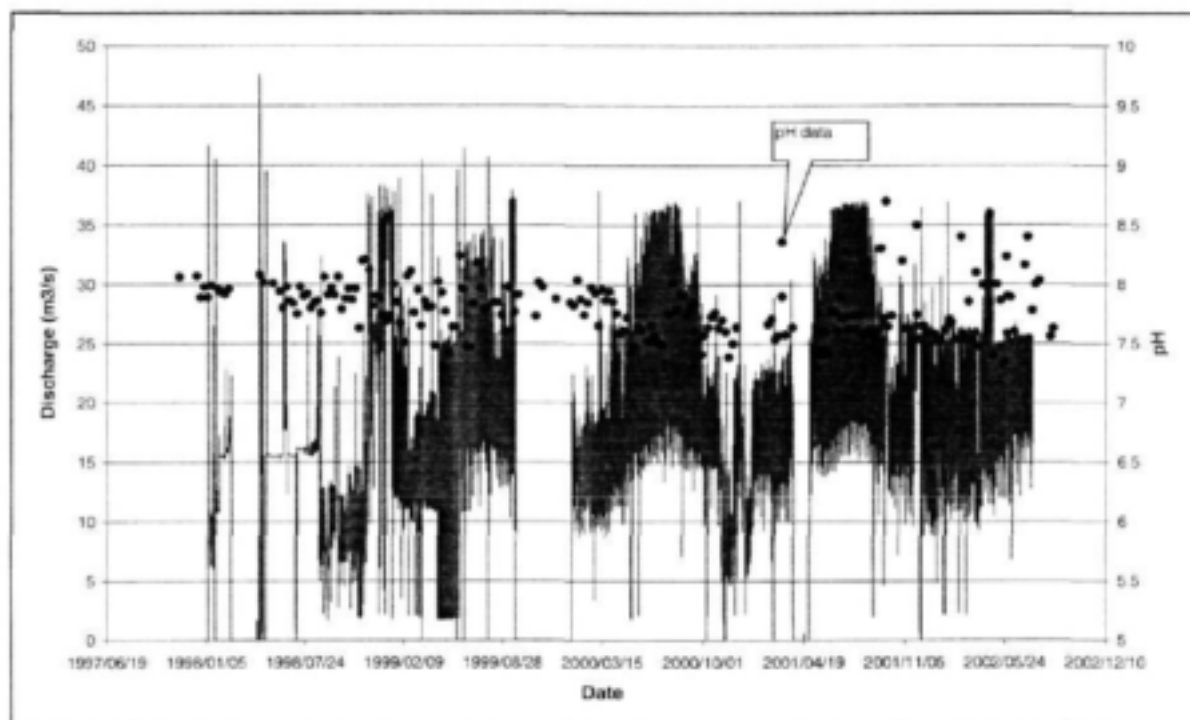


Figure 9.6-6 Delivery Tunnel North observed discharge and pH data

10 TUNNEL DESIGN AND OPERATIONAL GUIDELINES INCORPORATING TUNNEL AGEING

The following general calculation procedure is proposed to determine the long-term hydraulic roughness, rate of ageing and associated discharge capacity of a concrete lined tunnel:

Design aspects:

- a) Determine the dominant discharge, based on say the flow that is exceeded 20 % of the time. Consider changes in future water requirements, daily and seasonal variation.
- b) Determine whether the water is aggressive to concrete (soft water) by referring to Chapter 6.
- c) Calculate the rate of ageing (eq. 5.7.3-6).
- d) Calculate the probable (eq. 5.8.5-2) and maximum probable ultimate (eq. 5.8.5-3) hydraulic roughness. Use the minimum ultimate roughness equal to 0.6 mm to allow for possible concrete deterioration.
- e) Calculate the discharge capacity of the ageing tunnel, also considering pH sensitivity (standard deviation).
- f) Tunnels should not be oversized as this could lead to high ageing rates. Larger tunnel diameters lead to reduced flow velocities, which accelerates the rate of ageing and could lead to a relatively high ultimate roughness. A sensitivity ageing analysis should however be carried out for various tunnel diameters, with a costing exercise.

Operational aspects:

- g) Carry out a sensitivity calculation considering actual operational discharge less than the design discharge, and neglecting hydraulic flushing, to determine the ageing discharge capacity.
- h) If the water is aggressive to concrete (soft water), the initial operational discharge should ensure laminar sub-layer formation (Figure 5.1-4) as the tunnel ages, but with a minimum operational discharge given by the probable ultimate roughness (eq. 5.8.5-2) calculation or required roughness (capacity).
- i) If the water is not aggressive to concrete, make the dominant discharge as high as possible to limit ageing. The ageing can in some cases be reversed by hydraulic flushing at high discharge on an annual basis. The flushing operation should however be seen as a safety factor in the design since it will not work under all conditions. It is better to make the dominant discharge as high as possible.
- j) Determine the critical hydraulic condition for re-entrainment of non-cohesive sediment that could enter the tunnel and check whether deposited sand and gravel could be re-entrained during the dominant discharge (eq. 5.5.1-1).
- k) Calculate the sediment transport capacity of the tunnel at the dominant discharge for non-cohesive sediment (eq. 5.6-1).

- l) In soft water conditions, evaluate the maximum bed shear stress (and stream power) which should not exceed 10 Pa to prevent damage to the soft concrete surface (Table 5.5.2-1).

Comments on ageing remedial measures:

- m) Mechanical cleaning should not be considered for soft water conditions, and only soft brushes should be used in hard water conditions to prevent damage to the concrete surface. Drying of deposited sediment will remove it, but this should not be carried out in soft water conditions as the surface layer of concrete will also be removed.
- n) Hydraulic flushing should only be considered in hard water conditions. It is however better to make the dominant discharge as high as possible instead of allowing excessive ageing at first and then relying on flushing to reverse the ageing process
- o) Relining of concrete tunnels should not be necessary under most conditions, as long as a good quality concrete is used and the soft concrete is not scoured (in soft water conditions).
- p) When designing a tunnel, limit secondary losses as far as possible, but conduct a cost analysis. Also consider phased partial or complete doubling of the tunnel.

11 CONCLUSIONS

This study combined laboratory and field test data to obtain a better understanding of the ageing process in large diameter conduits such as water transfer tunnels. Field tests were carried out since 1998 at the Orange-Fish Tunnel, Roode Elsberg Tunnel and Theewaterskloof Tunnel, while laboratory investigations at the Universities of Stellenbosch and of Pretoria were also carried out on pipes. The following are key findings of this research project:

- Hydraulic ageing is caused by sediment deposition consisting of fine cohesive sediment, in some cases also containing some organic material. Manganese deposition by bacterial action also occur which is difficult to flush out at high discharge. In the Theewaterskloof Tunnel sand at the tunnel invert also deposited due to diversion of Berg River water into the tunnel.
- Soft water corrosion occurred at the Western Cape and LHWP tunnels. The effect is a softening of the upper 3 mm of concrete, but this was protected by a thin layer of sediment at all the tunnels investigated. The soft water corrosion can however lead to deep scour holes and etch marks in the concrete, where a poor quality concrete was used during construction (Theewaterskloof Tunnel). This is however not expected in the high strength LHWP concrete lining.
- A relationship was developed and calibrated to determine the effective roughness of a segmentally lined tunnel, including the joint step losses.
- A hydraulic rate of ageing equation was developed and calibrated on UK and SA data, for conditions where sediment deposition and sliming plays a role. The pH of the water is a dominant variable.
- An ultimate roughness equation was developed and calibrated. Probable and maximum probable ultimate roughness formulations have been proposed.
- Hydraulic flushing at high flows are effective in reducing the hydraulic roughness by flushing out sediment deposits from the tunnel. Where bacterial action however plays a role one cannot rely on flushing alone and other measures have to be considered.
- Chlorination has been used on a limited scale elsewhere in the world to reduce roughness, but was found more effective after cleaning of the tunnel. Depending on the system configuration there are environmental problems associated with chlorination.
- Mechanical cleaning of a tunnel in soft water conditions damages the tunnel and increase the roughness, no matter how careful you are. In hard water conditions, high pressure water jets damaged the tunnel surface. Literature indicate rapid increase in roughness after cleaning, within one to two years.
- Tunnel linings such as poly-urethane can be considered in conditions of soft water corrosion, but should preferably be applied during the tunnel construction. These lining are however expensive and their lifespan is unknown. A good quality concrete without a lining, designed and operated correctly, should be sufficient for most conditions.

- Literature reviewed indicates that there are two main factors influencing the deterioration of the concrete due to soft water attack namely the aggressiveness of the water and the composition of the concrete under attack.

No cement extenders were used in the concrete of the Theewaterskloof Tunnel and it was found that the leaching rate of Ca-ions increase exponentially with an increase in the water operating velocity in the pipe system. The main chemical process taking place in the concrete during soft water attack in the Theewaterskloof Tunnel is the leaching of Ca(OH)_2 . A maximum and minimum theoretical concrete deterioration depth rate was determined for the concrete. These rates also increase exponentially with an increase with water operating velocity in the pipe system. The permeability of this concrete is very high and therefore the leaching rates are also high.

The concrete mixture of the LHWP linings of the delivery tunnels contains fly-ash as a cement extender. Two different Ca-ion leaching rates for a specific water operating velocity were found for the LHWP concrete. It indicates that as a result of the fly-ash content in the mixture, decalcification of the C-S-H gel take place at a low rate and Ca(OH)_2 leaching at a higher rate. The theoretical concrete deterioration depth rates for the LHWP concrete are much lower than the rates for the Theewaterskloof Tunnel. There are not enough data available at this stage but it seems that the concrete deterioration rate increase with an increases of water operating velocity.

Although the soft water attack cannot be stopped with a specific concrete mix design, it is possible to decrease it significantly with an optimum design. The optimum concrete mixture as designed by Castro & McIntosh (1994) for the LHWP tunnels will still be the best mixture for resistance to soft water attack because the Ca(OH)_2 content is very low in this design mix.

- Design and operational guidelines for the determination of tunnel ageing and selection of tunnel diameter during design, have been proposed. The LHWP DTN has been used as a case study.
- Prediction of future ageing of the LHWP DTN indicates that under current operating conditions, the rate of ageing would be relatively small and the ultimate discharge capacity would be between 80% to 85% of the new tunnel capacity. Previous studies used a higher ultimate roughness (DWA, 1999) of $k_s = 4 \text{ mm}$ which resulted in an aged discharge capacity of about 73% of the original capacity. The current predictions therefore indicate that it might be possible to implement the next phase of the LHWP without doubling the tunnels: Low Mashai Dam, that would require a total tunnel discharge capacity (including phase I flow) of $35,8 \text{ m}^3/\text{s}$ without downtime, or $39,8 \text{ m}^3/\text{s}$ with 10% downtime.

12 RECOMMENDATIONS

It is recommended that the design guidelines (Chapter 10) are followed during the design of a tunnel. Even more important however, is that the tunnel should be operated according to the design.

- Regular hydraulic tests at say 5 year intervals should be carried out at the LHWP, Theewaterskloof, Orange-Fish and Roode Elsberg Tunnels to determine the hydraulic roughness to extend the current data base.
- The theory developed for this study should be calibrated against field data of pipelines for application in pipe design.

Attention to the following chemical aspects regarding soft water corrosion is necessary:

- More laboratory tests are necessary on the LHWP test section at higher water operating velocities to determine the relationship between the Ca-ions leaching rates and water operating velocities.
- Tests on the microstructure of the concrete of the test sections are necessary to calibrate the rate of the theoretical concrete deterioration depths and to determine the exact influence of the permeability and porosity on soft water attack.
- Tests on the Theewaterskloof Tunnel core have to be completed to calibrate laboratory tests.
- A core from LHWP is required to calibrate laboratory tests.
- The aggressiveness of the water actually flowing through the tunnels should be determined. Laboratory tests with the water from the field will be necessary to confirm the calibration of the accelerated rate of deterioration in the laboratory with the actual field behaviour.
- Further tests on concrete mixtures with silica-fume as cement extender will be valuable to determine a concrete mixture with a higher resistance to soft water attack.
- The effect of soft water in the concrete mix should be studied.
- Coatings should be applied in the Delivery Tunnel North to evaluate its long term performance.
- Tunnels should not be cleaned by mechanical equipment (brushes/brooms) or water jets, as this damages the concrete surface, leading to increased hydraulic roughness, based on field tests carried out during this study.
- The rate of ageing and ultimate hydraulic roughness predictors developed and calibrated in this study make use of hydraulic data and pH values. These equations should however be refined in future by incorporating water quality variables related to soft water corrosion and sliming.

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APPENDICES

APPENDIX A

Softwater corrosion laboratory data

APPENDIX A-1

Test data on the Theewaterskloof Tunnel section

DATE	TIME	TEMPERATURE OF WATER (°C)	UP-STREAM READING (cm)	DOWN- STREAM READING (cm)	DIFFERENCE IN READINGS (cm)
Depth at 90° V-Notch = 185 mm (Velocity 1.2 m/s)					
28/05/2001		17	72	82	10
29/05/2001		26	100	115	15
30/05/2001	09H30	25	105	120	15
30/05/2001	13H00	28	106	120	14
31/05/2001	08H30	28	104	118	14
31/05/2001	12H00	29	106	120	14
01/06/2001	08H30	29	104	118	14
01/06/2001	12H00	29	106	120	14
04/06/2001	09H00	29	103	117	14
04/06/2001	12H45	30	104	117	13
05/06/2001	09H00	30	100	113	13
05/06/2001	12H30	30	101	114	13
06/06/2001	08H15	30	104	117	13
06/06/2001	13H00	30	103	116	13
07/06/2001	8H00	30	104	119	15
11/06/2001	10H15	28	104	118	14
11/06/2001	13H30	29	107	120	13
12/06/2001	10H00	27	104	119	15
12/06/2001	12H45	28	105	121	16
13/06/2001	08H00	27	106	121	15
13/06/2001	12H00	28	106	120	14
14/06/2001	08H00	27	106	120	14
14/06/2001	12H30	28	106	120	14
15/06/2001	08H00	26	103	117	14
15/06/2001	12H00	28	104	117	13
18/06/2001	09H00	27	105	120	15
18/06/2001	12H30	27	108	122	14
19/06/2001	09H15	27	110	125	15
20/06/2001	10H00	26	108	124	16
20/06/2001	13H30	26	115	130	15

DATE	TIME	TEMPERATURE OF WATER (°C)	UP-STREAM READING (cm)	DOWN- STREAM READING (cm)	DIFFERENCE IN READINGS (cm)
Depth at 90° V-Notch = 224 mm (Velocity 1.94 m/s)					
21/06/2001	09H15	25	180	200	20
21/06/2001	13H45	26	180	201	21
22/06/2001	09H00	25	180	203	23
22/06/2001	12H30	25	185	208	23
Distilled water added					
02/07/2001	12H30	25	183	205	22
04/07/2001	08H30	25	179	200	21
04/07/2001	12H30	26	182	205	23
05/07/2001	10H00	25	183	205	22
05/07/2001	12H15	26	187	208	21
06/07/2001	08H30	24	182	204	22
06/07/2001	12H30	25	182	202	20
09/07/2001	09H30	25	182	205	23
09/07/2001	12H00	26	180	202	22
10/07/2001	08H30	26	179	200	21
10/07/2001	13H30	27	183	205	22
11/07/2001	09H30	25	180	202	22
11/07/2001	14H45	28	183	205	22
24/07/2001	09H00	20	178	200	22
25/07/2001	15H00	20	176	196	20
30/07/2001	08H30	21	174	195	21
31/07/2001	08H30	24	170	190	20
01/08/2001	08H00	25	180	202	22
02/08/2001	08H15	25	170	190	20
06/08/2001	13H00	26	172	194	22
07/08/2001	09H30	24	170	190	20
13/08/2001	09H45	25	170	191	21

DATE	TIME	TEMPERATURE OF WATER (°C)	UP-STREAM READING (cm)	DOWN- STREAM READING (cm)	DIFFERENCE IN READINGS (cm)
Depth at 90° V-Notch = 264 mm (Velocity 2.9 m/s)					
14/08/2001	15H30	30	350	391	40
15/08/2001	08H30	29	350	390	41
15/08/2001	14H00	32	347	385	38
16/08/2001	11H00	32	344	382	38
16/08/2001	15H45	33	344	383	39
17/08/2001	09H00	30	342	380	38
17/08/2001	14H00	32	344	376	32
20/08/2001	09H00	32	335	370	35
20/08/2001	13H00	33	333	370	37
21/08/2001	08H30	33	334	370	36
21/08/2001	13H30	34	335	372	37
22/08/2001	08H00	32	338	374	36
22/08/2001	12H30	34	336	370	34
23/08/2001	10H00	32	335	370	35
23/08/2001	14H00	33	336	372	36
24/08/2001	07H45	33	334	370	36
24/08/2001	12H30	34	336	371	35
27/08/2001	10H30	34	337	371	34
27/08/2001	14H30	34	336	373	37
29/08/2001	09H30	32	345	383	38
30/08/2001	14H30	32	345	382	37
30/08/2001	08H00	32	346	382	36
31/08/2001	13H30	31	350	384	34
31/08/2001	09H30	30	350	385	35

APPENDIX A-2

Leaching of Ca-ions from the Theewaterskloof Tunnel section

Date	Time	*[EDTA] mol/l	Volume of water sample	Volume of *EDTA (l)	mol Ca	Ca (g) per 50ml of water	Calcium ppm	pH-values
28-May		0.00125	0.05	0.0036	4.5E-06	0.00018	4.0	6.76
29-May		0.0025	0.05	0.0049	1.23E-05	0.00049	9.8	7.33
30-May		0.00125	0.05	0.0104	0.000013	0.00052	11.6	7.03
31-May		0.00125	0.05	0.01395	1.74E-05	0.000698	15.5	7.22
01-Jun		0.00125	0.05	0.0139	1.74E-05	0.000695	15.4	7.21
04-Jun		0.00125	0.05	0.0149	1.86E-05	0.000745	16.6	7.34
05-Jun		0.00125	0.05	0.01595	1.99E-05	0.000798	17.7	7.37
06-Jun	09:00	0.0025	0.05	0.0081	2.03E-05	0.00081	16.2	7.66
06-Jun		0.00125	0.05	0.01655	2.07E-05	0.000828	18.4	
07-Jun		0.00125	0.05	0.0172	2.15E-05	0.00086	19.1	7.84
11-Jun		0.00125	0.05	0.0198	2.48E-05	0.00099	22.0	7.55
12-Jun		0.0025	0.05	0.0098	2.45E-05	0.00098	19.6	7.73
12-Jun	09:00	0.00125	0.05	0.0197	2.46E-05	0.000985	21.9	
13-Jun		0.00125	0.05	0.0203	2.54E-05	0.001015	22.6	7.58
14-Jun		0.00125	0.05	0.0208	0.000026	0.00104	23.1	7.32
15-Jun	09:00	0.005	0.05	0.0061	3.05E-05	0.00122	24.4	7.64
15-Jun		0.00125	0.05	0.0212	2.65E-05	0.00106	23.6	
18-Jun		0.00125	0.05	0.0235	2.94E-05	0.001175	26.1	7.44
19-Jun	09:15	0.00125	0.05	0.0182	2.28E-05	0.00091	20.2	7.52
20-Jun	09:00	0.005	0.05	0.0057	2.85E-05	0.00114	22.8	7.57
20-Jun		0.00125	0.05	0.0194	2.43E-05	0.00097	21.6	
21-Jun		0.00125	0.05	0.0188	2.35E-05	0.00094	20.9	7.31
22-Jun		0.00125	0.05	0.0194	2.43E-05	0.00097	21.6	7.42
02-Jul		0.0025	0.05	0.0043	1.08E-05	0.00043	8.6	7.14
04-Jul		0.0025	0.05	0.0045	1.13E-05	0.00045	9.0	7.8
04-Jul	08:30	0.0025	0.05	0.0095	2.38E-05	0.00095	19.0	
05-Jul	12:15	0.0025	0.05	0.01	0.000025	0.001	20.0	7.77
06-Jul		0.0025	0.05	0.0048	0.000012	0.00048	9.6	7.31
06-Jul	08:30	0.0025	0.05	0.0094	2.35E-05	0.00094	18.8	
09-Jul	09:30	0.0025	0.05	0.0111	2.78E-05	0.00111	22.2	7.76

Date	Time	*[EDTA] mol/l	Volume of water sample	Volume of *EDTA (l)	mol Ca	Ca (g) per 50ml of water	Calcium ppm	pH-values
10-Jul	13:30	0.0025	0.05	0.0115	2.88E-05	0.00115	23.0	7.8
11-Jul	14:30	0.0025	0.05	0.0117	2.93E-05	0.00117	23.4	7.92
24-Jul	15:00	0.0025	0.05	0.0167	4.18E-05	0.00167	33.4	7.38
30-Jul		0.0025	0.05	0.0058	1.45E-05	0.00058	11.6	7.74
31-Jul	08:30	0.0025	0.05	0.0117	2.93E-05	0.00117	23.4	7.67
01-Aug	08:00	0.0025	0.05	0.012	0.00003	0.0012	24.0	7.38
02-Aug	08:15	0.0025	0.05	0.0124	0.000031	0.00124	24.8	7.88
06-Aug	13:00	0.0025	0.05	0.014	0.000035	0.0014	28.0	7.38
07-Aug	09:30	0.0025	0.05	0.0146	3.65E-05	0.00146	29.2	7.31
13-Aug		0.0025	0.0045	0.0047	1.18E-05	0.00047	10.4	
14-Aug	15:30	0.0025	0.05	0.0122	3.05E-05	0.00122	24.4	7.14
15-Aug	14:00	0.0025	0.05	0.013	3.25E-05	0.0013	26.0	7.28
16-Aug	15:45	0.0025	0.05	0.0134	3.35E-05	0.00134	26.8	7.23
17-Aug	09:00	0.0025	0.05	0.014	0.000035	0.0014	28.0	7.41
17-Aug	14:00	0.0025	0.05	0.0136	0.000034	0.00136	27.2	
20-Aug	13:00	0.0025	0.05	0.0166	4.15E-05	0.00166	33.2	7.34
21-Aug	13:30	0.0025	0.05	0.0168	0.000042	0.00168	33.6	7.38
22-Aug	12:30	0.0025	0.05	0.0163	4.08E-05	0.00163	32.6	8.01
23-Aug	10:00	0.0025	0.05	0.0179	4.48E-05	0.00179	35.8	7.37
23-Aug	14:00	0.0025	0.05	0.0173	4.33E-05	0.00173	34.6	
24-Aug	12:30	0.0025	0.05	0.0179	4.48E-05	0.00179	35.8	7.59
27-Aug	14:30	0.0025	0.05	0.0218	5.45E-05	0.00218	43.6	7.37
28-Aug	11:00	0.0025	0.05	0.0159	3.98E-05	0.00159	31.8	7.67
29-Aug	14:30	0.0025	0.05	0.0103	2.58E-05	0.00103	20.6	7.12
30-Aug	13:30	0.0025	0.05	0.0089	2.23E-05	0.00089	17.8	7.21
31-Aug	09:30	0.0025	0.05	0.0054	1.35E-05	0.00054	10.8	7.5
03-Sep	14:00	0.0025	0.05	0.0067	1.68E-05	0.00067	13.4	6.99
04-Sep	09:30	0.0025	0.05	0.0066	1.65E-05	0.00066	13.2	7.01

* [EDTA] Concentration of Di-sodium Ethylenediamine Tetraacetate

APPENDIX A-3

Extraction from

Portland Cement Institute:

Tip no. 169

Predicting the Ca(OH)_2 content and chemical vulnerability of pastes made with different binders.

B J Addis

February 1989

Assumptions

- The calcium silicate hydrate (C-S-H) is always of the form $\text{C}_3\text{S}_2\text{H}_3$, no matter which binder it is derived from.
- The calcium silicate compounds in Portland cement hydrate fully.

Atomic Masses

Ca = 40

O = 16

H (hydrogen) = 1

Si = 28

Al = 27

Fe = 56

Therefore: Ca(OH)_2 = CH = $40 + 16 + 16 + 1 + 1 = 74$

SiO_2 = S = $28 + 16 + 16 = 60$

CaO = C = $40 + 16 = 56$

H_2O = H = $1 + 1 + 16 = 18$

Al_2O_3 = A = $27 + 27 + 16 + 16 + 16 = 102$

Fe_2O_3 = F = $56 + 56 + 16 + 16 = 160$

C_3S = $56 + 56 + 56 + 60 = 228$

C_2S = $56 + 56 + 60 = 172$

C_3A = $56 + 56 + 56 + 102 = 270$

C_4AF = $56 + 56 + 56 + 56 + 102 + 160 = 486$

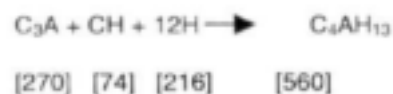
Reactions – Portland cement



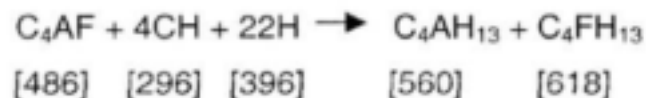
Therefore 100 kg C₃S produces 75 kg C₃S₂H₃ and 48,7 kg CH



Therefore: 100 kg C₂S produces 99,4 kg C₃S₂H₃ and 21,5 kg CH

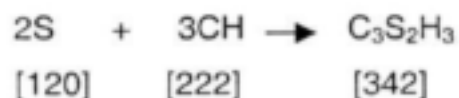


Therefore: 100 kg C₃A consumes 27,4 kg CH



Therefore: 100 kg C₄AF consumes 60,9 kg CH

Fly-ash (pozzolanic reaction)



Therefore 100 kg of S reacts with 185 kg CH to produce 285 kg C₃S₂H₃

APPENDIX A-4

Test data on the LHWP Tunnel section

Date	TIME	TEMPERATURE OF WATER (°C)	UP-STREAM READING (cm)	DOWN-TREAM READING (cm)	DIFFERENCE IN READINGS (cm)
Depth at 90° V-Notch = 220 mm (Velocity 1.9 m/s)					
11-Sep-01	12H00		177	198	21
12-Sep-01	09H00	25	174	197	23
12-Sep-01	12H30	24	175	195	21
13-Sep-01	09H00	23	174	195	20
13-Sep-01	13H30	19	179	200	21
14-Sep-01	09H30	18	175	194	19
14-Sep-01	12H30	18	173	193	20
17-Sep-01	09H00	26	169	188	19
17-Sep-01	12H30	27	170	189	19
18-Sep-01	08H30	25	172	190	18
18-Sep-01	14H30	25	173	191	18
19-Sep-01	09H00	24	172	189	17
19-Sep-01	14H00	24	169		18
20-Sep-01	07H30	25	171	189	18
20-Sep-01	14H00	24	174	192	18
21-Sep-01	07H30	24	172	190	18
21-Sep-01	13H00	25	173	192	19
Depth at 90° V-Notch = 220 mm (Velocity 1.90 m/s)					
27-Sep-01			170	186	16
28-Sep-01	10H00	20	185	206	21
15-Oct-01	10H30		225	250	25
16-Oct-01	08H00	26	227	257	30
17-Oct-01	08H00	27	225	255	30
19-Oct-01	08H00	25	225	254	29
22-Oct-01	11H30	28	227	256	29
23-Oct-01	09H00	29	227	255	28
24-Oct-01	08H00	28	225	252	27
25-Oct-01	08H00	29	225	251	26

Date	TIME	TEMPERATURE OF WATER (°C)	UP-STREAM READING (cm)	DOWN-TREAM READING (cm)	DIFFERENCE IN READINGS (cm)
26-Oct-01	12H00	27	227	253	26
29-Oct-01	11H00	25	228	254	26
30-Oct-01	11H30	26	226	251	25
31-Oct-01	14H30	25	227	252	25
01-Nov-01	08H00	26	226	252	26
02-Nov-01	08H00	26	228	254	26
13-Nov-01			370	413	43
Depth at 90° V-Notch = 257 mm (Velocity 2,73 m/s)					
14-Nov-01	14H00	25	394	424	30
15-Nov-01	11H00	26	395	424	29
16-Nov-01	08H00	25	388	417	29
20-Nov-01	10H00	25	287	317	30

APPENDIX A-5

Leaching of Ca-ions from the LHWP Tunnel section

Date	*[EDTA] mol/ℓ	Volume of water sample	Volume of *EDTA (ℓ)	mol Ca	Ca (g) per 50ml of water	Calcium ppm	pH-values
11-Sep	0.00125	0.05	0.00855	1.06875E-05	0.000428	9.499905	
12-Sep	0.00125	0.05	0.01775	2.21875E-05	0.000888	19.72203	7.4
13-Sep	0.00125	0.05	0.0189	0.000023625	0.000945	20.99979	7.39
14-Sep	0.00125	0.05	0.01915	2.39375E-05	0.000958	21.27757	7.52
17-Sep	0.00125	0.05	0.0209	0.000026125	0.001045	23.22199	8.13
18-Sep	0.00125	0.05	0.023	0.00002875	0.00115	25.5553	7.67
19-Sep	0.00125	0.05	0.0256	0.000032	0.00128	28.44416	7.63
20-Sep	0.00125	0.05	0.03015	3.76875E-05	0.001508	33.49967	7.82
21-Sep	0.00125	0.05	0.0165	0.000020625	0.000825	18.33315	7.99
28-Sep	0.00125	0.05	0.0196	0.0000245	0.00098	21.77756	6.98
01 Okt	0.00125	0.05	0.0174	0.00002175	0.00087	19.33314	7.92
15-Oct	0.00125	0.05	0.0232	0.000029	0.00116	25.77752	7.76
16-Oct	0.00125	0.05	0.0244	0.0000305	0.00122	27.11084	7.77
17-Oct	0.00125	0.05	0.0252	0.0000315	0.00126	27.99972	7.03
19-Oct	0.00125	0.05	0.0155	0.000019375	0.000775	17.22205	7.65
22-Oct	0.00125	0.05	0.0183	0.000022875	0.000915	20.33313	7.84
23-Oct	0.00125	0.05	0.0186	0.00002325	0.00093	20.66646	7.81
24-Oct	0.00125	0.05	0.0173	0.000021625	0.000865	19.22203	7.77
25-Oct	0.00125	0.05	0.0184	0.000023	0.00092	20.44424	7.78
26-Oct	0.00125	0.05	0.0194	0.00002425	0.00097	21.55534	7.92
29-Oct	0.00125	0.05	0.0217	0.000027125	0.001085	24.11087	8.2
30-Oct	0.00125	0.05	0.0231	0.000028875	0.001155	25.66641	8.17
31-Oct	0.00125	0.05	0.026	0.0000325	0.0013	28.8886	8.17
01-Nov	0.00125	0.05	0.0259	0.000032375	0.001295	28.77749	8.15
02-Nov	0.00125	0.05	0.0284	0.0000355	0.00142	31.55524	8.23
14-Nov	0.00125	0.05	0.0206	0.00002575	0.00103	22.88866	7.73
15-Nov	0.00125	0.05	0.0203	0.000025375	0.001015	22.55533	7.76
16-Nov	0.00125	0.05	0.0194	0.00002425	0.00097	21.55534	8.01
20-Nov	0.00125	0.05	0.023	0.00002875	0.00115	25.5553	8.37

* [EDTA] Concentration of Di-sodium Ethylenediamine Tetraacetate

APPENDIX A-6

Laboratory test pipe corrosion data

		Pump running		Samples taken	Measurements			Water volume		
Date	Time	Duration	Acc duration	Temperature	Overflow height (mm)	Flow (l/s)	Velocity	Depth at pump	Water added (l)	Total volume (l)
3-Oct	10:10			22°C						5100.00
	12:00	0.08	0.08	23°C	140	10.57	0.60			5055.01
	13:00	0.04	0.12	24°C	180	19.81	1.12			5030.48
	14:30	0.06	0.18	24°C	186	21.50	1.22			4993.67
	15:45	0.05	0.23	26°C	185	21.21	1.20			4963.00
4-Oct	8:15			24°C	188	22.08	1.25			
	12:45	0.19	0.42	26°C	185	21.21	1.20			4852.58
	17:00	0.18	0.60	27°C						4748.30
5-Oct	8:00			25°C	185	21.21	1.20			
	14:45	0.28	0.88	29°C						4582.67
	17:00	0.09	0.97	30°C						4527.46
6-Oct	8:20			27°C						
	12:45	0.18	1.16	30°C						4419.08
	18:00	0.22	1.38	32°C						4290.26
7-Oct	11:00			29°C	186	21.50	1.22			
	17:00	0.25	1.63	30°C						4143.04
8-Oct	10:30			26°C	185	21.21	1.20			
	18:30	0.33	1.96	26°C						3946.74
9-Oct	8:15			24°C	185	21.21	1.20			
	12:00	0.16	2.11	28°C						3854.72
	17:00	0.21	2.32	29°C						3732.04
10-Oct	8:00									
	18:00	0.42	2.74							3486.66
11-Oct	7:45									
	12:00	0.18	2.92							3382.38
	18:45	0.28	3.20							3216.75
12-Oct	8:30									
	17:20	0.37	3.57							3000.00

Date	Time	Pump running		Samples taken	Measurements			Water volume		
		Duration	Acc duration	Temperature	Overflow height (mm)	Flow (l/s)	Velocity	Depth at pump	Water added (l)	Total volume (l)
14-Oct	10:00								2100	5100.00
16-Oct	7:30	1.90	5.46							4191.34
	18:30	0.46	5.92							3971.66
17-Oct	7:45	0.55	6.47							3707.05
	17:00	0.39	6.86							3522.32
18-Oct	7:30	0.60	7.46							3232.75
20-Oct	7:15			27°C					1867.25	5100.00
21-Oct	11:00	1.16	8.62	30°C	185	21.21	1.20	650		4998.15
23-Oct	17:00	2.25	10.37	28°C	177	18.99	1.07	520		4556.80
24-Oct	6:00	0.54	11.41	29°C	178	19.26	1.09	460		4353.10
	17:00	0.46	11.87	33°C						4200.00
25-Oct	7:45	0.61	12.48	30°C	178	19.26	1.09	370		4047.55
	16:00	0.34	12.83	34°C						3850.00
26-Oct	9:00	0.71	13.53	30°C	178	19.26	1.09	280		3742.00
	17:30	0.35	13.89	31°C				200		3470.40
28-Oct	9:00			25°C				580	1290.1	4760.50
30-Oct	7:30	1.94	15.83	31°C	179	19.53	1.11	400		4149.40
	17:15	0.41	16.23	33°C						4020.00
31-Oct	7:00	0.57	16.81	29°C	177	18.99	1.07	310		3843.85
	16:30	0.40	17.20	31°C						3750.00
1-Nov	7:45	0.64	17.84	29°C	176	18.73	1.06	240		3606.20
	15:00	0.30	18.14	29°C						3500.00

APPENDIX A-7

CaO content in concrete sample of core drilled at Theewaterskloof Tunnel

Sample	Mass	Vol solution (ml)	[EDTA]	Vol EDTA	Moles CaO	CaO (g)	CaO/100g sample
1 (0-3mm)	5	250	0.02	0.031	0.0062	0.3472	6.944
2 (7-10mm)	5	250	0.02	0.0425	0.0085	0.476	9.52
3 (14-17mm)	5	250	0.02	0.0425	0.0085	0.476	9.52
4 (21-24mm)	5	250	0.02	0.0375	0.0075	0.42	8.4
5 (28-32mm)	5	250	0.02	0.0358	0.00716	0.40096	8.0192
6 (35-39mm)	5	250	0.02	0.0422	0.00844	0.47264	9.4528
7 (42-46mm)	5	250	0.02	0.0375	0.0075	0.42	8.4
8 (48-53mm)	5.51	250	0.02	0.041	0.0082	0.4592	9.184
9 (56-60mm)	5	250	0.02	0.0449	0.00898	0.50288	10.0576
10 (63-67mm)	5.179	250	0.02	0.0358	0.00716	0.40096	8.0192
11 (210-213mm)	5	250	0.02	0.037	0.0074	0.4144	8.288

APPENDIX B

Biological analysis on LHWP DTN sediment

Results and report-back: analyses performed on sediment sample

Ref. - [Project: WRC tunnels (microbiological and SEM analyses)]

It is my pleasure to inform you of the results obtained after analyses performed on a sediment sample delivered to me on 28 October 1999 by yourself. The sample was a dark sediment in the form of a slurry from a concrete pipe.

A) Materials and Methods:

The following analyses were performed on the sample:

Microbiological analyses:

- a) total aerobic bacterial count using Nutrient Agar (Merck) with 50 mg/l cycloheximide to inhibit fungal and yeast growth;
- b) total anaerobic heterotrophic count using Reinforced Clostridial Medium (Merck) with titanium chloride (Merck) as a reducing agent;
- c) total yeast and fungal count using Potato Dextrose Agar (Merck) with 250 mg/l chloramphenicol to inhibit bacterial growth;
- d) Sulphate Reducing Bacteria (SRB) using Iron Sulphite agar (Oxoid) with titanium chloride as a reducing agent;
- e) *Thiobacillus ferrooxidans* count using ferrous sulphate medium (Karaivko *et al.*, 1977);

All counts were performed after incubation for two weeks at room temperature, except the total aerobic bacterial count and the total yeast and fungal count, performed after incubation at room temperature for 48 hours.

Scanning Electron Microscopy (SEM):

SEM was performed using standard preparation procedures (2.5% glutaraldehyde as fixative in 0.075 M Phosphate Buffered Saline (PBS), immobilization of sample on 0.2µm filter, washing with 0.075 M PBS, drying with ethanol series, critical point drying, coating with gold).

B) Results and discussion:Table 1: Results for analyses done on sediment sample.

Analysis	Average	Standard deviation
Total aerobic bacterial count (cfu/g)	1.52×10^6	2.53×10^5
Total anaerobic heterotrophic count (cfu/g)	2.28×10^4	3.77×10^3
Yeast count (cfu/g)	1.45×10^3	6.19×10^2
Fungal count (cfu/g)	5.00×10^3	8.28×10^2
SRB count (cfu/g)	2.00×10^4	4.08×10^3
<i>Thiobacillus ferrooxidans</i> count (cfu/g)	<100	-

For the SEM work, photos and digital images were obtained. Photo negatives and contact prints, as well as digital images on a CD-ROM are included with this report.

SEM indicated the presence of at least three kinds of diatoms, as well as yeasts, fungal hyphae and rod-shaped and coccoid bacterial cells (Figures 1 to 4).

Figure 1: Rod shaped and coccoid bacterial cells.



Figure 2: A yeast cell.

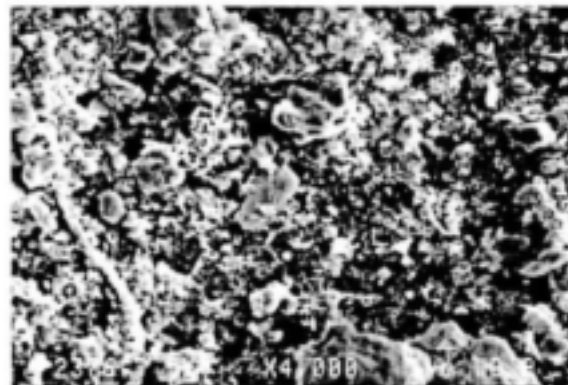


Figure 3: A rod-shaped cell.



Figure 4: A diatom.



Figure 5: Rod-shaped bacterial cells

The aerobic bacterial count was high (1.52×10^6 cfu/g). At least nine different bacterial species could be observed macroscopically.

Anaerobic heterotrophic counts (2.28×10^4 cfu/g) were almost two log-units lower than the total aerobic bacterial counts. Anaerobic growth at low dilutions was dominated by fermenters, with high levels of gas produced. At higher dilutions, non-fermenters dominated with little gas produced. Microscopic observation indicated the incidence of rod-shaped spore formers with round to oval sub-terminal spores.

A low incidence of yeasts and fungi was observed (1.45×10^3 and 5.00×10^3 cfu/g respectively). Macroscopically, at least seven fungal and one yeast species could be observed.

A low incidence of SRB was observed (2.00×10^4 cfu/g). These organisms are able to use oxidized sulphur compounds and reduce these to sulfide which can react with metals to form black precipitates of metal sulphides, thereby causing microbially induced corrosion.

No growth of *Thiobacillus ferrooxidans* was observed. The incidence of organic material in the agar used to select for these autotrophic organisms could have inhibited their growth.

Concrete can be dissolved by thiobacilli, causing major structural damage to highways and concrete sewage lines (Prescott *et al.*, 1990). In the presence of sufficient moisture and pyritic sulphur compounds of lower oxidation state than sulphate, these microbes produce sulphuric acid, resulting in the dissolution of concrete. Thiobacilli can be controlled by the selection of cements that do not contain appreciable levels of oxidizable sulphur compounds, the avoidance of contamination of concrete with soil microbes, and the use of organic inhibitors.

In general, there was concern over sampling and transport of the sediment sample. Firstly, it would be advisable to use a sterile container for sampling, as well as using aseptic sampling techniques. Secondly, samples should be kept in a cold chain of 4°C until delivery to a suitable lab for analyzing. There were doubts about the two prerequisites stated above. Also, to get a representative sample of a biofilm in a concrete pipe, it would be advisable to scrape the biofilm off the top and sides of the pipe sampled without including interfering sediment in the sample.

With all the points above taken into consideration, there are doubts as to the validity of all the analyses performed on the sample in regard to biodeterioration of the concrete pipe and results should be seen only as an indication of conditions inside the pipe as well as inside the sediment in the pipe. No final conclusions about the role of the sediment itself should be drawn to the possible biodegradation of the concrete pipe by microbes. Other phenomena like structural defects as well as speed of water flow, amount of sediment etc. should also be taken into consideration.

It is likely that not only *Thiobacillus ferrooxidans* and other thiobacilli, but a variety of organisms selected for in a biofilm could cause biodeterioration of concrete, mainly by production of secondary metabolites like organic acids by anaerobic metabolism inside the biofilm (Prescott *et al.*, 1990).

The lack of *Thiobacillus ferrooxidans* does not indicate the absence of other thiobacilli. It is suggested that more sensitive methods be used in future to indicate the presence of thiobacilli. These methods include the use of statistical methods to determine the Most Probable Number (MPN) or molecular probing techniques like Fluorescent In Situ Hybridization (FISH).

C) Conclusions:

No final conclusions about the role of microbes in the biodeterioration of concrete pipes must be drawn from the analyses performed on the sediment sample provided. A much more detailed and in-depth study should be conducted to investigate the possibility of specific microbes causing microbially induced corrosion of the concrete pipe. The high microbial numbers in the sediment sample does indicate the possibility of anaerobic microbes forming part of anaerobic biofilms and, thus, possible biocorrosion of the concrete pipe. It is also suggested that wash-down of sediment and soil be minimized by whatever means possible as the presence of sediments in the pipe creates a habitat suitable for the growth of microbes. Manual cleaning of pipes (for example pigging) and/or the possible introduction of organic inhibitors or non-ionic biocides can also be recommended.

DJ Oosthuizen.

D) References:

Prescott, L.M., Harley, J.P. and Klein, D.A. (1990). Microbiology. Wm. C. Brown Publishers, Iowa.

Karaivko, G.I., Kuznetsov, S.I. and Golonizik, A.I. (1977). The bacterial leaching of metals from ores. Technicopy, England.

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Environmentally significant morphological and hydraulic characteristics of cobble and boulder bed rivers in the Western Cape

V Jonker, A Rooseboom & AHM Görgens

This report presents the result of current research that was initiated in order to determine the environmentally significant morphological and hydraulic characteristics of cobble and boulder bed rivers in the Western Cape. With the increasing development of water-related infrastructure in mountain regions, knowledge of the hydraulic characteristics of rivers in the upper catchment areas is very important. Cobble and boulder bed rivers in the Western Cape are typical examples. They are characterized by steep gradients, great variability in sediment size and relatively low flow depths. The bed configuration displays a series of pools, steps, rapids, riffles and plane beds, while energy losses are high as a result of turbulence and local hydraulic jumps. Due to their characteristic morphological and associated hydraulic attributes, the physical habitats within these rivers are extremely diverse, both on a spatial and temporal scale.

Report Number: 979/1/01

ISBN: 1 86845 735 4

Development and implementation of biological cleaning techniques for ultrafiltration and reverse osmosis membranes fouled by organic substances

P Swart, A Maartins, J Engelbrecht, Z Allie and EP Jacobs

The project aimed to investigate and ameliorate the fouling of polysulphone ultrafiltration membranes used in the treatment of Cape brown waters, pulp and paper effluent, and abattoir effluent. The water and effluents were characterised and enzymes and other cleaning methods investigated for the cleaning of fouled membranes. In respect of the brown water and pulp and paper effluents, surface active and fouling prevention agents were evaluated and successfully applied. In the case of ultrafiltration membranes fouled in abattoir effluent, it was found that a blend of lipase and protease enzymes was effective as a cleaning agent to restore the membrane flux.

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