Proceedings of STRATEGIC HIGHWAY RESEARCH PROGRAM AND TRAFFIC SAFETY ON TWO CONTINENTS in Gothenburg, Sweden, 27-29 September, 1989

- Highway Operations
- Concrete and Structures
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PREFACE

The Swedish Road and Traffic Research Institute (VTI) and the US Transportation Research Board (TRB) of the National Research Council were jointly organising this international conference. The objective was to cover the present and future road research with special emphasis on the Strategic Highway Research Program (SHRP), as well as the research concerning drivers and vehicles as related to highway safety.

Under development for 2-3 years, SHRP is a fully funded, $150 million (US), five year program of research directed at asphalt, concrete and structures, highway operations, and long term pavement performance.

In the sessions on roads there were presentations which highlighted differences between European and US practices and needs, and the discussions were concentrated on how to promote international involvement in SHRP and application of its research, within the areas of Asphalt, Long Term Pavement Performance (LTPP), Highway Operations and Concrete and Structures.

In the different road safety sessions there were presentations of actual research in different countries and discussions of the differences that exist between Europe and the USA, trying to explain the reasons for them and examine whether they are reasonable and acceptable.

Linköping October 1989

Kenneth Asp

Proceedings of STRATEGIC HIGHWAY RESEARCH PROGRAM AND TRAFFIC SAFETY ON TWO CONTINENTS in Gothenburg, Sweden, 27-29 September 1989:

VTI RAPPORT 349A
- Opening
- International Harmonization of Test Procedures and Requirements for Roadside Safety Features, Workshop

VTI RAPPORT 350A
- Asphalt
- Long Term Pavement Performance

VTI RAPPORT 351A
- Work Zone Safety
- Heavy Truck Safety
- Highway Safety

VTI RAPPORT 352A
- Highway Operations
- Concrete and Structures

VTI RAPPORT 352A
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## HIGHWAY OPERATIONS

### Overview of SHRP's Highway Operations Research

Don M Harriott, Strategic Highway Research Program (SHRP), USA

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### Evaluation of Effectiveness of Pavement Preventive Maintenance

Roger E Smith, Texas A&M University, and Robert L Lytton, Texas Transportation Institute, USA

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### ERASME - An Expert System for Pavement Maintenance

Pierre Joubert, SETRA, and F Allez, CETE Méd, France

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### Use of Rubber Modified Asphalt for Snow and Ice Control

Hossein B Takallou, CTAK Associates, R Gary Hicks, Oregon State University, and Mojie B Takallou, University of Portland, USA

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## CONCRETE AND STRUCTURES

### SHRP's Concrete and Structures Research: Goals and Recent Developments

Damian J Kulash, Strategic Highway Research Program (SHRP), USA

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<td>Andreas Volkwein, Institute of Building Materials, R Petri and R Springenschmid, Technical University of Munich, Federal Republic of Germany</td>
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ABSTRACT

Papers presented at the seminar were as follows: Overview of SHRP's Highway Operations Research (Harriott, D M); Evaluation of Effectiveness of Pavement Preventive Maintenance (Smith, R E and Lytton, R L); ERASME - An Expert System for Pavement Maintenance (Joubert, P and Allez, F); Use of Rubber Modified Asphalt for Snow and Ice Control (Takallou, H B, Hicks, R G and Takallou, M B); SHRP's Concrete and Structures Research: Goals and Recent Developments (Kulash, D J); Concrete Microstructure Research and Its Applications in Highway Constructions (Idorn, G M); Chloride Removal and Corrosion Protection of Reinforced Concrete (Miller, J B); Chloride-Binding in Cement and the CL/OH-Ratio of the Pore Solution (Tritthart, J); Protection of Reinforcing Steel against Corrosion upon Chloride Impact (Hartl, G); Protecting Concrete by Flexible Waterproofing Slurries (Volkwein, A, Petri, R and Springenschmid, R).
STRATEGIC HIGHWAY RESEARCH PROGRAM AND TRAFFIC SAFETY ON TWO CONTINENTS

Gothenburg, Sweden

September 27-29, 1989

WEDNESDAY SEPTEMBER 27

OPENING

9.00 - 11.30

Chairman: Mrs Monica Sundström, Director General, Swedish Road and Traffic Research Institute (VTI), Sweden

Opening Speeches
Mr Åke Norling, County Governor, Gothenburg, Sweden
Mr Thomas B Deen, Executive Director, Transportation Research Board (TRB), USA
Mrs Monica Sundström, Director General, Swedish Road and Traffic Research Institute (VTI), Sweden

Overview of Traffic Safety in the United States
Mr Marshall Jacks Jr (Mr R Clarke Bennett, Director, Office of Highway Safety (FHWA), USA)

European Trends in Road Safety Research
Mr David Cornelius, Executive Director, Transport and Road Research Laboratory (TRRL), United Kingdom

Highways for the Future: The Strategic Highway Research Program (SHRP)
Dr Damian J Kulash, Executive Director, Strategic Highway Research Program, USA

Need for Road Maintenance Research in the Nordic Countries
Mr Tord Lindahl, Research director, VTI, Sweden
WEDNESDAY SEPTEMBER 27

INTERNATIONAL HARMONIZATION OF TEST PROCEDURES AND REQUIREMENTS FOR ROADSIDE SAFETY FEATURES, WORKSHOP

11.30 - 17.00

Chairman: Mr Thomas Turbell, VTI, Sweden

Status Reports on Current Regulations
Australia Mr Rod Troutbeck, ARRB
Canada Mr Randy Sanderson, Transport Canada
Mr Frank DeVisser, Ministry of Transport
France Mr Robert Quincy, INRETS
Italy Mr Francesco La Camera, La Sapienza
Mr Pasquale Colonna (written presentation only)
Netherlands Mr Tom Heijer, SWOV (including presentation of VEDYAC)
Sweden Mr Thomas Turbell, VTI
United States Mr Jarvis Michie, Dynatech

12.30 LUNCH

14.00

NCHRP Report 230 Update
Dr Hayes Ross, Texas Transportation Institute (TTI), USA

Current Issues in Work Zone Safety
Mr William Hunter, Highway Safety Research (HSRC), USA

Harmonization Within the European Common Market
Mr Thomas Turbell, VTI, Sweden

Manufacturers Views on Harmonization
Mr Hans Norin, Volvo Car Corporation, Sweden
Mr Mike Dreznes, Energy Absorption Systems, USA

Panel Discussion
Subjects: Requirements, Severity Index, Flail space, Theoretical Head Impact Velocity
Belted car occupants
Side impacts

Introduction by: Mr Gordon Bacon, MIRA
Introduction by: Mr William Hunter, HSRC
Introduction by: Mr Malcolm Ray, Vanderbilt University

Summary
Mr Thomas Turbell, VTI, Sweden
IV

WEDNESDAY SEPTEMBER 27

ASPHALT

11.30 - 17.00

Chairman: Mr Bo Liljedahl, The Swedish Asphalt Pavement Association, Sweden

SHRP's Asphalt Research: Progress and Products
Dr Edward T Harrigan, Strategic Highway Research Program, USA

Asphalt Research in the Finnish ASTO-project
Dr Asko Saarela, Technical Research Centre, Finland

Data Distributions for Asphalt Concrete Resilient Modulus and Indirect Modulus
Ms Mary Stroup-Gardiner (Prof David Newcomb, University of Minnesota, USA)

Bitumen and Asphalt Mixes in USA, how do they look like?
Mr Andre Gastmans, Nynas NV, Belgium

What can Europe Learn from SHRP Asphalt Research?
Panel discussion

THURSDAY SEPTEMBER 28

WORK ZONE SAFETY

9.30 - 13.00

Chairman: Mr R Clarke Bennett, Director, Office of Highway Safety (FHWA), USA

Work Zone Safety
Mr Rudolph M Umbs, Federal Highway Administration, USA (Mr R Clarke Bennett)

Analysis of Driver Behaviour and Accidents at Work Sites on German Motorways
Priv Doz Dr-Ing Wilhelm Kockelke, Bundesanstalt für Strassenwesen (BAST), Federal Republic of Germany

A Coordinated Approach to Traffic Control Management Through Multiple Highway Construction Contracts
Mr Hermann Guenther, Project manager, Daniel, Mann, Johnson and Mendenhall, USA

Safety at Roadworks
Mr Michael Marlow, Transport and Road Research Laboratory (TRRL), United Kingdom

Service Vehicle Warning Light Systems for Work Zones
Dr Richard F Pain, Transportation Research Board, USA

VTI RAPPORT 352A
THURSDAY SEPTEMBER 28

HEAVY TRUCK SAFETY

14.00 - 17.30

Chairman: Mr Robert E Spicher, Director, Technical Activities, TRB, USA

Analysis of Statistical Data on HFV-Accidents and Vehicle Measures to Improve Safety
Dr-Ing Ulrich Stöcker, Bundesanstalt für Strassenwesen (BASt), Federal Republic of Germany

Vehicle and Driver Factors in Relation to Crash Involvement of Heavy Trucks
Dr Ian Jones, Forensic Technologies International, USA

Evaluation of Handling and Braking Characteristics of Heavy Vehicles
Prof Dr-Ing Klaus Rompe, Director, and Dipl.-Ing. Andreas Schindler, TÜV, Federal Republic of Germany

Heavy Vehicle Braking Characteristics - US and Europe
Mr Richard W Radlinski, Vehicle Research and Test Center (NHTSA), USA
(Mr Robert E Spicher)

Braking Safety of Heavy Vehicle Combinations in the Nordic Countries
Prof Lennart Strandberg, VTI, Sweden

Safety Aspects of Heavy Goods Vehicle Design
Dr P H Bly, Transport and Road Research Laboratory (TRRL), United Kingdom
THURSDAY SEPTEMBER 28

LONG TERM PAVEMENT PERFORMANCE

9.30 - 17.30

Chairman: Dir Ivar Schacke, Head of the Danish Road Research Laboratory, Denmark

Long Term Pavement Performance: Progress and Products
Mr Neil F Hawks, Strategic Highway Research Program, USA
(Dr W R Hudson, Texas Research & Development Foundation)

Performance Monitoring and Data Acquisition for Pavement Performance Evaluation
Ms Cheryl A Richter, Strategic Highway Research Program, USA

Trends in Pavement Performance Based on the National Road Maintenance Condition Survey
Mr Peter Scott, Department of Transportation, United Kingdom

Canadian Long Term Pavement Performance Study: Recent Developments
Mr Greg Williams, Roads and Transportation Association of Canada, Canada

13.00 Luncheon

14.00

Future Pavement Performance Research Data Needs for Texas
Mr James L Brown, Texas Department of Highways and Public Transportation, USA

Means of Creating Links between US LTPP and European Monitoring Programs
Panel discussion

HIGHWAY OPERATIONS

15.00 - 17.30

Chairman: Mr Bo Simonsson, Swedish Road and Traffic Research Institute (VTI), Sweden

An overview of SHRP’s Highway Operations Research
Mr Don M Harriott, Strategic Highway Research Program, USA

Evaluation of Effectiveness of Pavements’ Preventive Maintenance Treatments
Dr Roger E Smith, Texas A & M University, USA (Mr Don Harriott)

ERASME – French Expert System for Pavement Maintenance
Mr Pierre Joubert, SETRA, France

Use of Rubber Modified Asphalt for Snow and Ice Control
Mr Hossein B Takallou, CTAK Associates, USA

VTI RAPPORT 352A
HIGHWAY SAFETY

8.30 - 12.30

**Chairman:** Prof Kårre Rumar, Swedish Road and Traffic Research Institute (VTI), Sweden

**Risk Analyses in Highway Engineering**
Prof Dr-Ing Walter Durth, Techn University of Darmstadt, Federal Republic of Germany

**New Methods for Evaluating Safety Measures Using Accident Data**
Dr Olga J Pendleton, Texas Transportation Institute, USA

**Evaluation and Comparison of Traffic Safety on High Standard Rural Roads**
Dr-Ing Ulrich Brannolte, University of Karlsruhe, Federal Republic of Germany

**Road Design and Safety**
Dr Karl-Olov Hedman, VTI, Sweden

**A Strategic Transportation Research Study for Highway Safety**
Mr Jerry A Reagan, Turner Fairbank Highway Research Center (FHWA), USA
(Mr R Clarke Bennett)

CONCRETE AND STRUCTURES

8.30 - 12.30

**Chairman:** Dr Damian J Kulash, Strategic Highway Research Program, USA

**SHRP's Concrete and Structures Research: Goals and Recent Developments**
Dr Damian J Kulash, Strategic Highway Research Program, USA

**Concrete Microstructure Research and Its Applications in Highway Pavements**
Dr G M Idorn, G M Idorn Consult, Denmark

**Chloride Removal and Corrosion Protection of Reinforced Concrete**
Mr John Miller, N O T, Norway

**Chloride Binding in Cement and the CL/OH-Ratio of the Pore Solution**
Univ Doz Dr Josef Tritthart, Graz University of Technology, Austria

**Corrosion Protection of Reinforcement Against Chloride Impact**
Univ Doz Dipl-Ing Dr Gerhard Hartl, Austrian Concrete Research Institute, Austria

**Protecting Concrete by Flexible Waterproofing Slurries**
Dipl Ing Andreas Volkwein, Technical University of Munich, Federal Republic of Germany

VTI RAPPORT 352A
Participants in international conference in Gothenburg 27—29 SEPT 1989
"Strategic Highway Research Program and Traffic Safety on Two Continents"

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Bladlund
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Hortlund
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Hunter
Hurley
Höbeda

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Royal Inst of Technology
Skanska AB
Roads and Waterways Adm
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M I R A
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Sabema Material AB
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NCC Bygg AB
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Politecnico Di Milano
Nat Com of Traffic Safety
Norwegian Road Administration
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National Asphalt Pavement Ass
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Royal Inst of Technology
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Texas Res and Dev Foundation
Highway Safety Research Center
BP Chemicals
VTI

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### Participants in international conference in Gothenburg 27—29 SEPT 1989
"Strategic Highway Research Program and Traffic Safety on Two Continents"

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Participants in international conference in Gothenburg 27—29 SEPT 1989
"Strategic Highway Research Program and Traffic Safety on Two Continents"

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An Overview of SHRP's Highway Operations Research

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HIGHWAY OPERATIONS RESEARCH

by

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ABSTRACT

The Strategic Highway Research Program (SHRP) includes research in Highway Operations. The studies making up this subprogram are Maintenance Effectiveness, and Snow and Ice Control. Investigations in Maintenance Effectiveness will determine the efficacy of different preventive maintenance treatments for flexible and rigid pavements. Surface dressings, crack sealing, and thin overlays will be evaluated for flexible pavements. Undersealing and crack sealing will be evaluated for rigid pavements. Instruments and equipment for identifying and measuring pavement distress which can be addressed with preventive maintenance procedures will be developed. Innovative materials and equipment for pavement maintenance will be evaluated in the program. Safety measures for maintenance workers and highway users in highway work zones will be developed. Finally, training information and training materials for maintenance managers and workers will be produced. US$16.6 million have been budgeted for this work.

Snow and Ice Control investigations include fundamental studies to prevent ice bonding to pavements; or, if formed, to destroy the bond. Surface modifications and deicing by chemical means are being investigated. Standardized evaluation procedures for deicing chemicals will be developed for highway agencies to use. More efficient snowplow designs will be produced and prototype plows will be tested. Weather and storm monitoring equipment and communications systems to enable the highway agencies to manage their snow and ice control resources better will be developed. The work in this part of the program is budgeted for US$6.6 million.
1. INTRODUCTION - CHAPTER 1

SHRP's Highway Operations research program includes two subprograms: Maintenance Cost-Effectiveness, and Snow-and-Ice Control. In both of these areas, the research emphasis is on improved equipment, materials, and processes.

Highway Operations contracts will total $23.2 million: $16.6 million in Maintenance Cost-Effectiveness contracts, and $6.6 million in Snow-and-Ice Control contracts. This is a long-needed and substantial funding commitment for research in an area that consumes almost one-third of all highway expenditures.

In 1986, the latest year for which figures were available, nearly $18 billion were spent in the United States on highway maintenance by state and local governments -- accounting for about one-third of total annual highway expenditures. Due to inflation and additions to highway systems, maintenance expenditures have grown at a rate of over 8 percent per year during the past 15 years. If past rates of growth continue, maintenance costs will double in 8 to 10 years.

Two major factors drive highway maintenance costs upward. First is the increasing size and complexity of the nation's highway system. Modern highway maintenance includes a broad scope of activities ranging from the repair of sophisticated electronic traffic control equipment to routine painting, mowing, sweeping, and plowing. The number of lane-miles of paved surface, the number of rest areas, and the number and complexity of signs, signals, lighting, electronic detectors, guardrails, and energy attenuation devices are increasing rapidly; all need upkep. Moreover, the density of traffic is increasing, the frequency of use is greater, and there is a demanding need for fail-safe, continuous duty service for many of the traffic control systems.

Second, maintenance workloads and costs are increasing as the highway system ages and traffic levels increase. During the 1950s and 1960s resources were poured into new construction. During the 1970s and 1980s, the system has aged, while the size, weight, and volume of traffic served has increased. The compound effects of
age and service are increasing maintenance needs and maintenance costs.

Coupled with the increasing workload is the increase in danger to workers who are required to expose themselves to high speed, high volume traffic in order to keep the roadways in service. SHRP will develop better traffic control devices, procedures, and systems to protect maintenance workers and motorists alike from the hazards of traffic movements through work zones. And in order to communicate the usable results and findings of the maintenance effectiveness research, SHRP will develop communications and training products for appropriate audiences of the state highway agencies and other public works organizations.

Despite the growing economic significance of highway maintenance activities, the relative cost-effectiveness of various maintenance technologies has not been defined. SHRP will evaluate the performance of maintenance treatments, materials, methods and equipment and develop criteria for their cost-effective use under specific conditions. SHRP will develop technological improvements in maintenance equipment, materials and processes.

Snow-and-ice control is the single most costly maintenance function performed by many northern states and cities in the United States. Nearly every state experiences at least occasional economic disruptions due to winter weather, as road travel is slowed or stopped until pavement surfaces can be returned to functioning condition. Although government agencies spend over $1 billion each year on snow-and-ice control, and additional large sums are lost due to delays resulting from inefficient snow-and-ice removal, there has been relatively little research directed toward the improvement of equipment, materials and techniques for this important and costly activity.

SHRP's snow-and-ice control research includes fundamental investigations of the ice/pavement bond. The results of these studies will be used to identify and develop improved deicing chemicals, and will lead to the development of non-chemical means of snow-and-ice control. The payoffs from this research will be reduced costs for materials, labor, and equipment. Moreover, environmental impacts will be mitigated and pavement and bridge deck durability will be enhanced.
2. MAINTENANCE COST EFFECTIVENESS - CHAPTER 2

Many highway maintenance organizations are still basing a large part of their budgets on historical experience. This perpetuates inequities and can result in widely diverse levels of maintenance being practiced within the same jurisdiction. Funds available for maintenance have not kept pace with inflation so each year's budget purchases less labor, equipment and material. SHRP's Highway Operations Program includes several projects aimed at defining the effectiveness of high cost maintenance activities. The program also includes projects which will improve work zone safety --- both for the workers who must do the maintenance, and for the motorists who must travel through the temporary work zone. Training and communications materials for delivery of the results of SHRP's research in these areas will be produced.

The major objectives and products of SHRP's maintenance cost-effectiveness program are:

- Quantitative evaluation of the effectiveness of different preventive pavement maintenance treatments under a range of climate, load, and pavement conditions.

- Assessment of measuring systems and instrumentation for evaluating the effectiveness of maintenance treatments and development of improved technology for such measurements.

- Development of improved technology in materials and equipment for pavement surface repairs and selection criteria for their use in specific conditions.

- Improved devices, materials, procedures and/or techniques for the protection of workers and motorists in highway maintenance work zones.

- Development of communications and training products that will facilitate rapid and effective implementation of SHRP's maintenance effectiveness research and which are specifically targeted toward users and the various management and procurement functions that determine what resources are used for pavement maintenance.
2.1 Pavement Maintenance Effectiveness

Pavement maintenance accounts for a large share of the highway maintenance budget. Despite the high cost of pavement maintenance and the share of maintenance resources which must be dedicated to its demands, the choices of pavement maintenance treatments are not based on real knowledge of what is most effective for the pavement conditions at hand. SHRP will study several different preventive maintenance treatments over a five year period and objectively assess their cost-effectiveness.

SHRP will evaluate quantitatively the effectiveness of commonly used preventive maintenance treatments for both flexible and rigid pavements. To enable pavement maintenance managers in highway agencies to select appropriate maintenance strategies, SHRP will provide objective and uniform assessment of six different pavement maintenance treatments (four for flexible pavements, and two for rigid pavements).

Treatments which represent the most common, most available pavement maintenance strategies in use in the U.S. will be evaluated. Using in-service pavement sections, SHRP will investigate the effectiveness of the various preventive maintenance treatments. Among the factors considered when selecting pavement sections for treatment are traffic volumes, environment, and pavement condition.

The principal product of this project will be guidelines for determining which treatments will work best under given site, climate, and traffic conditions and the optimum timing for applying the treatments. The methodology used in this experiment will provide a uniform model for state highway agencies to use for evaluation of maintenance treatments on a nationwide basis. Where state highway agencies also include supplemental treatments unique to their state (materials, procedures, etc.), comparisons of these additional treatments will also be possible. This methodology will include measurement units, analysis procedures, pavement damage model modification procedures, and procedures for the use of damage models to select alternative pavement maintenance strategies.

2.2 Maintenance Measuring Systems and Instrumentation

SHRP will evaluate existing measuring systems and instrumentation to determine performance
characteristics, initial cost, operating cost, precision/accuracy, portability, ruggedness, durability, maintainability, and stability. Where feasible, modifications to existing equipment will be developed to improve its utility. New instruments, equipment, or procedures will be investigated for the better measurement of important characteristics. This will include design, fabrication, and testing of prototype units, with complete specifications for production, estimate of costs, and recommendations for uses for each unit. Training guides and manuals for the operation of the new and/or improved devices will be prepared.

2.3 Materials and Equipment for Pavement Surface Repairs and Crack Filling

The materials and equipment used for maintenance are often the same as those used in the construction process. Because there are important differences between new construction and maintenance operations, materials and equipment developed especially for pavement maintenance applications would be more cost-effective. Equipment that is efficient on a construction site can be cumbersome and inefficient in a maintenance work zone, where space is more confined. Mechanization can replace labor-intensive practices, reduce pavement maintenance activity unit costs, and enhance safety by minimizing the number of personnel exposed to traffic and the amount of time they must be exposed. Mechanization also may improve the quality of repairs, so that they will last longer. Pavement maintenance operations are more labor-intensive than construction, and materials costs account for a much smaller proportion of maintenance costs. For pavement repair applications, more expensive specialty materials that may not be cost-effective in new construction may be the most economical and durable choice.

SHRP is compiling a data base of materials used for pavement repairs and documenting the performance of them under specific conditions. Analysis of these data will identify the materials that are most effective and efficient in preventing pavement deterioration in the widest range of circumstances. Some of the characteristics of effective maintenance materials for particular application might include: quick setting and rapid curing; tolerance for colder temperatures; stability under traffic and severe environmental conditions; quick application; good bonding properties; adequate surface skid resistance; compatibility with application equipment; effectiveness in preventing
water intrusion; non-toxicity to humans or to the environment. SHRP will identify the tests that best reflect field performance of the materials and then will evaluate the most effective and efficient materials (including new products) through laboratory and field tests. SHRP will publish manuals for selection and use of those maintenance materials determined to be most effective. The manuals will explain the recommended applications for the materials, and cover critical factors associated with their use.

SHRP is surveying and evaluating existing equipment for surface repairs and crack sealing. Very little specialised equipment for these maintenance functions is currently available.

2.4 Maintenance Work Zone Safety

Highway maintenance work is an extremely hazardous occupation, especially at temporary work sites without permanent safety barriers. SHRP is developing prototype devices and techniques to protect workers in short-term maintenance work zones. Examples of the types of devices that might result include: remotely controlled equipment (robotics) to replace workers in hazardous tasks; improved barrier devices; ways of speeding up maintenance operations to minimize worker exposure; and more effective methods to warn workers of errant vehicles and the drivers of the work ahead and the proper path through the work zone.

Highway maintenance workers have extremely hazardous jobs. Highway construction and maintenance workers suffer greater work-related injury rates than many of those with other relatively hazardous occupations, such as building construction workers. State highway agencies and highway construction and maintenance contractors are extremely concerned about the number of worker fatalities, particularly in short-term and moving work zones. Work in temporary highway maintenance work zones is generally more hazardous than in semi-permanent highway construction work zones because drivers are less aware of temporary work sites. Also, because maintenance workers move fairly rapidly from one work area to another, lightweight, portable safety barriers are used rather than the more protective but less portable concrete median barriers that are typically used at new-construction sites.

SHRP will develop innovative equipment, procedures, and techniques for protecting maintenance workers in short-term work zones. To solicit innovative ideas,
SHRP sponsored a national design competition presenting specific design challenges. More than 120 designs for devices to improve work zone safety were submitted. Of those, 37 were selected for further investigation and possible development. The more promising concepts will be developed. Prototype units will be constructed and tested. The final product will be specifications and application criteria for effective, practical systems for protecting the maintenance workers and making the temporary highway work zone safer for workers and motorists alike.

3. SNOW AND ICE CONTROL - CHAPTER 3

Rapid, effective removal of snow and ice from roads keeps people and goods moving on the vital highway transportation network in the United States. This is currently accomplished in many areas with the use of chemicals such as sodium chloride. Approximately 10 million tons of road salt are used each year in the U.S., with frequent contamination of the roadside soil. Runoff then carries the de-icing chemicals into streams and water supplies.

SHRP's snow and ice research is initially focused on physical methods of disbonding ice from pavements, or preventing the ice-pavement bond from developing. Fundamental studies will provide the background for developing innovative techniques, materials, and equipment to reduce or eliminate ice adhesion without the use of environmentally undesirable physical or chemical energy sources. Based on this research, SHRP will investigate both physical and chemical means for modifying the pavement surface to prevent and/or retard ice-pavement bonding, as well as physical and chemical means of removing the ice-pavement bond once it has formed.

Present snow plow design has evolved empirically with scant attention paid to the physical properties of the material being handled and with little consideration of the aerodynamic and hydrodynamic principles involved in the flow of fluidized snow. As a consequence, energy expended in moving snow is disproportionate to the work performed. SHRP will develop snow plow designs based on aerodynamic/hydrodynamic principles, the material handling characteristics of snow, and ice-cutting mechanics. This will involve several standard designs for different types of snow and climatic conditions.
The major objectives of SHRP's snow and ice research program are:

- Alternatives to chloride de-icing chemicals -- either by the development of de-icing materials which are more compatible with the nation's environmental goals, or through non-chemical means of snow and ice control.

- More effective rock salt with lessened adverse environmental effects.

- More efficient snow plows to remove the snow from the highways and more effective blowing snow control measures to minimize the amount of snow to be removed.

- Improved storm monitoring and communications systems for more accurate snow warnings for highway maintenance crews and the travelling public.

- Development of communications and training products which are specifically targeted towards users and the various highway agency management and procurement functions that determine the resources for snow and ice control operations.

3.1 **Fundamental Studies: Ice-Pavement Bond**

Much of the previous research on ice-pavement bond prevention has been approached empirically. Thus, there is neither a foundation on which to build a more complete picture of ice-pavement bonding nor on which to base development of methods for preventing bond development. SHRP is conducting a fundamental study of ice-substrate bond structure and mechanics of formation in order to prevent bond formation without adversely affecting desirable pavement properties. Also, SHRP has underway a fundamental study of ice-substrate bond structure and mechanics of formation to provide a sound basis for developing techniques and devices for disbonding ice after it has formed on a pavement surface.

3.2 **Alternatives to Chloride De-Icing Chemicals**

The level of chloride in the water supplies of many U.S. cities increased between the late 1940s and the early 1970s, paralleling the expanding use of salt on
the nation's highways and streets. In the early years of salt use on highways, open storage of stock piles of salt led to soil and water contamination caused by leaching during and following rainstorms or snowstorms. Increased salt contents in adjacent streams and ponds, soils, and subsurface water is often an undesirable effect of using salt on pavement surfaces.

The corrosive effects of the brine also cause significant problems. Vehicles using the highways rust, and metal structures on or adjacent to the highway corrode. Of particular importance is the corrosion of reinforcing steel in reinforced concrete bridge decks, piers, and abutments. Corrosion from deicing chemicals has been a major cause of the prevalent concrete bridge deterioration problem that has evolved in all but a few states over the last two decades.

The environmental and structural damage caused by using chemicals to remove snow and ice has created a strong need for alternative approaches. Materials which have the melting capability of sodium chloride or calcium chloride, but not the adverse environmental or corrosive effects, would offer maintenance crews an attractive alternative for winter maintenance. Obviously, an alternative chemical must compete with the relatively low cost of salt, or establish a favorable cost-benefit relationship when environmental and corrosion savings are included in the evaluation.

A procedure is needed to evaluate possible salt substitutes in winter maintenance programs. Key factors to be considered include cost, corrosion potential, toxicity, solubility, flammability, and ease of application. SHRP will establish criteria for evaluating deicing chemicals and standard procedures state highway agencies can use for evaluating and testing them.

SHRP will explore the feasibility of both chemical and physical modifications of the pavement surface for snow and ice control. Examples of physical surface modifications include grooving and other surface texture modifications, and coating with icephobic materials to prevent ice-pavement bonding.

Still another alternative to deicing chemicals is the physical disbonding of snow and ice from the pavement. When fully developed, the ice-pavement bond is stronger than the tensile strength of ice. The practical result of this is the difficulty in removing ice completely to
bare pavement by purely mechanical means. SHRP is investigating the feasibility of using electromagnetic energy (e.g. microwave and laser radiation) to destroy the ice-pavement bond without contacting the pavement surface. SHRP is also assessing the feasibility of applying chemical solutions with high-pressure jets to penetrate the ice so as to apply the deicing chemical to the pavement right at the ice-pavement interface. This could reduce the amount of deicing chemicals needed, thereby making it economically feasible to use a more costly (and hopefully more benign) chemical.

Because of its low cost and ready availability, salt will, no doubt, continue to be used widely regardless of its environmental drawbacks. That being the case, every effort should be made to modify it to overcome the drawbacks. SHRP will investigate methods, such as additives, for reducing the corrosion and adverse environmental effects of salt while at the same time improving its ice-melting characteristics. Modifiers with promise will be tested under winter traffic conditions to measure corrosion aspects and to observe environmental factors.

3.3 Improved Snow Plows and Blowing Snow Control

Snowplows presently in use have not been designed with sufficient attention to the material-handling aspects of moving snow, or to aerodynamic and hydrodynamic principles. Furthermore, plow and truck must be considered as a system for best performance; past practice has generally treated plow design independently of the design and capabilities of the truck. These design practices result in overloading the truck chassis and frequent breakdowns due to bent and broken frames. Because the snowplow operator has little control over how far the snow is cast, multiple passes are often necessary, and snow has to be rehandled. This wastes energy.

Though the primary function of a blade plow is the lifting and casting of snow to clear the travelled way, mechanical removal of ice is an increasingly important additional function of blade plows and one that requires experimental design. Underbody blades are used for removing compacted snow and ice as well as for clearing light snow accumulations. Front-mounted blade plows have greatest utility for snow displacement, but improvement in their capabilities for ice cutting is possible.
Design improvements in both types of plows and blades could increase the efficiency of state highway maintenance forces when getting the job done better and quicker is critical.

SHRP will establish design criteria and prepare design details and specifications for displacement plow systems based on aerodynamic/hydrodynamic principles, the material handling aspects of snow, and ice cutting mechanics, leading to standard designs for different types of snow and climatic conditions. This will include construction of full-scale prototypes which will be field tested and demonstrated in routine service.

SHRP also will improve snow fence design and develop improved guidelines for permanent and temporary fence placement, with emphasis on terrain interactions. SHRP will prepare design drawings and specifications for selection and implementation of measures to reduce or prevent snow drifting on both new and existing roads.

3.4 Storm Monitoring and Communications Systems

Snow and ice control is the single most costly maintenance function performed by many northern states and cities in the United States. Efficient snow and ice control operations require the rapid and early mobilization of personnel and equipment, and effective communication of conditions both within the organization and to the public prior to the start of a storm. False storm warnings waste resources and undermine the credibility of the organization. Maintenance organizations need accurate weather forecast information, as well as real-time knowledge of pavement conditions. Rapid communication both throughout the region and within the jurisdiction is necessary for effective resource management, and decision criteria are needed for selecting road treatments that are most appropriate to specific conditions.

SHRP will develop designs for communications systems components that will link winter maintenance organizations with their own crews, with weather forecasters, with government and school officials, and with the public media, so all parties can have access to current and accurate information about road conditions. SHRP will investigate the feasibility of using pavement sensors and micrometeorological stations to provide real-time information on pavement condition
to maintenance organizations, and to permit more accurate forecasting of future pavement condition. SHRP will develop criteria for evaluating alternative responses to the full range of conditions that can be encountered in a snow and ice control situation, and develop a procedure for selecting the most appropriate mechanical, chemical, or other treatment.

4. TRAINING FOR MAINTENANCE WORKERS - CHAPTER 4

An important component of all of SHRP's Highway Operations projects will be training and implementation packages. SHRP is committed to the development of communications and training products which are specifically targeted toward users and the various management and procurement functions that determine what resources will be available for maintenance. To encourage use of its Highway Operations research results, SHRP has funded a separate project with the objective of producing communications and training products that will facilitate rapid and effective implementation by the state highway agencies and other users.

The projects involving development of new materials, equipment, and safety devices all include production of implementation materials (specifications, recommendations for use, etc.). SHRP's contractors will work in close coordination with communication experts to develop the implementation materials into training packages which will communicate the results of the SHRP research to state highway agencies and other highway maintenance organizations who can benefit from it.
Evaluation of Effectiveness of Pavement Preventive Maintenance

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and

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1. INTRODUCTION - CHAPTER 1

Most pavement and highway engineers believe that applying preventive maintenance at the appropriate time is the most cost-effective method of extending the service life of the highway network, and highway agencies are spending millions of dollars each year on pavement maintenance. However, many of these agencies are experiencing a decrease in funding approved for pavement maintenance, especially preventive maintenance. Generally, funds available for the highway programs are never adequate to fund all of the needs. Funding requests are scrutinized by chief executive officers, commissioners and legislators, and the ability to substantiate the cost-effectiveness of the funds expended on the programs has an impact on which are funded.

Pavement management systems (PMS) are being used by many agencies to assist in determining maintenance and rehabilitation needs for pavements. They help determine how to allocate funds among several different programs and projects to provide the best pavement network condition for the funds expended. Cost-effectiveness of the various alternative approaches are normally used in the PMS to recommend fund allocation.

There is little information available concerning the cost-effectiveness of preventive maintenance treatments applied to pavements. This inhibits justifying funds to those who must allocate limited funds, and it hinders fund allocation analysis through pavement management systems. In addition, there is little information identifying the best time in the service life of pavements to apply the treatments. Nor is there information concerning the influence of environment, different traffic loads, support conditions, and other factors on the effectiveness of preventive maintenance treatments. Strategic Highway Research Program (SHRP) Project H-101, "Pavement Maintenance Effectiveness," was developed to address those problems.

The studies being developed by the H-101 study are also the preventive maintenance treatment SPS experiments for the LTPP studies, and have been designated SPS-3 for the flexible pavement study and SPS-4 for the rigid pavement study. This will insure that the study of the selected pavement sections can
continue beyond the initial five year period. In addition, by placing the actual sections adjacent to the LTPP GPS section, data collection costs and participating agency costs will be reduced to the minimum.

1.1 Study Objectives

The goals of SHRP Project H-101 include:

1. To establish the effectiveness of typical maintenance treatments in prolonging the life of asphalt and concrete pavements, as illustrated in Figure 1.

2. To develop methods of evaluating the cost-effectiveness of maintenance treatments.

3. To develop information on the effective timing of the application of maintenance treatments, as illustrated in Figure 2.

The basic approach of the study is to evaluate the field performance of pavements with and without the preventive maintenance treatments and record the data. The base of data will allow researchers, maintenance engineers, and pavement managers to develop a bench-mark analysis of the effectiveness of preventive maintenance treatments in extending the service life of pavements across the North American Continent. This information can then be used to determine the cost-effectiveness of the preventive maintenance treatments.

The treatments which were selected to be evaluated in the study include:

for flexible pavements:

- crack sealing,
- chip seals,
- slurry seals, and
- thin overlays;

for rigid pavements:

- joint and crack sealing, and
- undersealing.

In addition, participating agencies are encouraged to integrate their own experiments into the study.

2.2 Study Products

A set of data will be collected which can be used to demonstrate the impact of the selected maintenance treatments in
Figure 1. Effect of Treatment on Life Extension

Figure 2. Effect of Treatment on Life Extension of Deteriorated Pavement
extending pavement service life. This data will also help define the condition level at which the application of the treatments will provide the greatest change in life extension. A methodology has been developed to model the impact of the treatments on condition changes and service life modifications. These will provide the necessary information for developing cost-effectiveness analysis. In addition, the study will provide a better understanding of the basic mechanisms by which the preventive maintenance treatments extend the service life of the pavements.

2. STUDY DESIGN - CHAPTER 2

An experimental, or sampling, design was developed to identify the impact of important, controllable (key) factors on the effectiveness of the selected maintenance treatments. Some factors are common to all similar pavement studies, including the appropriate LTPP GPS studies, including: environmental conditions, traffic volume, and subgrade type. Others are specific to the types of treatments, pavements and materials being studied. There are also factors which cannot be controlled in the experimental design but which we know will affect the treatment and are considered co-variables.

The experimental or sampling design arranges these key factors so that their influences on the effectiveness of the treatments, considered the main effects of the experiment design and the dependent variables in the analysis, can be determined. The performance of pavements with the preventive maintenance treatments will be compared to the performance of similar pavements without the application of the treatments, which are called the control sections. Measures of performance will include: distress types, profile (or roughness), surface friction, structural capacity through deflection testing, and material properties.

Since the goal of this study is to determine the effect of the individual treatments in extending the pavement life, the impact of the individual materials or construction processes is not a primary concern. In addition, the overall goal is not really to compare the performance of one treatment to another, but rather to compare the change in performance of the treated section to the untreated section. The impact of the preventive maintenance treatment desired is that of the process, e.g., a slurry seal. Therefore, common treatment materials, mix designs, and treatment construction specifications which are known to work reasonably well in each individual climatic zone have been selected. Although localized materials and techniques are important to the local agency, they cannot be allowed to control the national experiment. Any comparison of performance of treatments due to different material characteristics and
construction techniques should be reserved for agency specific studies, which are encouraged by SHRP and will be integrated into the data collection and analysis.

2.1 Factors

The common (or site) factors in the experimental design for preventive maintenance for asphalt concrete and portland cement concrete pavements include:

1. moisture: Wet
   Dry
2. temperature: Freeze
   No-Freeze
3. subgrade type: Fine grained
   Coarse grained
4. traffic loading: Low
   High

Concern has been expressed that, for instance, low and high traffic levels will not be adequate to define the spectrum of traffic observed. The arrangement into the cells is only a mechanism to insure that there are different levels in the study, not to define discrete points. In fact, it is hoped that the variables will vary considerably within each range. This will provide sufficient data to develop continuous relationships for all of the variables of interest.

The second set of factors are generally different for the SPS experiments from those of the GPS experiments and different for the asphalt and concrete experiments. Obviously, the individual treatments, listed previously, are included. There is no plan to explicitly evaluate the effectiveness of combinations of the treatments; each will be considered a separate treatment and considered alone. However, surface preparation representing good engineering practice will be included.

Asphalt Pavements - It is believed that the two pavement factors which have the most influence on the performance of preventive maintenance treatments applied to flexible pavements are the condition of the pavement at the time the treatment is placed and the structural capacity of the pavement compared to the traffic loads being applied to the pavement. There was considerable discussion concerning the number of pavement condition levels to define. The preventive maintenance treatments are to be applied to the pavement sections in the hope of preventing, or reducing the rate of, deterioration. It is believed that this is most effective if the pavement is in
good condition, and the treatment is applied to retain the pavement in that condition level. It is believed that there is a condition level at, or below, which the preventive maintenance treatments will have little effect. Depending on the traffic level, there is believed to be some intermediate level at which the treatments will reduce the rate of deterioration but may not be as effective if applied earlier.

Three levels are required to define these; however, the primary goal is to define the effect of the treatments on pavements which are believed to be in a condition which will respond to the treatment. There is concern for spending money to show that pavements in poor condition will not respond well to light maintenance treatments. However, if the treatments are not applied to the pavements in all three condition levels, then it is possible that we will not be able to answer all of the questions. It is important to apply the treatments at a poor condition level to anchor our analysis; however, it is possible to use less than a full factorial of pavements in this condition. The structural capacity is considered a two level factor, and was initially based on structural number. The selected factors include:

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Rigid Pavements - The two factors known to affect the performance of preventive maintenance treatments applied to the rigid pavement are pavement condition at the time of treatment and the type of subbase. Pavement condition will be divided into three levels in an approach similar to the flexible pavement studies. The subbases to be considered are granular and stabilized bases. Basically, subbase replaces the factor of structural adequacy in the flexible pavement study design and all other factors are similar. However, the definition of high and low traffic level is considerably different. The selected factors include:

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<td>6. subbase:</td>
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VTI RAPPORT 352A
2.2 Covariables

Covariables are measured independent variables which are not used in the basic design to select the treatment locations. Although, they are suspected of having an impact on the performance of the preventive maintenance treatment, they cannot be controlled in the experiment. Primary reasons for not controlling them include: monetary constraints (keeping the number of sections within the study to a reasonable level), lack of available candidate sections, lack or prior knowledge during the site selection process, and others. Covariables which were identified for the preventive maintenance applied to flexible pavement include: age, layer thicknesses, layer material properties, shoulders, subdrainage, prior maintenance, quality of the treatment construction or application, treatment material properties, and environmental conditions at the time of treatment application. In addition to those covariables identified for the flexible pavement studies, the covariables for preventive maintenance applied to rigid pavements include slab length and load transfer. Several other uncontrolled and unknown variables are sure to exist.

2.3 EXPERIMENTAL DESIGN FOR SHRP-101

Flexible Pavements - The proposed experimental design for flexible pavements is depicted in Figure 3. This design is an unbalanced partial factorial design with fractional replication and missing cells. The nature of the imbalance and missing cell structure is designed to provide the maximum power for detecting significant treatment differences in those cells where the treatments are expected to show the most effect. This design will require 120 locations; however, the data will be collected in a frequency proportional to those areas of greatest concern.

Rigid Pavements - The proposed experimental design is depicted in Figure 4 for both the wet and dry zones for plain concrete pavements and for the wet only for reinforced pavements. This design is again an unbalanced partial factorial design with fractional replication and missing cells, similar to the flexible pavement design with slight modification. The nature of the unbalancedness and missing cell structure is designed to provide the maximum power for detecting significant treatment differences in those cells where the treatments are expected to show the most effect for rigid pavement treatments at the earliest time possible. This design will still result in 120 plain and reinforced sections; however, the data will be collected in a frequency proportional to those areas of greatest concern.
For both Wet and Dry

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Figure 3. Proposed Design with Replication for Treatments to Flexible Pavements

For both Wet and Dry plain and Wet reinforced sections

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Figure 4. Proposed Design with Replication for Treatments to Rigid Pavements
3.0 CONSTRUCTION PLAN - CHAPTER 3

3.1 Options for Controlling Construction Consistency

Some of the covariables which are known to affect the performance of the treatments include: material properties, mix design procedures, construction equipment, construction crew experience and training, and application procedures. Many of these factors change dramatically from location to location; however, since the use of a specific material or construction technique is often limited to a small geographic or political area, there is no need to determine their impact over a large area. Each additional factor varied in the analysis would at least double the number of sections which would have to be constructed for each treatment. Therefore, a common set of treatment materials, mix designs, and treatment construction specifications which are known to work reasonably well in each individual climatic zone have been selected.

If these factors can be controlled, the impact of the key variables and their interactions on the effect of preventive maintenance treatments can be determined. This will provide the desired North American "base line" analysis of the impact of the treatments on the performance of pavements across North America desired. Inferences which can be made from it will be limited to the range of the controlled factors; however, since most of these controlled variables are limited in a localized area, only a small number of test sections would have to be evaluated in the local area to develop a "shift factor" for use with the overall base line analysis. The results of the overall study then could be applied to local analysis as well.

There has been considerable concern over how to control these variables from the very beginning of the project. There is no money in the SHRP budget to pay for construction of the treatments. The participating agencies must fund the construction of the treatment test sections. The concept advocated in the original proposal was to develop a strict set of specifications and allow the participating agencies to control application. However, the H-101 staff was advised by the SHRP Highway Operations Advisory Committee to develop an approach which would limit the construction to one crew, one equipment set, and one set of material sources for each treatment. This concept was agreed to at the first meeting of the H-101 Expert Task Group (ETG). However, the concept was modified to allow the use of single crews, equipment sets, and material sources in each of the four climatic regions for each of the treatments except the thin overlay. It was apparent that the thin overlay could not be controlled in the same manner because of the equipment required to prepare the asphalt concrete.
At the time this paper is being prepared, the potential participating agencies are reviewing the proposed options for controlling construction, and no final decision has been reached. In addition to the regional contractor option, an alternative provides for the participating agencies to control the application but requires centralized material purchase, a strict set of specifications, specialized quality control assistance, and centralized mix design, all accompanied by special training. The second alternative allows the participating agencies to construct the treatments using a set of common specifications agreed upon by the participating agencies within the region. It is hoped that regional contractors will be used, and at least, a single source of materials for the crack sealant, slurry seal, chip seal, undersealing material, and joint sealant materials will be used for each region.

Regional meetings are being organized to finalize the construction plans within each region. It is expected that the majority of these will be completed by the time of the conference, and the results will be presented at that time. The general descriptions of the options are provided below.

Regional Contractors - The participating agencies will form a regional task group (RTG) in each region which will work together with SHRP representatives to select a regional contractor to apply each of the treatments, except for the thin overlay. Participating agencies will be responsible for constructing the thin overlays. Each contractor will purchase all of the materials required for the sites within his region from one set of sources at one time and stockpile the material if necessary. Using a single crew and a single set of equipment, the contractor will apply the treatments at each site within the region. The contractor will be responsible for quality control and traffic control during the construction process. The participating agency would be responsible for quality assurance and project acceptance. The RCOC would conduct extensive checks on the construction of the treatments, including the thin overlays, if possible. All material sampling, material testing, and mix designs would be conducted by the SHRP sampling and testing contractors, except for the thin overlay materials. Materials sampling, materials acceptance testing, and mix designs will be conducted by the participating agencies for the thin overlays.

Participating Agency Responsible for Construction with Restrictive Specifications and Centralized Material Purchase - Each participating agency in each region would select the contractor or force account crew to apply the treatments for their agency. Using a single crew and a single set of equipment, the agency would apply the treatments at each site within its area of responsibility. The agency will be
responsible for the traffic control during the construction process if they used force account crews; if a contractor was used, it is assumed that the contractor would be responsible for traffic control. Each construction crew would be responsible for quality control. Quality assurance will be the responsibility of the participating agency. The RCOC will conduct extensive monitoring checklists during the construction. Material sampling and material testing would be conducted by the SHRP sampling and testing contractors. Mix designs would be conducted by the participating agencies. Materials sampling, materials acceptance testing, and mix designs will be conducted by the participating agencies for the thin overlays.

**Participating Agency Responsible for All Phases** - Each participating agency in each region will select the contractor(s) or force account crew(s) to apply the treatments for their agency. The agency will be responsible for testing, accepting, and purchasing all materials and submitting the results of tests to the SHRP RCOC. The agency will apply the treatments at each site within its area of responsibility according to the specifications. The agency will be responsible for the traffic control during the construction process; however, it is assumed that this would be a part of the construction contract if the treatment is applied by contract. Quality assurance will be the responsibility of the participating agency. The participating agency will be responsible for all material sampling, testing, and mix designs and reporting the results to the SHRP RCOC.

### 3.2 Test Sections

To conserve limited testing funds and reduce the effort of participating agencies related to the number of site locations, it is planned to place H-101 test sections in a construction site containing an LTPP GPS test section, if at all possible. Figure 5 shows how this might appear for treatments applied to flexible pavements and Figure 6 illustrates how it might appear for rigid pavements. The actual sequence of the test sections will be randomly selected and will be affected by terrain, structures, and other constraints. They need not be contiguous either with the GPS section or other H-101 test sections. The actual test section will be only the outside lane; however, the treatment will be applied to both lanes.

The test sections on flexible pavements will be 500 feet long and will extend for at least 50 feet beyond the end of the actual test section; the chip seal will extend at least 100 feet beyond the end of the actual test section. The test sections on rigid pavements will be 500 feet, or ten slabs, long (whichever is longer). A transition length will extend beyond the actual test section a distance of 50 feet of two slabs (whichever is
Figure 5. Flexible Study Test Sections

Figure 6. Rigid Study Test Sections
longer). A separate control section is required to which no maintenance will be applied until it reaches a deteriorated condition, except emergency maintenance required to prevent liability problems.

The treatments will be placed on the lane adjacent to the LTPP GPS test section first. Then, the traffic control will be switched to the lane containing the H-101 treatment test sections, and the treatments will be placed on the actual test section.

3.3 Measuring Quality of Construction

The participating agencies will be responsible for normal quality assurance. These programs generally insure that the treatment application process meets the required specifications. If the specifications are properly prepared, and followed, they should prevent the treatment from being an immediate failure. However, within the specifications, there is considerable allowance for variation. This variation probably affects the long-term performance of the treatment and its affect on the life of the pavement.

The H-101 project staff is preparing a set of check lists which will be completed on each treatment application. Each item in the specification, as well as other information deemed important will be observed and recorded. Where direct measurements are taken, such as temperature, they will be recorded on the scale used. Where subjective observations are made, a rating scale is being developed to be checked by the observer.

Each item on the check list will be weighted as to importance. These weightings will be initially developed from the recommendations of the H-101 Expert Task Group. It is possible that they can later be adjusted based on regression analysis with recorded information and performance data. They will be multiplied times the rating scales for each observation to determine an overall rating of the quality of construction.

To observe the changes in quality of construction in the non-test section lane versus the test section lane and between test sections, control tables concepts will be used check variation in measured quantities.

4.0 EXPECTED RESULTS - CHAPTER 4

Highway agencies will be able to use the performance data from the SHRP H-101 study to show the impact of the treatments studied on extending life of pavements. Those that participate with their companion studies which parallel the SHRP studies but
address problems specific to the agency will be able to evaluate the impact of localized conditions as well. When these are combined with their cost data, the cost-effectiveness of the treatments will be available.

The cost-effectiveness data will allow pavement management systems and managers to more accurately project the needed preventive maintenance and their effects. This will lead to more requests for more efficient allocation of scarce maintenance dollars. In addition, it will improve the ability of those developing and presenting budget request to justify the maintenance budget requests. This will improve funding of pavement maintenance and result more effective expenditure of funds allocated to pavement maintenance and rehabilitation.

The improved knowledge gathered concerning the mechanisms by which the treatments extend the life of the treated pavements will allow new and improved treatments to be developed which can better address those mechanisms of which can address other mechanisms. The quality if construction rating system and quality of construction control tables will help agencies better evaluate the quality of treatment application and determine its impact on performance of other treatments.

5.0 ACKNOWLEDGEMENTS - CHAPTER 5

Mr. T. J. Freeman of Texas Transportation Institute, Dr. T. D. White and Mr. T. Chapin of Purdue University, Dr. D. Y. Lee of Iowa State University, Dr. S. H. Carpenter of ERES, Consultants, Inc., and Mr. S. C. Shah the SHRP staff have been instrumental in this study. The H-101 Expert Task Group has been most helpful. The efforts these and others are gratefully acknowledged.
ERASME - An Expert System for Pavement Maintenance

Pierre Joubert
SETRA
France

and

F Allez
CETE Méd
France
ERASME

An Expert System for Pavement Maintenance

(French acronym for highway maintenance assisted by a multi-expert system)

To make an expert system available to the operational managers of the road networks seems to be a well adapted response to the problem: an important economical activity (5 Billions of French Francs per annum, in France), a widely distributed decision making which is not always strictly rational, well known but rarely available experts. Such is the aim of ERASME (Road Maintenance assisted by a Multi Expert System).

ERASME assists the operator for the diagnosis and the design of the pavement works for a homogeneous section (a section is homogeneous if all the significant parameters of the problem are sufficiently identical for the whole of this section). ERASME reproduces the thinking of a specialised engineer trying to understand the problem. The questions it asks are not numerous and all to the point. ERASME is able to explain its questions and to justify them for the user.

After a diagnosis stage, ERASME establishes work solutions. For an identical diagnosis it proposes several alternatives that will meet with the requirements as established by the user. If the case arises that missing data or a too complicated problem lead to multiple solutions, ERASME is able to show the way to the user towards the most relevant enquiries (additional visual checks, lab tests) in order detect the most appropriate medicine, or even, if the case is too complicated, to send the user back to a human expert. ERASME is the result of the work of a multi disciplinary team (computer engineers, experts, users) working under the supervision of Works Leading Committee that gathers executives from the Directorate of Roads, which finances the project, and of future users (District Public Work Directorates, District Technical Services and Motorway Companies).

The expertise shown in ERASME is representative of the French abilities and know how in the field of pavement maintenance. Especially, the problem of road maintenance has been broken down into a score of more specialised sub-problems, among which for example: wearing course fatigue, frost, traffic structure adequacy. For each one of these sub-problems, a search was launched, within the lab network, for the specialist likely to supply the highest expertise and his knowledge has been modelized into an expert system. ERASME gathers all these expert systems which work under the
authority of a supervisor.

ERASME is linked with calculation software used in the field of road maintenance and takes profit from all the advanced technologies in the field of Artificial Intelligence: non monotonous reasoning, multiple states, reasoning on the reasoning, which are integrated into the SMECI generator. ERASME will be available on the 32 bits SUN work stations rigged up with UNIX, Le_Lisp (INRIA) and X-Windows.

It is being tested (for flexible pavements only) since June 1989, by some ten pilot users who have the task to detect the last discrepancies of a system that was already widely tested during the second semester of 1988 by six experts. This ultimate tuning is necessary in order to supply the "general public" users with an easy to use product, even for a person not familiar with data processing; the goal aimed at represents about a hundred work locations fitted with the program during the coming two or three years.
ERASME
An expert system for pavement maintenance

P. JOUBERT (SETRA) F. ALLEZ (CETE Méd.) - August 89

1. Introduction

Road maintenance represents an important economical activity the yearly turnover of which may be evaluated as more than 5 billions FF for the whole French network. The management systems that are progressively implemented, in France like in many other countries, supply this activity with a precious help.

But management systems, as sophisticated as they may be, are mere quite imperfect tools; especially, the complexity of the problems, mainly when a close attention is given to a pavement stretch, strongly limits the appeal to algorithmic processes.

This is why, engineers, all over the country, determine every year, maintenance jobs to be done according to the informations available to them, informations that are both qualitative and quantitative, but most of the time incomplete, redundant or contradictory. To deal with difficult problems, decision makers call on experts, whose number does not exceed a few scores and who most of the time belong to the Civil Service technical departments (essentially laboratories).

To supply the operational managers of road networks with an expert system appears thus as a quite suitable answer to the problem: an important economical activity, a widely spread decision making, some well identified but rarely available expects.

These are the reasons why the Directorate of Roads of the French Ministry for Planning, Housing, Transportation and the Sea, has decided to produce ERASME, an expert system dealing with road maintenance (Entretien Routier Assisté par Système Multi Expert - Road maintenance assisted by a multi-expert system).
2 - THE EXPERT SYSTEM ERASME

2.1 The facilities

2.1.1 A homogeneous section

ERASME assists the manager in the diagnosis of a pavement and in the design of the jobs for a homogeneous section. A section is homogeneous if all the significant parameters of the problem are sufficiently alike along the whole of this section, i.e.:
- the deflection measurement,
- the pavement structure,
- the nature and the time of the maintenance jobs,
- the traffic,
- etc. ...

The user is the judge of the homogeneity of the section he wants to work on.

Fig. 1 The User interface of ERASME
The homogeneous sections have varying lengths ranging from a few hundred meters to some kilometres. The expert system we are dealing with does not take network level problems into account. As a matter of fact, ERASME is a complementary tool to the approach developed within the frame of technico-economical management systems.

2.1.2 Diagnosis

ERASME helps the engineer to specify and then to identify the problems the pavement has. During this stage, the expert system reproduces the thinking of a specialised engineer trying to understand the problem. The questions are not numerous and are oriented towards the essential.

The engineer in charge of the maintenance of a pavement is sometimes supplied with very few informations to make an accurate diagnosis. In this case, the system is able to list several diagnoses and to point out the tests and (or) the measurements that could reduce the number of the competing diagnoses.

2.1.3. Design

After the diagnosis stage, ERASME proposes work solutions to correct the recorded discrepancies. Road maintenance is a field in which the number of technically feasible solutions may be high. To reduce the number of solutions, ERASME asks the user to specify a certain number of constraints that the solutions should comply with. In some ways, it consists in a description of the schedule of conditions of the possible solutions, in terms of life duration, level of service (improvement of the roughness) and budget.

Fig. 2 Design with constraints
2.1.4. Prediction

Unfortunately, in France, like in many other countries, to establish that a road section needs maintenance work is not a sufficient reason to obtain the necessary funds. The art of pavement management is to bring as accurate a reply as possible to the question: "If I do not perform this job, what is the risk I am taking?" This function will make ERASME a tool without equivalent within the French technical network, supplying the management with accurate and evaluated elements.

2.2 Characteristics

ERASME is implemented in SMECI, a high level expert system generator, offering objects, methods, production rules and multiple worlds. ERASME is available on UNIX work stations of the SUN 3/60 type and is being carried over to other types of equipment. The portability of ERASME will be that of Le_Lisp (ILOG-INRIA) and X-WINDOWS.

2.2.1 A collection of expert systems

The wide number of functions to fulfil and the complexity of the field of road maintenance obliged to break up the problem in several sub problems. ERASME is no expert system but a collection of expert systems gathered into a structure that allows them to collaborate in an intelligent manner. The main advantage of this architecture is the module construction that allows progressive set up and above all an easy maintenance.

Four main expert systems build up or will build up the body of ERASME: diagnosis, design, prediction and definition of the missing tests. Each one of them may be broken down more finely.

2.2.2 Specialists and supervisors

Within ERASME, an expert system is designed as an assembly of expert sub-systems co-operating under the authority of a supervisor. The specialists share the results in a common data base. They communicate between them thanks to the sending of requests. Module structure is the key idea of this architecture.

For example, the expert system for diagnosis gathers 17 specialists.
2.2.3 The thinking

The non monotonous thinking: the solution process in the diagnosis phase is non monotonous.

This means that certain facts are labelled as hypothesis and that they may be put in question again at a later stage of the thinking. This "suspicion" mechanism, which is quite similar to that of the human mind, leads to the construction of different worlds and provides for a better explanation of the past behaviour of the pavement.

The thinking on the thinking: each expert system is supplied with its own base of knowledge. When an expert system
is thinking, it may consult the thinking tree of its colleagues. This set-up is called "thinking on the thinking".

2.2.4 Computation

The experts have faith in their judgement, but they do not disregard using algorithmic models allowing one to specify more accurately this or that trend, or to check the adequacy of a given simplification hypothesis.

ERASME is no exception to the rule, and uses when needed, the computation programs that have proved their efficiency in the field of maintenance:

- ALIZE, a program that computes constraints and deformations within a multi-layer elastic structure,
- ORNIER, a program that computes the depth of ruts produced by the flowing of bituminous layers,
- GEL, a computation program for the development of frost within the pavement layers and the base soil,
- FISSTHERM, an ageing crack build-up computation program for bitumen.

2.2.5 Classifying and modelling

The knowledge enclosed into ERASME is displayed in various manners. Schematically, one can distinguish:

- the knowledges that refer to a classification process. They belong mainly to a field where the theory is not yet well established and for which expertise refers more to the "feeling" than to reason. It is also said that it is a "shallow" knowledge. The pathology of surface dressings is typical of this category.
- the knowledges that call for modelisation. They are mainly related to a field where algorithmic and deductive methods allow one to build models that are compared with the object to be evaluated, in order to ascertain the adequacy of the hypotheses retained for the construction of the model. It is also referred to "deep" knowledge. The traffic structure adequacy is typical of this series.

2.2.6 A reception structure

All the characteristics briefly listed above lead one to consider ERASME not as a new program but rather as a reception structure meant to integrate the bulk of the knowledge in the field of road maintenance. The handling of ERASME brings the user all the riches and the power of tools and knowledge reserved to the expert because they require a training and a frequent practice.
3 THE DEVELOPMENT OF ERASME

3.1 The development team

The development of ERASME has been awarded to a team of the Méditerranée Centre d'Etudes Techniques de l'Equipement - CETE - (Regional Planning Engineering Center), located near Aix en Provence (South Eastern France). This team is made of four computer technicians specialised in Artificial Intelligence and of two experts labelled as "generalists". The task of the team is the true realisation of ERASME, that is the gathering and the representation of the knowledge, the design and the realisation of the user interface, the writing of the manuals.

3.2 The experts

Three groups of experts are involved into the project.

The first group includes the two generalists mentioned above. They deal with all subjects and ensure the consistency between the knowledge scientists and the computer engineers on the one hand, and the second group of the "specialists experts on the other hand. As a matter of fact, the problem of road maintenance has been broken down into some twenty more specialised sub-problems, such as, for instance: the fatigue of the wearing course, frost, the traffic-structure adequacy. For each one of these sub-problems, we have looked throughout the network of our laboratories, for THE specialist likely to supply us with the best possible expertise while keeping the indispensable coherence of the whole.

The third group of experts intervenes to criticise, to question, to widen the expertise as it has been formulated by the two previous groups. The experts in that group have a basic role to play, although they were not associated to the gathering of knowledge phase, for it is their advice that will complete the knowledge stored within ERASME.

3.3 The partners

Thriving for efficiency, the team has chosen to call on the competence of some partners especially learned in very difficult topics.

SHELL brought its knowledge of flowing in the expertise part. The computer engineers made sure to get the help of the Institut National de la Recherche en Informatique et en Automatisme - INRIA (National Research Institute in data processing and automation), developer of the SMECI generator, as well as the assistance of the Société Intelligence
Logicielle -ILOG - (The Programme Intelligence Company) to develop performing interface equipment.

3.4 The pilot users

It is not possible to produce a program meant for a large distribution without seeking the opinion of the potential users. Therefore, ten voluntary pilot users did accept to test, during six months, the product and to supply the team with their remarks in order to make ERASME able to cope with the needs and the expectations of the customer.

4 THE PROGRESS OF THE PROJECT

4.1 The ownership

ERASME is being developed for the account of the Directorate of Roads of the Ministry for Planning, Housing, Transports and the Sea, which provides for the necessary funding. The very special characteristics of the expert systems prevents the project to be specified before the study and the specification is only developed progressively. One has therefore to provide for a structure that is both firm and flexible, that will ensure that the major goals of the Owner will be reached within the set time limits but that will also leave sufficient clearance to the developers to allow them to make profit of all the possibilities arising as the projects progresses. This project management structure gathers, under the name of "Ownership Committee", fifteen persons representing the Owners (Service d'Etudes des Routes et Autoroutes - SETRA - Roads and Highways Engineering Department, Laboratoire Central des Ponts et Chaussées - L.C.P.C. - Central Road Research Laboratory) and also future users (District Public Works Directorates, District Technical Services, Motorway Companies). The Committee is regularly informed of the progress of the project, and two Committee members are in standing contact with the development team.

The Ownership Committee also calls upon a consulting firm specialised in the Artificial Intelligence projects, COGNITECH.

4.2 The stakes

The cost of ERASME has been estimated at a 8 Billion French Francs level, two years ago; this budget will probably be slightly exceeded. Therefore, ERASME is apparently an expensive product. Of course, the Owners have been concerned with the profitability of such a financial effort and a primary approach has provided for an estimate of 2% of the total maintenance expenditures, i.e. : 100 billion francs the im-
pact of the use of ERASME.

One should furthermore emphasise the fact that beyond this coarse calculation, non quantifiable gains are already acquired:

- the review of all the knowledge in the field of maintenance,
- the outlining of the points on which efforts on research should be intensified,
- the production of an unmatched training tool: ERASME, like any expert system, is able to justify its questions, and to explain its thinking. It effectively constitutes a choice base for the training of the engineers in charge of maintenance.

4.3 The calendar

The first ideas concerning an expert system devoted to road maintenance, showed up on mid-85. During 86 a model was built, a small expert system dealing with one part of the problem, which allowed one to emphasise the tremendous contribution that Artificial Intelligence could bring into the treatment methods for pavement maintenance. For these reasons, it was decided, in February 1987 to launch ERASME. The first circulated version will show up in 1990. It concerns only flexible pavements: granular unbound body topped by at least 15 centimetres of bitumen treated materials. Versions for the bituminous pavements and for hydraulic binder treated pavements will be produced later.

4.4 Validation

In Autumn 1988 we entered the most delicate stage of the production cycle: validation. ERASME is an expert system designed for road engineers. To achieve a fully successful operation, the expertise included into ERASME must be correct, but also, the system has to be effectively used by those for which it is made.

ERASME is submitted to a two stage validation process:

First stage: validation of the knowledge. This is the role of the third group of experts as mentioned in section 3.2. Those six persons, four of which have had no participation into the previous stages of ERASME, have the task to test the product through the submission of real cases and the appreciation of the quality of the answer by ERASME. Each expert treats about twenty cases.
It is granted that for certain cases, too complicated, ERASME declares itself as not qualified and sends the user back to a human expert. The validation of the knowledge is concluded by a seminar gathering the validating experts and the development team, during which modifications to be brought are put in form.

Second stage: use on pilot sites. Around ten people, representative of the future users, test ERASME, especially for what concerns presentation, user's friendliness, appeal. As during the previous stage, the users deal with real cases and a seminar marks the end of the stage.

The first stage started in October; it appeared that the validation phase is longer and more complex than what was assumed, and although the evaluation seminar did take place during February 1989, the validation experts are still associated to the development of the product. We consider that at the present time, ERASME issues satisfying answers in more than 90% of the cases.

The second stage started in June 1989, with a product revised to take the remarks of the experts into account. The first impression of the users is extremely positive and shows that with well designed expert systems the training period is widely shortened.

5. THE INTEGRATION OF ERASME

All conditions are met that will make a high level tool available to the Directorate of Roads, a tool that allows one to solve most of the problems that occur during the planning of maintenance jobs.

We already know for sure that experts in the labs will be the privilege users of ERASME. But this target, which in any case, had to be satisfied, is not sufficient to reach the initial goals. ERASME will truly be a success from the moment it will be circulated in places where it is most needed, that is very near from the terrain.

One of the limits to the use of ERASME lays in the equipment. The selection of the Work Station comes from a compromise between the competence of the system and the target considered. The present cost of the equipment (about 150 kF for a fully equipped site) and its development towards cheaper price allow one to consider on a realistic fashion the fitting of approximately a hundred locations within the two or three coming years.

The second problem concerns the effective interest that the
field personnel has in ERASME. What will be the welcome of a very new product, likely to put a certain number of habits in question? The presence of users representatives within the Ownership Committee, the validation period on pilot sites all represent efforts to take this aspect into account at all stages of the development process. The first contacts with field personal who did not know anything about ERASME were very encouraging and seem to show that the method adopted was correct. Furthermore, a study is in progress, to evaluate what could be the impact of the circulation of such a product on the structures and the habits. It may contribute to modify the goals of the project itself or to give the marketing a better preparation to ensure a better penetration.

CONCLUSION

6.1 The development of an important expert system is a complex task that involves several qualifications: the result is the fruit of a team job.

6.2 It is interesting to note that the French operation is very different from that of our North American colleagues. Over there, the upsurge of expert systems is mainly the result of the attitude of road experts who see in AI an efficient means to continue and to circulate their knowledge. Here, ERASME is born from the certitude of AI specialists who were successful in convincing the executive officers of the road community. It is more than likely that the two procedures are complementary.

6.3 The modelisation of knowledge is an important task that sets up the conditions of the expertise quality. But a great care should also be granted to presentation as well as to the strictness of the validation which are the warrants of the possibilities to use the system.

6.4 The production of an ambitious expert system is very different from that of a classical programme. The impossibility to define practically, accurate specifications at the beginning of the study has to be compensated for by a careful follow up and above all a large confidence. Great professional qualities are required from the development team, but nothing can be built if there are no at least as great human qualities.

6.5 The power of the tools born from Artificial Intelligence allow one to bridge the gap between various fields of the same discipline and thus to achieve system concepts unifying the efforts of numerous experts.
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ERASME, the fruit of a team work

List of the persons associated to the development of ERASME

Meaning of the acronyms

CETE : Centre d'Etudes Techniques de l'Equipement
DDE : Direction Départementale de l'Equipement
DI : Division Informatique
DTC : Division Terrassements Chaussées
LCPC : Laboratoire Central des Ponts et Chaussées
LR : Laboratoire Régional
SCILE : Service Central Informatique et Logistique de l'Equipement
SETRA : Service d'Etudes Techniques des Routes et Autoroutes
STD : Services Techniques Départementaux

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DENEL et BRIAVAL STD Pas de Calais
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Use of Rubber Modified Asphalt for Snow and Ice Control

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USE OF RUBBER MODIFIED ASPHALT FOR SNOW AND ICE CONTROL

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Use of Rubber Modified Asphalt Concrete Pavements for Snow and Ice Control

by

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ABSTRACT

Rubber-modified asphalt pavements have been used in Sweden and the United States since the late 1960's. Rubber-modified asphalt concrete consists of ground recycled tire rubber particles 6.4mm (1/4 in. minus) added to a gap gradedaggregate and then mixed with hot asphalt cement. This paving system distributed under the trade names "Skega Asphalt", "Rubit" or "Rubtop" in Scandinavia and "PlusRide" in the United States, was found to provide a new form of wintertime ice control because of the increased flexibility and the action of protruding rubber particles. Based on a review of several case histories presented herein, the resultant paving mixture appears to have the following important characteristics for cold regions applications: (1) improved resistance to load induced tensile cracking and low temperature thermal contraction cracking; (2) improved skid resistance under ice conditions; (3) good resistance to studded tires and/or chains; and (4) reduced tire noise levels. Observations of the skid reduction benefits under icy road conditions have been made with a British Pendulum Tester and a vehicle equipped with a Tapley Brake Meter in the United States. Tests by the Alaska Department of Transportation and Public Facilities indicate that significant reductions in city road stopping distances nearly always resulted from the use of the rubber modified asphalt paving system. For 21 testing dates over three winters, stopping distances were reduced by an average of 25 percent. The effectiveness of rubber modified asphalt concrete ("Rubit") has been investigated by Swedish road authorities in several field investigations. The field observations indicated the accidents related to surface frost have been completely eliminated in the areas with rubber modified asphalt pavement.

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1.0 INTRODUCTION

1.1 Background

Snow and ice control is the single most costly maintenance function for many states and cities in the United States and Sweden. Roadway agencies in the United States spend over one billion dollars ($1,000,000,000.00) each year on snow and ice control, and additional large sums are lost due to delays resulting from inefficient snow and ice removal (1).

Roadway surface ice deposits became a major problem in urbanized areas with high traffic volumes and stop and go traffic movements. Costs of maintaining ice-free pavements through de-icing chemicals or improving traction through sand applications are very high and create other problems for roadway agencies and users. For example, the use of sands for ice control provides only temporary skid resistance and sand must be continually reapplied to maintain skid resistance. Stopping distances on sanded ice are also much greater than on dry pavement. In addition, sand must be removed from gutters and inlets in urban areas following a spring thaw to avoid blockage of drainage systems. Some recent analysis of the costs and benefits of using salt to remove roadway ice have indicated that the ultimate costs to the road user may be more than 10 times as high as the sum of the benefits (5). The major cost item, premature vehicle destruction through corrosion, greatly outweighed the benefits of reduced maintenance and accident costs. Salts also present the possibility of contamination of ground and surface waters from roadway runoff. Therefore, development of a method to modify asphalt concrete pavements to prevent the formation or bonding of ice has been of interest to researchers for many years (5).

In the late 1960's, experimentation was done in Sweden on the effects of mixing rubber particles into asphalt concrete pavements. A system incorporating 3 to 4\% by weight of total mix, relatively large 1.6mm to 6.4mm (1/16" to 1/4") rubber particles into an asphalt pavement was developed to increase skid resistance and durability. This system, distributed under the
trade names "Skega Asphalt", "Rubit" and "Rubtop" in Scandinavia and "PlusRide" in the United States was also found to provide a new form of wintertime ice control. The control mechanism apparently results from the lack of adhesion between the rubberized road surface and the ice layer, the flexing of the protruding rubber particles and the greater flexibility of the mix under traffic action, which cause a breakdown of surface ice deposits. The benefits of adding granulated recycled tire rubber to the asphalt cement mixture, besides ice control, are increased fatigue life, increased flexibility, resistance to studded tires, and improved resistance to crack reflection.

Also, in recent years, the growing problems of tire disposal have caused many public road agencies and municipalities to use waste tires as an additive in an asphalt concrete mixture. Each year the United States disposes of about 200,000,000 passenger tires and 40,000,000 truck tires. This represents about 2.1 million tons of scrap passenger tires and roughly 1.9 million tons of scrap truck tires, a 4 million ton total. While a limited number of these 4 million tons of tires are used for resource and energy recovery, the vast majority go to the landfills or are disposed of in an environmentally unacceptable manner. This presents a series of problems including the loss of a scarce resource and a potential health problem, since the tires serve as shelters and habitats for vermin and insects which often carry disease (8).

1.2 Purpose

The purpose of this paper is to summarize the results of field performance surveys and laboratory testing programs aimed at identifying the critical factors in designing and constructing rubber-asphalt pavements in cold regions. Also, the results of field performance of rubber modified asphalt concrete pavement for ice and snow control in the United States and Sweden installations are summarized.

2.0 USE OF RUBBER IN ASPHALT MIXTURES

2.1 Methods of Rubber Addition

Ground tire rubber has been used as an additive in various types of asphalt pavement construction in recent years. The use of rubber is
of interest to the paving industry because of the additional elasticity imparted to the binder.

In recent years, the most overlooked aspect of rubber modified asphalt is the attention it has received by Congress as it relates to solving the ecological problems of disposing of discarded tires. Congress, in order to stimulate the use of recycled materials, directed the Environmental Protection Agency and Federal Highway Administration to issue procurement guidelines. In response to the request, the February 20, 1986 issue of the Federal Register describes a proposed ruling by the Environmental Protection Agency for "Federal Procurement of Asphalt Materials Containing Ground Tire Rubber for Construction and Rehabilitation of Paved Surfaces" (9). The impact of this proposed guideline remains to be seen. However, many agencies are currently evaluating the use of discarded tires to modify hot mix asphalt for road surfacing (3,4,5).

Two different methods of incorporation ground tire rubber into paving mixes have been developed. The method of adding rubber to asphalt mixtures, which will be discussed in this paper, was originally developed in the late 1960's by Sweden and patented under the trade names of "Rubit" in Sweden and as "PlusRide" in the United States. In this system, rubber-asphalt mixtures are prepared by a process that typically uses 3 to 4% by weight of relatively large 1.6mm to 6.4mm (1/16in. to 1/4 in.) rubber particles to replace some of the aggregate in the mixture.

The second type of rubber modification (not discussed herein) uses finely ground rubber tier "buffings" which are mixed into the hot asphalt cement to create a "rubberized asphalt" binder, which is then added to a normal paving aggregate.

2.2 Mix Ingredients

Rubber-modified asphalt paving mix is prepared by a process that typically uses 3% by weight of granulated coarse and fine rubber particles to replace some of the aggregate in the mixture. Based on experiences in the United States and Sweden three different aggregate gradation bands have been recommended to serve different traffic levels (Table 1).

A review of these aggregate grading bands reveals some critical differences between rubber modified
and conventional mixtures. To provide space for the rubber particles, it is necessary to create a "gap" in the gradation curve for the aggregates, primarily in the 3.2mm to 6.4mm (1/8" to 1/4") size range as shown in Figure 1. Also, the rubber particles used in these mixes are to be produced in "roughly cubical form" from grinding of waste tires, which have had the steel wires in the tire bead area removed. The rubber may include some tire cord and steel fibers from tire belts, and must meet the gradation specifications in Table 2.

The asphalt cement in a rubber-modified mix is the same as used in the conventional mix. The mix typically requires 1-1/2 to 2% more asphalt than the conventional mix.

2.3 Mix Properties

A laboratory study was performed at Oregon State University to evaluate the effect of mix variations on properties of rubber-asphalt mixes. The asphalt cement (AC-5 produced by Chevron, USA's Richmond Beach Refinery) and aggregate (crushed river gravel from Juneau, Alaska) used in the study were obtained from Alaska DOT & PF. The recycled rubber was provided by Rubber Granulators in Everett, Washington. The Marshall mix design procedure was used to determine optimum asphalt contents for the different mix combinations. Once the optimum asphalt contents were determined for the different mix combinations, the resilient modulus and fatigue life characteristics of the mix were evaluated (6).

Mix Design Results: The laboratory mix design results indicate that the required asphalt content to reach a certain minimum voids level for rubber-modified mixes depends on the aggregate gradation, rubber gradation, and rubber content (Table 3). The laboratory results show the mixture with gap-graded aggregate and 3% coarse rubber (coarse rubber is defined as rubber particles with 80 to 90% in a size range from 2mm to 6.4mm (No. 10 to 1/4")); the remaining rubber content is buffings, in a size range from No. 40 to No. 10 required the highest design asphalt content (9.3 percent based on dry aggregate weights). Reducing the rubber content to 2% resulted in a reduction in asphalt content to 8.0%. The mixture with 3% coarse rubber and dense aggregate grading required 7.5%. The conventional asphalt mix (no rubber) had the lowest design
asphalt content (5.5 percent). The asphalt contents reported were all for 2% air voids (2).

Modulus and Fatigue Results: To evaluate the effect of mix variables on the elastic response of rubber-modified asphalt, 20 different mix combinations were considered. The resilient modulus (ASTM D-4123) and fatigue life characteristics were evaluated at two temperatures (+10 C and -6 C) (2). These variables included: two void contents, two rubber contents, three rubber gradations, two mix temperatures, two cure times, and use of surcharge. The mix property test results indicate that the modulus and fatigue life of rubber-modified asphalt mixes depend on rubber gradation, aggregate gradations, and rubber content. The mixtures with the finer rubber gradations had higher resilient modulus and lower fatigue life values than mixtures with coarser rubber gradations. In addition, the dense-graded aggregate has a higher modulus value. At -6 C, the fatigue life was less for mixes with gap-graded aggregate compared to mixes with dense-graded aggregate. This unusual performance is mainly due to the response of the rubber particles in the mixture. At +10 C the rubber particles are responding as an elastic aggregate. However, at -6 C the rubber particles lose their elasticity and in the gap-graded mixture work as a weak aggregate. Reducing the rubber content to 2% also resulted in higher resilient modulus and lower fatigue life values as compared to mixes with 3% rubber content. The findings of this study indicated that the rubber gradation, rubber content, and aggregate gradation have a considerable effect on required asphalt cement content, fatigue life, and resilient modulus of the mix. This study also showed that the rubber-modified mixes had a substantially greater fatigue life compared to a conventional mix (2).

3.0 CASE HISTORIES

3.1 Alaska Department of Transportation (5)

The Alaska Department of Transportation and Public Facilities installed twelve experimental pavement sections totaling 51.2 Lane-km (34.1 lane-miles) in Fairbanks, Anchorage, and Juneau between 1979 and 1986 utilizing different pavement mixtures to
analyze the benefits of rubber-asphalt pavements. The job mix formulas for both the rubber-modified and conventional mixes for this project are given in Table 4: Tests were conducted to determine the resilient modulus of Marshall compacted specimens. Also specimens with different contents of coarse and fine rubber and aggregate gradations were tested for fatigue.

Construction. The most common construction problem noted was achieving the proper gap in the grading curve, and obtaining sufficient fines (No. 200) to serve as a void filler. Both problems may be related to contractor inexperience in producing aggregate to the gap gradation requirements. In the preparation of rubber-modified asphalt mix, use of a "batch" plant was preferred because the required quantities of rubber, asphalt, and aggregates can be measured exactly and added separately to the "pugmill" or mixing chamber. However, both continuous mix and drum-dryer mix asphalt paving plants have since been used without difficulty. Mixing temperatures and asphalt grades used have been similar to those for normal paving mixes.

Post Construction. Results of all stopping distance tests made on the Fairbanks area rubber-modified sections from 1980 to 1983 indicate an average reduction of 25 percent from 30 to 22 meters (from 91 to 67 ft.) in stopping distance compared to conventional asphalt. The use of coarse sands for ice control in similar areas would normally result in reduced stopping distances for only a short period of time, as the sand rolls off under traffic action. Following the 1984 season, in which no new rubber-modified asphalt sections were constructed, seven new installations were placed in 1986. These installations totaled 41 lane km (27.2 lanes-miles) as detailed in Table 5. Most of these sections were constructed without significant problems. Minor construction problems occurred on the 1986 Minnesota-O'Malley project as a result of initial asphalt contents which proved too high, resulting in flushing and rutting of the mix at intersections. This was subsequently corrected by lowering the asphalt content from 7.5 to 7.0%, and dropping the mix temperature from 163°C to 143°C (325°F to 290°F). On the Airport Road project in Fairbanks, slight to moderate flushing of the asphalt was noted within a few days after placement, during unusually warm weather.
This effect was first noted, and became excessive, at the intersections. In these areas, the traction and braking forces caused increased mixture densification and the excess asphalt was flushed to the surface. Subsequent core testing revealed that the asphalt contents and aggregate gradations were well within specifications. However, the rubber contents appeared to average only about 2.1% as compared to the 3% specified. This resulted in a significantly reduced asphalt demand for the mix, and was considered to be the cause for the flushing. The principal effect has been increased slipperiness when wet and a reduction in the wintertime traction benefits which normally result from the use of rubber granules in a paving mix.

Stopping distance tests on Airport Road were done on six test dates with an average of 16 tests per date. Airport Road is a four-lane divided urban collector route, with signalized intersections at frequent intervals. The purpose of the test series was to evaluate the icy-road friction characteristics of the newly placed Airport Road rubber-modified pavement. Tests were performed in a 1984 Chevrolet Station Wagon, using a Bowmonk (brand) Brake-Meter. This meter is functionally similar to the Tapley (brand) Meter used in previous tests, and results are considered to be directly comparable.

In all tests performed on Airport Road during area-wide icy road conditions, the stopping distances were noted to be variable with traffic levels and with the degree of surface flushing. Tests on November 28, 1986, focused on the moderate and severe flushing areas, compared with non-flushed areas. Stopping distances at 40 kph (25 mph) averaged 30, 50 and 27 meters (91, 150, and 81 feet), respectively, for these three conditions. Comparisons between non-flushed and moderately flushed areas on December 5, 1986, showed an increase of 37% in stopping distances resulting from the flushing of excess asphalt to the surface of the pavement during construction. On this date, Airport Road was also compared with College Road which had a conventional asphalt pavement. Stopping distances were found to average 44 meters (133 feet) on College Road and only 27 meters (81 feet) on the non-flushed areas of the Airport Road PlusRide project, and improvement of 39%.
Stopping distance on Airport Road generally demonstrated that braking efficiencies were affected by traffic volumes and speeds and by the degree of flushing, as well as by the presence of the granulated rubber in the mix. While no extensive tests were conducted to investigate the effects of traffic levels and speed, it was noted that stopping distances were as much as 30% greater in the highest traffic sections of this route. These high-traffic sections typically have lower average speeds, more stop and go movements, and possibly more extensive asphalt flushing than other portions of the route. The effects of traffic on icy road surface slipperiness appear to result from the polishing action of sliding tires during braking and acceleration at stoplights, and possibly also from the condensation of exhaust water vapor onto the cold pavement surface. In Fairbanks the most common cause of icy roads is the formation of surface frost during almost every atmospheric warming cycle. Under these conditions, the combination of relatively warm moist air with cold pavement surfaces results in the condensation of moisture to form a thick frost layer. Traffic action then polishes the frost layer and further reduces the skid resistance. This condition commonly exists over a four month period during the winter season. It typically ends in March when the increasing solar radiation begins to warm the pavement surface.

Measurements were made in early 1987 of stopping distances on conventional and rubber-modified pavements, on four different paving projects in the Anchorage urban area. These tests were performed with a Tapley meter in a full-size pickup truck equipped with Uniroyal brand mud and snow tires. Testing on four different days when area roads were generally icy, showed an average reduction in stopping distances of 15% for the rubber-modified pavements.

3.2 Minnesota Department of Transportation (4)

To reduce the amount of chemicals used for ice and snow control without reducing the level of service Minnesota DOT constructed two experimental projects using a rubber-modified asphalt mix. Both test projects are four-lane divided highways and have a two-way average daily traffic of 10,000 with 17 percent being truck traffic. These projects were constructed during the month of September,
1984 and are approximately 1.0 km (0.7 miles) in length.

The job mix formula for both projects are summarized in Table 5. Tests were conducted to determine the diametral modulus of the Marshall compacted specimens. At room temperature, the resilient modulus value was between 126,000 and 188,000 psi. Typical conventional bituminous mixtures show a general range of resilient modulus values between 250,000 and 600,000 psi and some samples with higher value.

Construction. Both test sections were completed during September 1984. Project No. 1 is an overlay on an existing bituminous overlay on a PCC concrete pavement. The 32mm (1-1/4") thick rubber modified asphalt mixture was placed over a 25.4mm (1.0") leveling course. The construction of project No. 2 involved milling the existing bituminous pavement to a depth of 70mm (2-3/4") and then resurfacing with 38mm (1-1/2") of asphalt concrete leveling course and 38mm (1-1/2") of rubber modified asphalt mixture.

Dryer drum asphalt mixing plants were used on both projects. The granulated rubber material was added to the mixture at the center of the drum where salvaged materials are commonly added. After mixing, the granulated rubber, aggregate and asphalt cement were separated using vacuum extraction procedures. The test results indicate that some of the fine rubber particles may have been lost during production. It is not known if the loss of rubber material was due to the production process, absorption into the asphalt cement, or from the testing procedures.

Field compaction consisted of rolling the mixture with vibratory steel drum rollers. On both projects roller pickup was a problem. The density of the compacted mixture was measured using nuclear density tests. The nuclear test values were lower than the values obtained from roadway core samples.

This indicates a need for a correction factor for rubber-modified asphalt mixtures determined with a nuclear gauge. Field cores show that this correction is as much as 4 PCF. Modulus values for the cores were measured and the results were similar to the values determined for the trial
mixes. The resilient modulus values were from 114,000 to 121,000 psi.

Post Construction Evaluation. After construction of the rubber-modified test sections was completed, friction, and deflection measurements were taken on the projects. The lock-wheel pavement friction test (ASTM E-274) at 64 kmh (40 mph) was utilized to determine friction numbers. The test results on comparable sections indicate that the skid numbers for the rubber-modified asphalt test sections are greater than those on the conventional sections but not on this project.

The model 8000 Dynatest Falling Weight Deflectometer was used to gather surface deflection data on both projects. The testing was done to determine the increase in surface deflection, if any, owing to the incorporation of granulated rubber in the mix. The deflection at the center of the load was used to compare the deflection before and after the rubber modified asphalt mixture was placed. Testing was performed on both the rubber modified asphalt and the conventional mixtures. The measurements indicate that the difference in surface deflection between the rubber modified mixture section and the conventional mixture section is negligible.

To evaluate the ability of the rubber-modified asphalt wearing course to reduce the amount of snow and/or ice adhering to its surface, the test sections were observed under conditions of snow and ice. These observations were conducted during the winter of 1984-1985 and 1985-1986. The observations suggest that on several occasions the rubber-modified test section performed better than the conventional pavement. Field inspectors also noted that the rubber-modified test section appeared to absorb more solar energy which causes the snow to melt faster.

3.3 Rubber Modified Asphalt Concrete Field Experiences In Sweden

Swedish road authorities have installed several experimental pavement sections using the "RUBIT" rubber modified asphalt system to evaluate the effectiveness of this pavement system for snow and ice control. The majority of these experimental sections are located in areas where frost on the pavement surface caused more accidents than the
average road. According to the reports from road authorities, police and road users, the road sections with rubber modified asphalt concrete have caused considerable reduction in surface frost related accidents. Also, the road authorities reported, in some areas resurfaced with rubber modified asphalt pavement, the accidents related to surface frost have been completely eliminated (10).

Rubber modified asphalt concrete has also been used to improve the accessibility of roads by reducing the bond between ice layer and the surface of the pavement. In Karlstad, a city in the western part of Sweden, all ramps to and from the by-passes are paved with rubber modified asphalt concrete to eliminate the risk of stranded trucks and buses on slippery roads during the winter.

Based on observations from field installations, the effectiveness of rubber modified asphalt pavement for snow and ice control were investigated. The Swedish National Road Administration in their pavement policy stated that the use of rubber modified asphalt concrete reduced the risk of slippery roads caused by frost. Therefore, the use of rubber modified asphalt concrete is recommended for localities where slippery surfaces are caused by frequent frost. They also stated that the greatest effect of rubber modified asphalt concrete is seen during early winter when frosted surfaces will occur in only some pavement areas with the rest of the road having good friction.

The effectiveness of rubber modified asphalt concrete pavement will be investigated further by Swedish road authorities in additional field installations. In the summer of 1989 a 14 km test section in the middle of Sweden will be paved with rubber modified asphalt concrete. Also, two more roads, about 10 km each, will be paved with rubber modified asphalt concrete in 1990. The location of these two test sections will be, one in the south of Sweden, in an area with a coast climate and one in north Sweden with a typical winter climate. The Swedish Road and Traffic Research Institute (VTI) will be responsible for the evaluation these test sections to determine the long term effect of using rubber modified asphalt pavements as a snow and ice control system (11).
4.0 CONCLUSIONS

Based on the laboratory and field tests reported herein, the following conclusions appear warranted:

1) The laboratory mix design results show that the required asphalt content to reach a certain minimum voids level for rubber-modified mixes, depends on rubber gradation, aggregate gradation, and rubber content.

2) The resilient modulus properties of rubber-modified asphalt mixtures are significantly lower when compared to conventional mixtures.

3) The rubber-modified mixes have laboratory fatigue lives which range from 2 to 7 times longer than conventional mixes.

4) Rubber-modified asphalt mixtures are more susceptible than conventional mixtures to preparation and compaction problems when adverse weather or equipment problems occur. However, with adequate equipment and favorable weather conditions, the rubber-modified asphalt mixture placement is similar to conventional mixture.

5) Extensive measurements by Alaska DOT&PF showed an average reduction in stopping distance of 25% for the rubber-modified pavements in icy conditions. De-icing benefits have been reported by Minnesota DOT and Alaska DOT & PF in the United States and Swedish road authorities.
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Figure 1. Comparative Aggregate Gradation Curves for Conventional Asphalt and Rubber-Modified Asphalt Pavements.
Table 1. Recommended Aggregate Specifications for Rubber—Asphalt Paving Mixtures for Different Levels of Traffic (Takallou, 1987).

<table>
<thead>
<tr>
<th></th>
<th>PlusRide™ Mix No. 8</th>
<th>PlusRide™ Mix No. 12</th>
<th>PlusRide™ Mix No. 16</th>
</tr>
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<tr>
<td>Average Daily Traffic</td>
<td>2,500</td>
<td>2,500 to 10,000</td>
<td>10,000</td>
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<tr>
<td>Minimum Thickness (in.)</td>
<td>0.75</td>
<td>1.5</td>
<td>1.75</td>
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<tr>
<td>Aggregate % Passing</td>
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<td></td>
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</tr>
<tr>
<td>Sieve Size:</td>
<td></td>
<td></td>
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</tr>
<tr>
<td>3/4 in.</td>
<td>——</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>5/8 in.</td>
<td>——</td>
<td>100</td>
<td>——</td>
</tr>
<tr>
<td>3/8 in.</td>
<td>100</td>
<td>60-80</td>
<td>50-62</td>
</tr>
<tr>
<td>1/4 in.</td>
<td>60-80</td>
<td>30-44</td>
<td>30-44</td>
</tr>
<tr>
<td>#10</td>
<td>23-38</td>
<td>10-32</td>
<td>20-32</td>
</tr>
<tr>
<td>#30</td>
<td>15-27</td>
<td>13-25</td>
<td>12-23</td>
</tr>
<tr>
<td>#200</td>
<td>8-12</td>
<td>8-12</td>
<td>7-11</td>
</tr>
<tr>
<td>1/4 in. to #10 Size Fraction</td>
<td>——</td>
<td>12 max.</td>
<td>12 max.</td>
</tr>
<tr>
<td>Preliminary Mix Design Criteria</td>
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<td>Rubber, % of Total Mix (by wt.)</td>
<td>3.0</td>
<td>3.0</td>
<td>3.0</td>
</tr>
<tr>
<td>Rubber, % by Volume (approx.)</td>
<td>6.7</td>
<td>6.7</td>
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</tr>
<tr>
<td>Asphalt, % of Total Mix (by wt.)</td>
<td>8-9.5</td>
<td>7.5-9.0</td>
<td>7.5-9.0</td>
</tr>
<tr>
<td>Maximum Voids (%)</td>
<td>2.0</td>
<td>2.0</td>
<td>4.0</td>
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Table 2. Particle Size Specifications for Granulated Rubber (Takallou, 1987)

<table>
<thead>
<tr>
<th>Sieve Size</th>
<th>Coarse Rubber (% Passing)</th>
<th>Fine Rubber (% Passing)</th>
<th>80/20 Rubber Blend¹ (% Passing)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/4 in.</td>
<td>100</td>
<td></td>
<td>100</td>
</tr>
<tr>
<td>No. 4</td>
<td>70-90</td>
<td></td>
<td>76-92</td>
</tr>
<tr>
<td>No. 10</td>
<td>10-20</td>
<td>100</td>
<td>28-36</td>
</tr>
<tr>
<td>No. 20</td>
<td>0-5</td>
<td>50-100</td>
<td>10-24</td>
</tr>
</tbody>
</table>

¹The "80/20" is 80% coarse and 20% fine rubber in combination.
Table 3. Recommended Asphalt Content and Mix Properties @ 2% Air Voids (Takallou, et al.)

<table>
<thead>
<tr>
<th>Aggregate Gradation</th>
<th>Rubber Blend (%)</th>
<th>Rubber Blend (% Coarse/ % Fine)</th>
<th>Design Asphalt Content (%)</th>
<th>Marshall Stability (lbs)</th>
<th>Flow (.01 in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gap-Graded</td>
<td>2</td>
<td>0/100</td>
<td>7.0</td>
<td>920</td>
<td>15</td>
</tr>
<tr>
<td></td>
<td>60/40</td>
<td>7.2</td>
<td>690</td>
<td>21</td>
<td></td>
</tr>
<tr>
<td></td>
<td>80/20</td>
<td>8.0</td>
<td>665</td>
<td>23</td>
<td></td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>0/100</td>
<td>7.5</td>
<td>600</td>
<td>19</td>
</tr>
<tr>
<td></td>
<td>60/40</td>
<td>7.5</td>
<td>650</td>
<td>22</td>
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<tr>
<td></td>
<td>80/20</td>
<td>9.3</td>
<td>436</td>
<td>33</td>
<td></td>
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<tr>
<td>Dense-Graded</td>
<td>0</td>
<td>No Rubber</td>
<td>5.5</td>
<td>1500</td>
<td>8</td>
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<tr>
<td></td>
<td>3</td>
<td>80/20</td>
<td>7.5</td>
<td>550</td>
<td>22</td>
</tr>
</tbody>
</table>

Table 5. Job Mix Formulas for Minnesota Experimental Projects.

<table>
<thead>
<tr>
<th>Project No.</th>
<th>Aggregate</th>
<th>Rubber</th>
<th>% of Total Mixture (120/150 pen)</th>
<th>Granulated Rubber %</th>
</tr>
</thead>
<tbody>
<tr>
<td>(1)</td>
<td>5/8 in.</td>
<td>100</td>
<td>100-100</td>
<td>7.5 ± .3</td>
</tr>
<tr>
<td></td>
<td>3/8 in.</td>
<td>70-80</td>
<td>70-80</td>
<td>3.0 ± .15</td>
</tr>
<tr>
<td></td>
<td>No. 4</td>
<td>35-40</td>
<td>No. 4</td>
<td></td>
</tr>
<tr>
<td></td>
<td>No. 10</td>
<td>25-32</td>
<td>No. 10</td>
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<tr>
<td></td>
<td>No. 30</td>
<td>17-25</td>
<td>No. 20</td>
<td></td>
</tr>
<tr>
<td></td>
<td>No. 200</td>
<td>8-11</td>
<td>No. 20</td>
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</tr>
<tr>
<td>Densities (pcf):</td>
<td>Maximum Theoretical = 146; Marshall = 143.4</td>
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<tr>
<td></td>
<td>Target = 138.7-143.1</td>
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<tr>
<td>(2)</td>
<td>5/8 in.</td>
<td>100</td>
<td>100-100</td>
<td>7.8 ± .3</td>
</tr>
<tr>
<td></td>
<td>3/8 in.</td>
<td>70-80</td>
<td>70-80</td>
<td>3.0 ± .15</td>
</tr>
<tr>
<td></td>
<td>No. 4</td>
<td>35-38</td>
<td>No. 4</td>
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<td>No. 10</td>
<td>22-30</td>
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<tr>
<td></td>
<td>Target = 137.9-142.2</td>
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*5% mineral filler added

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<tr>
<td>1/2 in.</td>
<td>53-57</td>
<td>52-76</td>
<td>47-60</td>
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SHRP's Concrete and Structures Research: Goals and Recent Developments

Damian J Kulash
Director
Strategic Highway Research Program (SHRP)
USA
SHRP's Concrete and Structures Research: Goals and Recent Developments

Damian J. Kulash
Director
Strategic Highway Research Program

1. Introduction - Chapter 1

SHRP's program in Concrete and Structures has two major goals: increasing the durability of portland cement concrete in highway applications; and rehabilitation and protection of existing steel-reinforced concrete bridges subject to corrosion. In both cases, the research has been planned to result in tangible products that can be applied in the field immediately and have the potential to result in substantial savings in the construction and operation budgets of highway agencies.

Concrete and Structures contracts will total $18.3 million; $9.3 million in Concrete (Figure 1) and $9.0 million in Structures (Figure 2). The potential economic benefits of applied research in portland cement concrete are substantial. The U.S. highway industry spends about $1 billion annually on portland cement, accounting for about 16 percent of portland cement sales in the United States. Almost all bridge decks are portland cement concrete, as are piers and abutments, even where steel is used for the superstructure. Over 85,000 miles of U.S. roads are paved with portland cement concrete, which is also the material most commonly used for curbs, sidewalks, and median dividers.

The United States currently has about $20 billion worth of repair work to do on corroded steel-reinforced concrete bridges. That figure is growing by about $500 million annually. More economical methods are needed to arrest the spread of corrosion and to repair the damage to thousands of bridge decks constructed between 1950 and the mid-1970s. These bridges were constructed with insufficient (typically 1 1/2 inches) of concrete cover, and without epoxy-coated rebars or other methods of corrosion protection that have come into widespread application subsequently. Bridge repair will be the single most costly expenditure for our highway system in the next decade.
1.2 Objectives of SHRP's Concrete Research

The general objectives of SHRP's concrete research are:

- To enhance the constructability and durability of portland cement concretes. More durable concrete will be easier to build—less prone to mistakes in mix design, in mixing, in placement, or in curing;

- to characterize new concrete materials with high-performance characteristics;

- to develop better techniques for quality assurance; and

- to develop processes for increasing the service life of existing concrete pavements and highway structures.

SHRP will make mix design more reliable and efficient by providing engineers with needed guidelines for specification of various cements, admixtures, and aggregates. In this era of budget austerity, cost-effective materials use is more important than ever to highway agencies. SHRP is developing new tests that will permit rapid, accurate materials characterization, which, when combined with improved engineering guidelines, will enable highway agencies to use more economical cements and aggregates with confidence. New tests and guidelines also will take the guesswork out of use of waste materials such as fly ash, ground slag, and other pozzolans. This will have the two-fold benefit of contributing to the solution of solid waste problems while encouraging the use of these materials as performance-enhancing admixtures.

Quick, accurate, easy-to-use field tests are the key to more durable concrete construction. Quality control and quality assurance at the construction site have become increasing concerns in recent decades as the training and experience level of construction site labor has decreased. Close field supervision by qualified engineering personnel is needed, but that, too, is in short supply. Most of the quality control/
quality assurance site tests currently in use must be performed by qualified personnel. Moreover, these tests are too time-consuming and do not give sufficiently precise results. SHRP is developing non-destructive field tests for quality control that can be applied by minimally trained personnel and produce quick, accurate results. These can form the basis for new systems for quality control acceptance, form stripping, and placing-in-service.

As the U.S. infrastructure ages, prolonging the service life of existing pavements and structures is of great importance. SHRP is developing new nondestructive tests for assessing the condition of in-place concrete, as well as new methods for repair and prevention of further deterioration.

The foundation of the concrete research program is investigation of the properties of cement and concrete systems at the microstructure level. (Contract C201, Pennsylvania State University). The microstructure research projects will have tangible products: improved engineering guidelines, new tests and materials, and recommendations for their application.

Other SHRP concrete projects are focusing on specific performance concerns; alkali-silica reactivity (Contract C202, Construction Technology Laboratories) and freeze-thaw damage, including D-cracking (Contract C203, University of Washington). Again the emphasis is on usable results. SHRP is developing improved tests and techniques for avoiding these problems in new construction, and for correcting them in existing construction. SHRP also is evaluating new materials that will be resistant to a wide range of deteriorative processes. The researchers will select the concrete processing techniques and materials combinations that have the best potential for superior performance with a range of aggregate types, and develop criteria and recommendations for their development and application.

2. RECENT AND POTENTIAL INNOVATIONS IN CONCRETE - CHAPTER 2

Research and innovation are not new to the concrete pavement industry. Looking back just 30 years, numerous improvements are evident--
including innovations in materials, designs, and construction techniques.

In the materials area, we have new cements and admixtures; we are making greater use of pozzolans and recycled materials; we can design more versatile—for example early-strength gain; we have improved corrosion-resistant joint hardware; better-performing joint-sealing materials are in use; and we have new drainage materials, including fabrics, underdrains and open-graded bases.

Our designs, concepts, and applications have evolved to meet the challenge of improving the performance of both new pavement construction and rehabilitation. For example, we have CPR, fast-track, and whitetopping as rehabilitation alternatives. We have improved our techniques for drainage of pavement structures; improved design details for all concrete pavement types—including plain, reinforced, Continuously Reinforced Concrete Pavement, and even some prestressed structures.

In the area of equipment and construction, we have witnessed dramatic improvements over the past 30 years. For instance, compare the difference between the 34E Paver in use in the 1950s to a modern-day high-capacity central-mix plant, feeding a mile-a-day and multilane slipform paving train! We have mesh depressors, dowel implanters, grinding and sawing equipment, texturing devices, and improved equipment for controlling and measuring roughness.

2.1 Products Expected From SHRP

Despite the tremendous recent progress in concrete industry, much remains to be done, and SHRP, as one of the largest concrete research programs ever, has ambitious goals. SHRP research is attempting to take the guesswork out of concrete construction. There are a number of situations where engineers are forced to guess, or to cut corners, because the available techniques are too time-consuming, too unreliable, too expensive, or too difficult to use. For example:

- We know we need to avoid reactive aggregates, but project schedules don't always give us the 12 or more months we
need for the mortar bar test, so we use the shorter chemical method, and hope it is accurate enough.

- This is risky and may prove costly. SHRP is developing a faster test for reactive aggregates, and also is developing screening criteria for aggregates so that the likely reactivity of a larger range of materials can be assessed even before testing.

- We thought we knew how to get freeze-thaw durability by controlling air content. If we kept air in the 4- to 8-percent range, we generally stayed out of trouble. Then we started adding fly ash and other modifiers to improve performance or save money. Our conventional guidelines on air voids don't always work with these new materials. To get the best combination of strength and freeze-thaw durability, we sometimes need to get air voids down under the old range, but there is a lot of guesswork involved. SHRP is exploring the relationship between air voids and performance in modified concretes, and will produce more specific guidelines that get rid of some of this guesswork.

- The air meters that we now use to measure air content do the job, but they are not easy to use. Calibration is difficult, and if the test is not conducted carefully, it can yield false results. The degree of skill and care needed for the test can prove to be difficult to achieve at the construction site. To make this test more reliable under these conditions, SHRP is developing an automated air-void measuring device that will make calibration simpler, and that will automate some of the aspects that now cause problems. The result will be a device that makes monitoring of this critical factor easier and more reliable.

- Improved techniques for predicting how quickly concrete pavement strength will develop can facilitate work scheduling and make it possible to open pavements to traffic sooner. SHRP will evaluate and refine the various existing strength-
prediction techniques. As a result, we will be able to make decisions about strength development earlier and more confidently.

Many professionals involved in the design and rehabilitation of concrete pavements have identified the gaps in current knowledge and practice and have planned the research needed to fill them. Research contractors are executing those plans now. Through SHRP they will develop a number of products, including better techniques for preventing and repairing problems such as alkali-silica reactivity, freeze-thaw damage, and D-cracking; an improved quality control/quality assurance system, with emphasis on quicker, more reliable non-destructive tests; and better design guidance for use of admixtures and aggregates.

3. CONCRETE BRIDGE COMPONENTS: BACKGROUND - CHAPTER 3

The principal cause of the rapid and premature deterioration of concrete bridge decks is the use of road salt in winter maintenance operations. The salt penetrates the concrete and causes corrosion of the embedded reinforcement. In marine environments, sea salt has the same effect.

Once it begins, corrosion is extremely difficult to arrest. As the steel corrodes, it expands and creates stresses that eventually cause the concrete to break, resulting in cracking and deterioration of the bridge deck surface. The same process also causes deterioration of piers, abutments, and other concrete bridge components, which must be repaired to prevent serious structural damage. In extreme cases, concrete falling from bridge components can be a hazard to road or marine traffic under the bridge. Repair of bridge substructures present major logistical problems. The repair of substructures has received less attention than the more obvious problem of bridge deck corrosion. SHRP is devoting significant attention to repair of corroded bridge substructures as well as deteriorated bridge decks.

Corrosion is an electrical phenomenon that occurs where both moisture and oxygen are present. Steel embedded in concrete is normally protected from corrosion by the high alkalinity of the water in
the concrete pores. The alkaline substances help to form a protective oxide layer on the steel, which grows impervious to further corrosion.

The chloride ion in salt can break down the protective layer on the steel, allowing corrosion to proceed rapidly. Once in sufficient concentration at the reinforcing steel, the chlorides act as catalysts, promoting the corrosion reaction without being consumed. The product of corrosion—rust—is at least twice the volume of the steel. Even very small amounts of rust generate pressure within the concrete. Concrete is brittle, and will crack and break with as little as one-hundredth-of-an-inch increase in the diameter of the reinforcement bar. Such a small amount of rust can be produced in one year of corrosion. Corrosion-induced deterioration leads to a continual cycle of failure and repair as corrosion proceeds to break off (delaminates) the concrete cover of the reinforcing steel.

Present methods for detecting the presence of corrosion before spalling becomes visible are generally inconclusive and indirect. One extensively used technique is to measure the electrical potential of the steel against a standard half cell. This measurement indicates the susceptibility of that particular area of the reinforcing to corrosion. This technique is limited in its applicability and is not always accurate. Another diagnostic method is to drag a chain across the bridge deck and listen for hollow sounds that would indicate delamination of the subsurface concrete. Neither of these methods provides a direct measure of the amount of corrosion or its rate.

In the absence of effective diagnostic tools, corrosion usually goes undetected until damage is visible. Even then, highway engineers have difficulty assessing the amount of damage and its location. At present there are no nondestructive methods for detecting cracking (delamination) under asphalt-covered decks or in bridge support structures without direct access. Nor are there methods for measuring the effectiveness of membranes and sealants that are used on decks and substructures to keep salt out. Field techniques also are needed for measuring the chloride content of the concrete, and its water and chloride permeability.
Information regarding the rate of corrosion is useful in determining the best time and method for repair. Devices that can measure the rate of corrosion of steel in concrete are in the early stages of development. These devices have not been evaluated and field tested uniformly. If the available techniques prove effective, development of a standard application procedure would be necessary.

If all of these field test methods are developed—and it seems technically feasible to do so—highway engineers would for the first time have adequate means to diagnose the extent and location of chloride damage in bridges and other concrete structures, and to choose the best repair technique.

4. CONCRETE BRIDGE COMPONENTS: PLANNED SHRP PRODUCTS - CHAPTER 4

SHRP will develop diagnostic techniques for assessment of the physical condition of concrete bridge components, including:

- A method for measuring the corrosion rate of steel imbedded in concrete;
- A method for detecting delamination and cracking in bridge component structures, and another for detection of delamination under asphalt-covered decks;
- Methods for measuring the effectiveness of membranes and sealers used on decks and substructures;
- Field techniques for measurement of the chloride content of concrete; and
- Methods for measuring the water and chloride permeability of existing concrete.

All of these techniques will be field tested and calibrated, and fully documented, with the objective of producing recommended specifications for adoption by one or more of the national consensus standard-setting organizations. SHRP also will develop improved techniques for repair and protection of concrete structures. Three different types of approaches are being refined.
within SHRP—electrochemical techniques, chemical corrosion inhibitors, and improvement of conventional physical repair methods.

4.1 Electrochemical Repair and Protection Techniques

Cathodic Protection: Corrosion occurs at anodic sites, which are positively charged; but not at cathodic sites, which are negatively charged. Cathodic protection involves installation of a low-voltage electrical circuit between an external anode and the reinforcing steel, so that all of the steel becomes cathodic. The feasibility of cathodic protection has been established over the past decade through successful application to bridge decks and, to a lesser extent, to substructures. Further assessment is needed to develop guidelines for selecting the optimal systems for specific application. Development of formal engineering criteria is needed to allow cathodic protection to achieve widespread application as a repair technique. Cathodic protection is one of the few technologies that can stop corrosion regardless of the chloride content of the structure. The payoff for development of cathodic protection techniques that are reliable, easy to use, and easy to monitor will be very large.

SHRP will:

- Survey the performance of existing cathodic protection installations, and conduct field tests of deck and substructure systems in a range of environments.
- Evaluate the control and protection criteria used in operation of cathodic protection systems, as well as the long-term effects of cathodic protection systems and the durability of their components.
- Develop a guide to installation of cathodic protection systems. This will provide highway engineers with information on design, construction, acceptance, activation, and monitoring of cathodic protection systems.
- Make recommendations for cathodic protection system specifications.
Electrochemical Chloride Removal: Because chloride ions are negatively charged, a strong electric field applied between a temporary anode on the concrete surface and the reinforcing steel will pull the chloride ions away from the steel and out of the concrete. This technique has been attempted in small-scale applications only. If it proves feasible and cost-effective, this technology could make it possible, in the course of a few days, to remove chloride from a bridge, and then to seal it against further damage with a coating or overlayer. This would be a major technical breakthrough with a large pay-off. However there is a risk of damage to the steel or the concrete.

SHRP will:

- Undertake laboratory studies to determine the limitations of electrochemical chloride removal and the likelihood of structural damage. If the laboratory tests indicate that electrochemical chloride removal is not feasible, the project will be terminated.

- If feasibility is proven, SHRP will develop practical engineering methods for applying the technique, as well as methods for sealing the concrete after chloride removal to prevent recontamination.

- After field testing, SHRP will develop an implementation package for use by highway engineers.

Electrochemical Injection: An alternate method of protecting the steel without removing the concrete is to inject to the level of reinforcing steel a protective chemical using a temporary anode. SHRP will investigate the feasibility of using an electric field to drive corrosion inhibitors into the concrete. These inhibitors are electrically charged and can be attracted to the steel by forced diffusion through the pores of the concrete. If the inhibitors prove effective and stable, then this innovative approach will provide a valuable new tool for bridge engineers.
4.2 Chemical Protection

The cost-effectiveness and performance of chemical treatments that can be applied to in-situ concrete to inhibit corrosion need to be evaluated. These techniques include chloride and oxygen scavengers that will "absorb" the incoming chlorides and oxygen; corrosion inhibitors; methods for removing water (dewatering); and in-situ polymerization.

SHRP will conduct a field and literature survey of existing conventional and chemical repair and protection techniques, and determine the feasibility of various new approaches to protecting corroding bridge components. These include:

- Techniques for cost-effective removal of concrete from decks and substructures.
- New techniques for repair and protection such as oxygen and chloride scavenging, chemical dewatering, corrosion inhibition, and polymer impregnation, along with ways of minimizing the volume of concrete removal while still protecting the steel from subsequent corrosion.

After field testing, SHRP will produce a manual that documents effective methods for application of the repair techniques found most effective.

4.3 Conventional Repair Techniques

At present the prevalent repair technique is to remove and replace some or all of the cracked and chloride-contaminated surface concrete, and subsequently add a low-permeability overlayer to stop further salt ingress. This is an expensive and difficult process that is prone to failure if active corrosion continues. The cost-effectiveness of various methods for removing salt-contaminated concrete has not been assessed completely in terms of such factors as speed, control, effectiveness, and damage risk. Methods of concrete removal include high-pressure water jetting, scarification (milling and grinding), and jack-hammering.

Where the conventional concrete removal-and-replacement method is applied, insufficient information is available regarding the amounts of concrete that must be removed (determining the
minimal amount is an important economic consideration), and the relative merits of replacement materials. Better information is needed regarding the most chloride-resistant mixes, and the use of corrosion-inhibiting patch and repair mixes, coatings, and sealers.

4.4 Repair Manual

The results of SHRP's work in all three corrosion protection and repair techniques (electrochemical techniques, chemical protection, and conventional physical repair) will be combined into a manual. The manual will provide a rational, cost-effective approach to bridge repair.

Drawing on the results of other SHRP structures research, a decision model will be presented detailing how to assess the rate of deterioration, options for repair, constraints on each option, and the life-cycle costs of repair options. The decision model will be tested, validated, and calibrated through field trials by highway agencies. The users' manual will present the decision model and explain how it is used and adjusted for various field conditions.
Concrete Microstructure Research and Its Applications in Highway Constructions

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ABSTRACT

The purpose of this research is to generate knowledge which will enable improvement in the quality of highway concrete by controlling the development of the microstructure. This study emphasizes those features which will aid in the management of processing of fresh concrete and early curing of concrete in highway construction work.

The specific program of the research in progress is an outgrowth of experience which has shown that the homogeneity and the workability of fresh concrete, and the conditions to which the concrete is exposed during its early curing, have a profound influence on the homogeneity and associated characteristics of the microstructure of the hardened concrete, and thereby on the durability of concrete structures. This research is therefore concentrating initially on an exploration of the principles of mixture proportioning of the concrete components which determine the packing characteristics of the concrete components as mixed, and also on the chemical reactions in the fresh cement paste, which influence its rheological characteristics. This knowledge is indispensable for the development of design methods, which can result in practical monitoring for optimum quality of the fresh concrete, forming the basis for the design of the corresponding monitoring methods.

Other emphases of the research include the adaptation of methods for monitoring the early curing of concrete. This is important because thermal stresses originating during early curing may cause severe microcracking, and available systems for monitoring the curing appear to be adaptable for application to highway construction work. Finally, diagnostic methods for evaluation of the microstructure and associated properties are the focus of much of the current effort.
INTERNATIONAL CONFERENCE
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AND TRAFFIC SAFETY ON TWO CONTINENTS

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CONCRETE MICROSTRUCTURE RESEARCH AND ITS APPLICATIONS
IN HIGHWAY CONSTRUCTIONS

by

G.M. Idorn*

ABSTRACT

The paper surveys the concrete technology issues of the SHRP C-201 project: "Microstructure of Concrete" of which professor D.M. Roy, Materials Research Laboratory, The Pennsylvania State University, USA, is principal investigator and G.M. Idorn Consult A/S together with F.L. Smidth & Co., Denmark, are subcontractors.

Emphasis in this presentation of the project is on engineering application of the research results and of supplementary, available knowledge. As the research is still in progress through to 1990, the presentation is preliminary as far as data input and concluding evaluations are concerned.

The contents concentrate especially on the properties of fresh concrete and curing, because the concrete production has an overbearing impact on the performance quality of concrete in highway structures. Further illustrated is how optical microscopy can be used to reveal essential features of the microstructure of concrete in service.

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INTRODUCTION

It may not be easy for highway engineers in any country on the basis of experience to envisage that the microstructure of concrete is something sincerely related to highway engineering practice.

In most countries, the prevailing perception is still simply that the lower the w/c of fresh concrete is, the higher the strength of the hardened concrete, and therewith follows denseness and "automatically" also concrete durability. On principle and qualitatively this is also true. The classical clarification of the w/c-strength interrelationship, e.g. by Bolomey, and of the w/c-permeability interrelationship, e.g. by Powers, have served the development of concrete technology and construction with singular profitability, considering the means available for their establishment compared with their subsequent unanimous worldwide acceptance.

Why then touch upon such holy columns of general concrete practice?

There are several reasons indeed not to devaluate or discard the classical knowledge, but rather for profoundly refining it and updating the guidance for the making of modern durable concrete.

The classic perceptions were based on simple empiric site testing of concrete specimens made with the prevailing Portland cement, sand, stone and water, and handled and cured as one would handle and cure laboratory samples. In those days, i.e. before about 1950, these were the conditions all over under which concrete was produced in engineering practice. Usually, for instance, concrete construction work was stopped during winter seasons and precast fabrication with steam curing had not yet been developed. Chemical admixtures, high strength/heat yielding Portland cement based upon high C₃S contents and fine grinding of the cements, or high cement contents in concrete were not in use. Moreover, during the performance of concrete in comparatively low-stress reinforced or unreinforced structural systems, there were not ever increasing high frequency vibrating live loads to contend with, or the use of deicing salts in the winter season for that matter. Besides, severe environmental climatic conditions were usually left uninhabited or requiring only scarce construction
investments. Altogether, the classic concrete was made and used under consistently benign conditions vis a vis concrete of today.

The boom of reconstruction and new developments all over in the industrial parts of the world since about 1950 have changed all that.

In highway construction with concrete as a remarkably versatile material for pavements, bridges etc. the N. American development ran far ahead in time and extent of the post World War II European programs. The acquired classical technology knowledge and skill were, though, so solidly established and easily adaptable, that it remained the principal working basis. This was despite the development within the source materials, especially cement and admixtures, the mechanisation and acceleration of the production of concrete, and the concurrent innovations within the design of structures and elements of structures. Winter concreting also became widely adopted, and with the increasing need for all-year safe automobile traffic deicing was introduced later on and profusely applied. Traffic intensities have generally also far exceeded what the highway systems and the concrete were designed for.

Meanwhile, the development of knowledge regarding the basic structure of hardened cement paste and concrete took major steps forward along with the industrial innovations of the analytical instrumentation for cement and concrete research. Optical microscopy, electronic transmission, scanning and back scanning microscopy, x-ray diffractometry, mercury porosimetry, calorimetry are among the most spectacular means made available during the 1960's and 1970's for the advancement of cement and concrete research.

Concurrent with this progress in engineering practice and research - or rather with a certain, but in many cases astonishingly brief, delay - there have appeared unprecedented and unsatisfactory early deterioration of concrete highway structures. The USA highway authorities have, under the impact of the complexity of possible reasons for this, activated coherent and concerted research programs on a number of issues, and done so most powerful and systematic in the SHRP program.

The microstructure of concrete, SHRP C-201 project, with the Pennsylvania State
University, professor Della Roy, as principal contractor and G.M. Idorn Consult A/S with F.L. Smidth & Co. as subcontractors - is one of these.

The purpose of this research is to generate knowledge which will enable improvement in the quality of highway concrete by controlling the development of the microstructure. The study emphasizes those features which will aid in the management of processing of fresh concrete and early curing of concrete in highway construction work.

The specific program of the research is an outgrowth of experience which has shown that the homogeneity and the workability of fresh concrete, and the conditions to which the concrete is exposed during its early curing, have a profound influence on the homogeneity and associated characteristics of the microstructure of the hardened concrete, and thereby on the durability of concrete structures. The research is therefore concentrating initially on an exploration of the principles of mix proportioning of the concrete components which determine the packing characteristics of the concrete components as mixed, and also on the chemical reactions in the fresh cement paste, which influence its rheological characteristics. This knowledge is indispensable for the development of design methods, which can result in practical monitoring for optimum quality of the fresh concrete, forming the basis for the design of the corresponding monitoring methods.

Other emphases of the research include the adaptation of methods for monitoring the early curing of concrete. This is important because thermal stresses originating during early curing may cause severe microcracking, and available systems for monitoring the curing appear to be adaptable for application to highway construction work. Finally, diagnostic methods for evaluation of the microstructure and associated properties are the focus of much of the current effort.

THE MICROSTRUCTURE OF CONCRETE

In the context used in the present research the microstructure designates the morphological characteristics and arrangement, and the mineralogical/chemical composition of the compounds which form the matrix in concrete as visually
observed at the microscopic level. It also includes the arrangement of pores, capillaries, and other flaws in the structure.

The common perception of microstructure is associated with the characteristics of hardened concrete. Thus, the proportion of hydrous and unhydrated phases (the degree of hydration of cement paste), its porosity and pore structure, the occurrence of interstitial compounds like crystalline calcium hydroxide and ettringite, and the nature and structure of the bond between cement paste and aggregates are essential descriptive features of the microstructure. Homogeneity of the cement paste matrix is a desirable quality, whereas a heterogeneous or segregated microstructure often makes concrete susceptible to deterioration caused by any kind of deleterious physical, mechanical, or chemical factor.

Optical microscopy with ranges of magnification of up to about x400 has been developed to identify and classify the components and the composition and morphology of the microstructure of hardened cement paste and concrete. The multinational experience has also made quantification possible of the microscopic observations for characterisation of the quality or deterioration of concrete in the macroscopic and megascopic regimes.

The scanning, transmission, and back scattering electronic microscopy operates at ranges of magnification of up to x100,000. This can reveal essential "submicroscopic" features of the basic structure of the materials examined. The research has embarked upon the problems of development of quantification of the submicroscopic observations as part of a modelling system. This would, through the microscopic range of observations also represent basic features of the macro and megascopic features, and incorporate interpretations of chemical and physical measurements.

As a result of investigations of individual cases of concrete structures showing evidence of deterioration, the alterations of the microstructure have long been recognized as essential elements in the diagnosis of different deleterious processes, their stages, and their damaging effects. Examples are observations of micro (and macro) cracking, carbonation, occurrence of alkali silica gel, abundance of secondary sulfate compounds, etc. However, until recently, it has been less widely recognized in engineering practice that the processing stages of concrete making may significantly affect the microstructural development so
as to make it either sound and an essential factor for ensuring the durability of concrete in service, or alternately to build in initial major defects. Consequently, the means and methods to enable the design engineers to specify and the production engineers to carry out suitable monitoring of fresh concrete processing during mixing and placement, and during the curing stage have not yet been established and implemented in updated highway concrete construction practice.

RESOURCE ECONOMY

A recent survey of the natural aggregate situation in the United Kingdom (Concrete, August 1989), reveals concern about the growth of consumption for concrete vis a vis the resources. Concurrently the UK is exporting aggregates to the USA, due to local depletions (ENR, 29 June 1989). Other signals illustrate tendencies in the same direction elsewhere. Cement manufacturing technology is developing so as to make the reduction of alkali contents difficult or impossible (Worning and Johansen, 1983). The writer believes that more penetrating analysis of the resource problems will enhance "the writing on the wall" of required attention towards exhaustion of concrete materials.

The SHRP C-201 project thus also becomes an element of R&D aiming at improved resource economy of highway concrete construction technology.

The microstructure comes in this respect to the fore because a dense homogeneous microstructure created in the fresh concrete and preserved during the curing represents resource economy during the production phase (including the resources spent by the production means, i.e. the concrete making, transportation of materials etc.). The consequential beneficial effects on durability represents resource economy during the lifetime of concrete in service in the highway systems.

In the overall synthesis of the SHRP concrete research programs the resource economy aspects of C-201 will then become an integral part of the entity.
BACKGROUND AND PREVIOUS RESEARCH

Earlier collaborative research (Roy and Idorn, 1982; Wu and Roy, 1983; Idorn and Roy, 1984; Idorn and Roy, 1985) pointed to some specific areas of microstructural improvement and revealed essential knowledge concerning the advantageous influence of granulated blast furnace slag on the microstructure of cement paste. This was found to be partly related to the influence of the slag on the rheological properties of the cement paste in its fresh stage and during the hydration process (Roy, et al, 1982). These initial effects were found to result in improved homogeneity of the cement paste and lower porosity and permeability of the hardened concrete. Interrelations between physical and chemical effects were found, and the reactivity of alkalis from the Portland cement with the slag component of the cement paste was found to affect beneficially the porosity and pore structure and the long term microstructural development in concrete. Concurrent research in Europe, (Bakker (1981), Regourd (1980), Smolczyk (1980)) and petrographic examinations by G.M. Idorn and N. Thaulow of 40 - 50 year old European concrete made with slag cement confirmed that actual concrete made with blends of slag and Portland cement exhibits very dense, impermeable and homogeneous microstructures. In this the amorphous slag hydration products were associated with an increased density of the microstructure compared with the more fibrillilic structure of pure Portland cement paste.

Comprehensive research at the Pennsylvania State University's Materials Research Laboratory (MRL) on the effects of fly ash and silica fume showed, in accordance with comprehensive, contemporary research elsewhere, that the same advantageous effects on the microstructure are attainable when Portland cement is used in blends with fly ash and silica fume and the reduction in permeability which can be achieved. (Kumar and Roy, 1986; Roy, 1986; Roy, 1989a,b). The progress of knowledge attained through this and contemporary research elsewhere has also been documented in the recent ACI technical reports on the use of blast furnace slag, fly ash and silica fume in concrete.
SUMMARY OF OBJECTIVES, APPROACH AND ACCOMPLISHMENTS

As described above, the goals of the project are to develop the technical bases for establishing the means to control concrete microstructural development and thereby improve concrete durability. This will be achieved through improved understanding of the factors controlling cement hydration, the development of concrete microstructures, how the microstructure controls the properties, and how the microstructure and properties can be optimized to increase the durability of concretes in various environments and applications. This project thereby focuses on the means to develop reproducibly improved concrete microstructures, with low permeability.

FIGURE 1 summarizes in a simple form the system to address the concrete microstructural development towards durability. This follows, generally, the progression from factors to be controlled (input parameters); models which relate to the design of a concrete mixture, the mixing and curing stages; microstructural development; and consequences for concrete properties, especially durability.

Currently, the following have been the focus of efforts and accomplishments:

* A packing model has been developed that both predicts space filling and can be related to rheology (workability).
* Comparisons have been made in the workability of concrete as determined by various instrumental methods (e.g. Tattersall) and common measurements such as slump.
* A model has been developed to describe the pore size distributions in cement.
* A preliminary, computer based model simulating the packing at an interface has been developed.
* The outline of a computer based guide has been developed for evaluation by highway engineers of the need for special curing precautions in highway pavement construction.
* An experimental apparatus to rapidly measure permeability has been designed, constructed, tested, and experiments have been initiated.
* Thin section petrography has been applied for characterisation of the
microstructure of laboratory samples of concrete for comparison with experience from petrography of samples of field concrete.

* An automated image analysis method to determine porosity is under development.

In the paper "Concrete Microstructure - a Key to Durability and Performance" presented by D.M. Roy at the MRS symposium "Materials Problems of Infrastructure" the scientific aspects of the C-201 projects are discussed. The concluding last year's work of the project is now approaching. Our company as subcontractor has concentrated on the engineering side of the project research. We are also as consultant engineers, and based on previous research, experienced in the fields of:

* Packing and rheology of fresh concrete
* Application of modern curing technology
* Studies of the microstructure of concrete by optical microscopy

G.M. Idorn Consult A/S has contributed to the work program on this basis of experience, while concurrently we have made progress in the same fields in a doctorate thesis (P.J. Andersen, 1989) and in studies for clients, other than PSU/SHRP. For the 3rd year program we are proposing advisory guidelines to be developed for highway engineers within the above project areas including accomplishments from our company as supplementary and complementary to the MRL research accomplishments.

PACKING AND RHEOLOGY

The most densely packed unisized balls leave a pore space of about 38 vol. % between the touching points of the ball surfaces. Obviously, smaller balls in the pore spaces would increase the denseness of the system, i.e. the packing.

Particle size grading of the solids and mix compositions of cement, sand and stone - fine, medium, coarse - represent the potential for dense packing, which reduces the requirement for water and the ultimate porosity of the concrete. Therewith dense packing favours the two decisive performance qualities of concrete: strength and durability.
For the purpose of making it possible in engineering practice to optimize particle size gradings a computer coded model has been developed. It can be used for prediction of the porosity (i.e. denseness) of a given mix composition of the solids of concrete materials.

FIGURE 2 shows the result of a computer simulation of the packing of given types of cement, sand and stone. Inside the triangle are lines drawn connecting points of identical packing densities, calculated from mean particle sizes per fraction, and proportional mix compositions.

The composition of the maximum packing density is identifiable. Theoretically this is at one point. In highway engineering it is a narrow range of compositions.

FIGURE 3 shows the correlation between packing densities obtained by the computer program and experimentally. The correlation is seen to be excellent.

FIGURE 4 shows an example of how variations of the characteristics of fresh concrete,

* good rheology, i.e. workability
* bleeding
* separation
* bad workability,

are found within the range of highest packing density for the given materials used. Generally it appears that the mix compositions of the best workability, and also those of lowest permeability and highest strength of hardened concrete, are found within the range of highest packing density.

FIGURE 5 illustrates the packing densities of mix compositions recommended by PCA and C&CA, respectively. Compared with the position of maximum packing density, it is generally found that the optimum mixture is richer in coarse aggregates than the two recommended ones.
Incidentally, it is worth noticing that addition of cement paste in concrete at a given w/c and aggregate composition will not make the concrete denser. This is because the intrinsic porosity of hardened cement paste is 25 - 35 vol. %, i.e. orders of magnitude larger than the porosity of ordinary concrete aggregates.

Concerning technology, the conventional means of measurement and quality control of rheology - in engineering terms: workability - is the slump test. In principal the slump test measures the effect of the force of gravity on the concrete sample in the steel core, when the core is lifted away.

The classic Swedish Ve/Be method also measures the rate of deformation of the slump cone, when a vibratory force is added to the gravity. This was an important progress when vibration was introduced in concrete making. Later on a "two point tester" has been introduced by Tattersall (Tattersall & Banfill, 1983) and new methods are under way.

FIGURE 6 shows the deformability, i.e. the rheology, of fresh concrete measured by the shear force vis a vis the shear rate, which gives a true picture of the actual mechanics of the deformation.

As can be seen, two different concrete mix compositions may have widely different rheology, i.e. workability when placed and compacted, but have the same slump value.

G.M. Idorn Consult A/S service to actual concrete producers has experienced that the relationship between shear stress and shear rate - the so called Bingham model - has widespread applications for the determination and optimization of the rheology of fresh concrete, especially due to the increasing use of chemical admixtures in modern concrete, and of placing technology like concrete pumping.

FIGURE 7 shows an example of optimization of the use of high range water reducing agent (HRWA - superplasticizer) in concrete. As can be seen, increasing concentrations of HRWA result in decreasing shear stress at maintained viscosity. However, at a certain concentration no further effects are obtained by increasing the addition. Thus, this concentration is the optimal.
In the research project further aspects of the illustrated rheology/workability issues are under study. These are still in progress, and approaching the stage at which application technology can be systematized.

CURING

When optimum mix design with regard to dense packing of cement, sand and stone has been specified and tested by subsequent field trial and production quality control, a good initial microstructure is ensured. It is essential that this be preserved during the subsequent curing of the concrete with accompanying heat development and dissipation.

FIGURE 8 illustrates the heat development and dissemination during the early phase of concrete curing, and the subsequent heat loss in cooler environments.

In terms of economy of energy as a resource, the initial temperature increase accelerates the hardening process, and preservation of the heat in the concrete is therefore advantageous.

Nevertheless, at a very early temperature rise the initial strength increment may be insufficient to absorb the corresponding thermal deformation stresses, and microflaws may result. Besides, at temperatures above + 60 - 70°C the hydration process changes and inflicts damage in the submicroscopic structure.

Moreover, temperature differences between the interior of concrete masses during heating and the surfaces which are exposed to cooling, may result in stresses causing initial cracking leading to later deterioration.

Long term R&D investments in Denmark, actually since the early 1950's, have resulted in advanced curing technology systems which make it possible:

* To specify concrete for given performance requirements so as to ensure optimum output of the heat of hydration while omitting risks for getting too high interior temperatures and thermal cracking caused by too high surface/interior temperature differences.
* To check the ability of a tendered concrete to comply with the requirements on available time during its hardening in the construction.

* To adjust the curing procedures in accordance with changes of the materials (cement, concrete compositions) or the environmental conditions.

The "Concrete of the Faroe Bridges" (The Danish Road Directorate, Ed. G.M. Idorn, 1986) exemplifies the advantages of this curing technology as applied in major, contemporary highway engineering. Maage and Helland (1988) describes further application in Norwegian practice.

The SHRP project C-201 has access to this knowledge and research is in progress within the project to examine basic parameters of the calculatory system vis a vis the characteristics of American cements, and also regarding the fundamental modelling of the hydration of cement.

FIGURES 9 and 10 illustrate the G.M. Idorn Consult A/S contribution through the 2nd year's progress of the project regarding curing of concrete pavements.

For each chosen variety of realistic concrete compositions, different ranges of air temperatures, initial concrete temperatures and slab thicknesses, we have applied the curing technology calculation system to determine:

- TG risk of too large Thermal Gradients within the concrete slab.
- EF risk of Early Freezing or insufficient strength development.
- HT risk of too High Temperatures within the concrete slab.

and

Satisfactory curing conditions.

FIGURE 11 is a survey of the parameters "behind" the calculations upon which the advisory leaflets are based.

A primary aim of this approach is to advise highway engineers about:

* when curing technology precautions are not required, and
* when application of the curing technology system design and site control are advisable.
We believe that this resource economy concept of the research is advantageous at an early stage of broad implementation of available curing technology systems in highway concrete construction technology. There are obvious reasons why under many practical conditions special investments in curing technology will not pay off for concrete pavements, and the investments thus not show a return. There are other combinations of conditions where the system will pay off in terms of improved durability.

For bridges, tunnels, buildings etc. in the highway system a layout for similar advisory guidelines have not been produced within the program because the conditions in most cases will require individual considerations, but the applicable calculation systems are available for the adjustments which the current progress of the research deem appropriate.

MICROSCOPIC FEATURES OF THE MICROSTRUCTURE OF CONCRETE

The following few photomicrographs have been selected from thin sections made and used during many years of application of optical microscopy in our concrete research and consulting operations.

We have deliberately chosen samples from actual concrete structures in order to illustrate the applicability of this investigation technique in highway construction practice.

In the SHRP C-201 we are supplementarily examining samples made by MRL of controlled compositions and manufacturing conditions in order to check and better understand the phenomena seen and evaluated in highway concrete.

FIGURE 12 is a concrete with a remarkably inhomogeneous cement paste consisting alternately of very condensed and very diluted regions. The overall w/c was about 0.5 as specified. However, anyone can imagine that drying shrinkage, thermal stresses, chloride ingress, alkali silica reactions, corrosion forces have easy access to inflict damage - due to the inhomogeneity of the cement paste. This concrete was virtually destroyed in the course of 10 years in a hot climate.
FIGURE 13 is from a concrete, about 50 years old, made with a blast furnace slag cement, and evidently with a microstructure with a high degree of homogeneity. The concrete was also of experienced durability.

FIGURE 14 is from a hot country concrete showing structural cracking before one year. Analyses of the circumstances substantiated that the cracking was due to thermal stresses early during the curing phase - too high interior temperatures and surface/interior temperature differences.

FIGURE 15 is a good quality microstructure in concrete made with ordinary Portland cement. The fluorescence technique has made it possible to quantify the observations so as to determine the w/c with $\pm 0.2\%$ accuracy. Concurrently, the homogeneity can be classified.

We believe it possible within the C-201 project to make an educational "Picture Book" of especially illustrative features of microstructure in concrete of various compositions and origins. Such a book could be used as guidance in highway engineering for quality control, where needed, for special investigations and for general education purposes.

The MRL research samples which are presently under examination will be considered also for such utilisation.

CONCLUSIONS

It is a deliberate, personal choice on my part in this presentation to emphasize the engineering aspects of the C-201 project. For a lifetime I have appreciated and relied upon that research institutes and universities in the USA have maintained and developed exploratory research closely associated with the basic natural sciences in the fields of cement and concrete.

Everything I have personally been associated with in applied research and concrete technology innovation has been built on pioneering American exploratory and explanatory research efforts and open handed exchange.
It is gratifying that SHRP has comprehended the value of such integrated non-goal oriented research in depth and applied efforts to achieve improved highway engineering construction technology. We appreciate being an accepted, foreign partner in such an integrated approach, with MRL, PSU as the principal contractor of the cooperative program.

There is still experimental work in progress at MRL and belonging microscopy to be done by us. Then the challenge to synthesize and combine the analytical findings must be confronted, and the essentials of the findings presented so as to appear fruitful, applicable innovations for the users of the research.

ACKNOWLEDGEMENTS

The research described herein was supported by the Strategic Highway Research Program (SHRP). SHRP is a unit of the National Research Council that was authorized by section 128 of the Surface Transportation and Uniform Relocation Assistance Act of 1987.

The emphasis on application in highway construction of the accomplishments of the SHRP C-201 research project "Microstructure of Concrete" in this presentation makes it necessary to acknowledge sincerely the indispensable background of research efforts invested at Materials Research Laboratory, The Pennsylvania State University by professor D.M. Roy, professor P. Cady and colleagues.

In professor Roy's presentation at the MRS symposium "Materials Problems of Infrastructure: How New Materials Technology Can Help Them", San Diego on 27 April 1989, the overall research plan and the research at MRL as the principal contractor is duly reported.

In the present paper the complementary phases of the C-201 project which have been undertaken by the consulting company G.M. Idorn Consult A/S, Denmark, has especially been reviewed on the basis of experimental work and analyses by P.J. Andersen, K.T. Andersen and N. Thaulow, and partly originating from consulting services and research beyond the C-201 project. The aim of this approach is to illustrate that general knowledge is available for integration with
the research accomplishments in SHRP C-201 in the remaining phase of the project and for further projects of implementation in highway engineering practice.
FIGURES
An overall qualitative survey of the phases of concrete manufacture and the belonging aspects of concrete technology. The SHRP C-201 research project in particular explores the nature of fresh concrete and the issues of curing with the aim of developing improved production control systems for highway engineering.
FIGURE 2
Ternary display of the degree of packing for combinations of cement, sand and coarse aggregates.

FIGURE 3
Correlation between packing densities determined experimentally and theoretically, respectively.
Concrete compositions of cement, sand and stone, and a w/c = 0.5 which correlates the packing of the solids to the rheology of the concrete, classified with regard to workability, bleeding and separation.

Packing densities of mix compositions recommended by PCA and C&CA, respectively. Compared with the position of maximum packing density, optimum mix composition is usually richer in coarse aggregates than the two which are recommended.
Bingham model:

![Graph showing the Bingham model for the deformation of fresh concrete under impact of gravity and external load.]

**FIGURE 6**
The Bingham model for the deformation of fresh concrete under impact of gravity and external load.

**EFFECT OF SUPERPLASTICIZER**

![Graph showing the effect of superplasticizer on shear stress and shear rate.]

**FIGURE 7**
Increasing the addition of a High Water Reducing Agency (superplasticizer) to fresh concrete results in decreased shear stress at maintained viscosity, i.e. less vibratory power required for placement of the fresh concrete. At a certain level of concentration no further decrease of the shear stress is obtained. No benefit is thus gained by exceeding this dosage of HWRA.
FIGURE 8

The development and dissemination of heat caused by the hydration of cement in concrete during the curing phase.
## FIGURE 9

Diagram for preliminary evaluation of the need for special curing precautions at concrete pavement constructions. 600 lbs Ordinary Portland cement/cu.yd.
### Table

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<th>deg. F</th>
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<th>inches</th>
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<td></td>
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<tr>
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<td>*</td>
<td>HT</td>
<td>TG/HT</td>
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<td></td>
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<td>*</td>
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<td>*</td>
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<td>HT</td>
<td>HT</td>
<td>HT</td>
<td>HT</td>
<td>8</td>
<td></td>
</tr>
</tbody>
</table>

### Notes:

- * = satisfactory curing conditions
- TG = risk of too large Thermal Gradients within the concrete slab
- EF = risk of Early Freezing
- HT = risk of too High Temperatures within the concrete slab

---

**FIGURE 10**

Diagram for preliminary evaluation of the need for special curing precautions at concrete pavement constructions. 650 lbs Rapid Portland cement/cub.yd.

VTI RAPPORT 352A
## INPUT PARAMETERS USED IN THE MODELLING PROGRAM

- **Cement type**: type I, III and V
- **Mix compositions**: 6 mixes, cement content: 555-675 lbs/cu.yd.
- **Concrete temperature**: 50 - 100°F (10 - 37°C)
- **Air temperature**: 0 - 100°F (-18 - 37°C)
- **Pavement thickness**: 8 - 20 inches (200 - 500mm)
- **Base temperature**: 32 - 90°F (0 - 32°C)
- **Cover type**: Below 32°F: Straw. Above 32°F: Burlap

## CONCRETE MIX COMPOSITIONS CHOSEN FOR PROGRAM CALCULATIONS

1. Slump is selected to 2 inches (50mm).
2. The air content is selected to 5 - 7% by vol of concrete.
3. Water content: The water/cement ratio is chosen to 0.45.

<table>
<thead>
<tr>
<th>Mixture</th>
<th>Cement lbs/cu.yd.</th>
<th>Water Max. size</th>
<th>Aggr. w/c</th>
<th>Air vol %</th>
<th>Air vol %</th>
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<td>1</td>
<td>675</td>
<td>305</td>
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<td>0.45</td>
<td>7.0</td>
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<td>655</td>
<td>295</td>
<td>1/2</td>
<td>0.45</td>
<td>6.0</td>
</tr>
<tr>
<td>3</td>
<td>620</td>
<td>280</td>
<td>3/4</td>
<td>0.45</td>
<td>5.5</td>
</tr>
<tr>
<td>4</td>
<td>600</td>
<td>270</td>
<td>1</td>
<td>0.45</td>
<td>5.0</td>
</tr>
<tr>
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<td>575</td>
<td>260</td>
<td>1 1/4</td>
<td>0.45</td>
<td>5.0</td>
</tr>
<tr>
<td>6</td>
<td>555</td>
<td>250</td>
<td>1 1/2</td>
<td>0.45</td>
<td>5.0</td>
</tr>
</tbody>
</table>

**FIGURE 11**

Parameters on which the calculations for Figures 9 and 10 are based.
FIGURE 12

Thin section of concrete with exceptionally heterogeneous cement paste, consisting of compact, partly hydrated patches alternating with dilute areas. Overall w/c of the fresh concrete = 0.5.

FIGURE 13

Thin section of 50 year old concrete with homogeneously hydrated cement paste, made with a blast furnace slag cement.
FIGURE 14
Thin section of concrete from a hot country, having developed structural cracking before one year of service life. Based on thorough analyses of the circumstances and examination of the concrete it was inferred that thermal stresses had caused the cracking during the curing of the concrete.

FIGURE 15
Thin section showing good quality microstructure in concrete made with Ordinary Portland cement. w/c has been determined to 0.45 ±0.02 by assessment of the colour image of the cement paste in UV illumination.
REFERENCES AND SOURCE MATERIAL


Chloride Removal and Corrosion Protection of Reinforced Concrete

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Technical Director
Norsk Overflate Teknikk A/S (NOT)
Norway
ABSTRACT

The author and his co-inventors have developed an electro-chemical technique for removing chlorides from salt-contaminated concrete. The method, which is commercially available, and which is in use in several countries, is briefly described. The method uses low-voltage DC current to cause migration of chloride ions out of the concrete and into an external electrolyte which is then removed. Treatment times for most structures vary from between 2 and 8 weeks.

The paper touches upon problems associated with electro-chemical treatment such as monitoring and documenting their success, hydrogen embrittlement, and the question of alkali-aggregate reactions. A novel lead reference electrode is briefly described.

John B. Miller
Norsk Overflate Teknikk A/S
CHLORIDE REMOVAL AND CORROSION PROTECTION OF RE-INFORCED CONCRETE.
John B. Miller
Technical Director
N.O.T. (Norsk Overflate Teknikk A/S)

1. INTRODUCTION

The chloride ion is one which is destructive to re-inforced concrete by virtue of its ability to penetrate and disrupt the protective oxide film formed in alkaline concrete on the surface of re-inforcement steel, thus exposing the steel to corrosion. This corrosion will take place when all of the following are fulfilled:

a) sufficient moisture is present
b) sufficient electrical conductivity exists
c) sufficient oxygen is present
d) sufficient numbers of chloride ions are present.

If any one of these prerequisites is absent or restricted, the corrosion process will stop, or be reduced.

Attempts have been made in the past to hinder corrosion by changing one or more of these conditions:

- Drying out concrete to a moisture level of below 45 % RH, though difficult to do and maintain, will effectively stop corrosion.

- Additions of microsilica to the concrete mix can decrease electrical conductivity to an extent where corrosion, for all practical purposes, will not take place, though obviously this can only be applied to new concrete.

- Reduction of oxygen availability has been attempted by the application of coatings, or by saturating the concrete with a liquid such as oil or water. Saturation by oil or water can be effective, but only where the structure is totally immersed in the liquid.
To the author's knowledge, no coating attempt has been successful except on a small laboratory scale.

The purpose of this paper is to present the latest developments concerning the remaining possibility - the reduction of the numbers of chlorine ions present.

2. ELECTRO-MIGRATION OF CHLORIDE IONS

Electrochemical desalination, as developed by Mr. Øystein Vennesland, Mr. Ole Arnfinn Opsal, and the author, is being used (along with the allied methods of electro-chemical re-alkalinisation and dehumidification), at the time of writing, on a commercial basis to treat concrete structures in Norway, Sweden, Holland, Denmark and Germany. Other countries known to follow soon will be the U.K., U.S.A., Belgium, Switzerland, Australia and Hong Kong.

Desalination is a basically simple method. The principle is well known: the re-inforcement in a concrete structure is used as negative electrodes; a positive electrode net is mounted externally close to the concrete surface in an electrolyte; the two electrodes are then connected to a low voltage DC power supply, whereupon the negative chloride ions are attracted to the external electrode and are thus drawn out of the concrete. After a time, the chloride ions will have accumulated in the external electrolyte, and can be removed by removing the latter.

Fig.1 : Principle of Desalination
Previous attempts to desalinate concrete as undertaken by Slater and others, though successful in removing chloride, failed due to degradation of the concrete. These failures were due to the high voltages and high currents used, which lead to extreme heating, cracking and increases in porosity. In the method of desalination presented here, voltages and currents are very moderate, and do not give the disruptive effects noted earlier.

In practice, access to the structure's reinforcement for establishing electrical connection is gained by chiselling, for example, in spalled areas. The external electrode net, which is installed on wooden battens bolted to the concrete, is either a small dimension standard reinforcement net, or platinised titanium mesh, depending on circumstances. The electrolyte is either lime-water, or a cellulose fibre material containing lime-water which is sprayed onto the surface to completely cover the net. The latter is the most used (see fig 2), since the former, though more effective, requires the installation of some form of watertight container since it is a liquid. Sealing the edges of the container against the concrete surface can be difficult, and this technique is therefore not so often used.
Voltages are in the range 6 to 24 V DC, though in special cases, such as those of high cover, or very dense concrete, voltages up to 40 V DC may be used. Current consumption varies from about 0.5 A up to 3 A/m² of concrete surface. Preferred working values are 0.5 - 1.0 A/m², which is regulated by adjusting the DC voltage.

For small jobs, single phase AC/DC convertors are used, which limit available amperage, and therefore area under treatment, to about 70 A and 100 m² respectively. For larger jobs, 3-phase AC/DC convertors are necessary, the standard model now in use being able to tackle 400 A or up to 800 m² of concrete surface. Time necessary for most desalination works depends on the voltage (and, therefore, amperage) used, the chloride concentration in the concrete and its distribution, the configuration and amount of reinforcement present, and, to a great extent, the density of the concrete. The time actually taken to reduce chlorides to acceptable levels has varied from 2 weeks in porous concrete, up to 8 weeks in dense concrete. Generally speaking, for concrete qualities in the common range of C 25 to C 45, 2 to 4 weeks will be required whereas qualities C 45 to C 65 will require 4 to 8 weeks. Chloride extraction in higher qualities, though perfectly possible, has so far been judged impractical due to the time involved.

3. CHLORIDE LEVELS

Chlorides in concrete can be due either to chloride accelerator added during mixing, or to contamination by brines seeping into concrete over a period of time. Examples of the latter can be found in marine structures (seawater), roads and bridges (de-icing salts), cooling plants (brine leakages) and, of course, salt bins and silos.

Whether the chloride is cast-in or not, practice has proved that chloride levels can be extremely variable in all three directions in concrete. It is thus difficult to ascertain exactly how much chloride a particular concrete structure contains. In the author’s experience, the best estimates are gained from a few well placed fairly large diameter cores (70 - 120 mm) which
are sliced into 10 mm discs, each disc then being crushed and analysed.

The collection and analysis of dust samples from small-diameter hammer drilling does not give a dependable estimate unless the number of samples is high.

Whatever the chloride level may be (values as high as 15% of the cement content of concrete have been found), the object of desalination is to reduce the level to below that which is considered critical.

In fresh concrete containing ordinary reinforcement (as opposed to tensioned steel) most countries accept that a chloride content of up to 0.4% of the cement content can be tolerated. This then is the objective of desalination in practice.

4. REPASSIVATION

Many people are correctly of the opinion that the chloride content, which can be tolerated in fresh concrete, does not apply to concrete in which chloride corrosion has already occurred. In other words, to stop corrosion, chloride levels must be reduced much further than to 0.4%, at least at the steel surface.

In electro-chemical desalination, this is exactly what happens. Chloride ions, being negative, are strongly repulsed by the negatively charged re-bars in the concrete. The steel surface quickly becomes depleted of chloride ions. Analysis of samples taken close to re-bars after desalination show very low chloride levels.

Simultaneously with chloride ion migration, another process takes place. At the re-bar surface, OH− ions are generated as a result of a continuous electrode reaction. These ions quickly raise the pH-value at the steel surface and in the surrounding concrete to values very close to 14. Under these conditions, i.e. chloride free steel surfaces in a chloride free, highly alkaline environment, strong repassi-
vation of the steel surfaces occur upon switching off the polarising current. The re-bars can therefore be regarded as new bars in fresh concrete, and the critical chloride content applying after desalination can be taken as that applying to fresh concrete.

That this is so in practice is borne out by analysis as mentioned above, and also by measurements of electro-chemical potentials before and after treatment.

5. ELECTRO-CHEMICAL POTENTIALS

ECP-measurements can be made on concrete containing chlorides using any conventional reference cell to reveal areas where corrosion is underway. The measurements can be repeated after desalination is finished and they will confirm the passive state of the steel. Such measurements cannot be made immediately after desalination, since the polarisation induced by the current takes some time to dissipate. It is best to take readings as a function of time, when it will be found that the values tend to an asymptotic passivation value. For the purposes of monitoring ECP's, it is better to use embedded electrodes. For this purpose, reference electrodes made of pure lead have been found to be extremely stable over long periods of time (see fig.3).

![Fig.3: Sketch of lead-electrode installation](image)

Lead electrodes already have a problem free service record of about 5 years, providing they are correctly embedded in a suitable cement paste. They have also the advantage of being cheap, fast, and give dependable reproducible readings.
For lead, the following ECP-limits can be used in partially saturated concrete:

Protected Steel: > + 515 mV
Propensity to corrode: + 515 mV to + 415 mV
Corroding Steel: < + 415 mV

Since embedded electrodes only allow measurements at fixed points, these points must be chosen intelligently.

6. HYDROGEN EMBRITTLEMENT

One of the worries in using electro-chemical method on structures containing stressed steel, is that of hydrogen embrittlement due to the possible production of nascent hydrogen at the steel surface. This question has not yet been fully investigated in the case of desalination. However, the author considers that, in view of the relatively short time the current is applied, the danger is far less than for cathodic protection systems where the current is applied permanently. In addition, it would appear that the production of hydrogen during desalination can, in large part, be avoided by monitoring the potential developed at the steel surface using a reference half-cell (during intervals with no current flowing). Plotting these potentials as a function of time yields a curve which gives warning as to when the threshold potential at which hydrogen starts to form is approached (see fig.4). The current can then be switched off and the structure allowed to rest, or the current can be reversed for a short time to restore the potential of the steel at a value (about -600 mV) well away from the hydrogen value.
This technique of alternatively applying current for desalination, and switching off to allow monitoring, interspersed with current reversals for depolarisation, though still under development, shows much promise. A prototype piece of electrical equipment to perform this task automatically is under test at the author's laboratory.

7. ALKALI-AGGREGATE REACTION

It is known that impressed current cathodic protection systems can lead to alkali-aggregate reaction problems in concrete containing certain types of reactive aggregates. The reason for this is generally thought to be the concentration of sodium and potassium ions around the negatively charged rebar. Since, in cathodic protection systems the current runs permanently, there will never be any alleviation of this concentration. Also, several cathodic protection systems use anodes which are embedded in a concrete screed laid on the concrete surface to be treated. This screed worsens the situation by virtue of its content of alkali metal ions which will migrate in the concrete towards the steel being protected.

In the case of desalination, there is no external source of alkali metal ions. Nor does the current run permanently, which means that after switching off and dismantling the installation, any ionic concentrations will quickly diffuse away until eventually the alkali metal
ion content has equalised throughout the concrete mass. Desalination, therefore, should not worsen the situation, save perhaps temporarily, as regards the possibility of alkali aggregate reactions. Work to further illuminate this question is underway in Norway and elsewhere.

To those who wish to study desalination themselves, a word of warning. The author has experienced that tests done by others on test pieces containing very high salt levels (6-12 % NaCl) have resulted in micro-cracking which was thought to have been induced by the current. Further investigation showed that the micro-cracking was, in fact, due to alkali-aggregate reaction provoked by the high alkali metal content introduced by the salt added. To suspect the current was understandable, since the cracking took place in conjunction with aggregates which showed no previous history of alkali-aggregate reaction.

8. CONCLUSION

Desalination using the described techniques is today a commercially available method for the treatment of salt-contaminated structures. The method is the only one, with the possible exception of cathodic protection systems, which can effectively eliminate the danger of chloride corrosion. The method is also cheaper to apply than both cathodic protection systems and the mechanical removal of chloride infected concrete. It is also non-destructive, and has the immense advantage that the work done, and its effect, can be quickly and easily documented using chloride analysis and ECP-measurements.

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Chloride-Binding in Cement and the CL/OH-Ratio of the Pore Solution

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Chloride binding in cement and the Cl/OH-ratio of the pore solution

J. Tritthart

Abstract:

Although it is generally recognized that only the free chloride can damage the steel in concrete it is the total chloride content which is normally used for the assessment for the corrosion risk. Thus it is worth while to have precise information on chloride concentration in the pore solution of hardened cement paste as well as of the factors influencing chloride binding. The investigations showed that the residual chloride concentration in the pore solution of samples prepared with the addition of chloride, made with the same cement and with 1% total chloride content varied on dependency of the w/c-ratio (between 0.4 and 1.0) and the used chloride compound (NaCl or CaCl₂) up to a factor of >8. Therefore the total chloride content alone can not be a reliable indicator. Results of different authors show that the Cl/OH-ratio of the pore water is the significant value. Although the Cl/OH-ratio allows without any doubt a much better assessment than the total chloride content alone but it is also not the indicator which gives an absolutely sure information. This because the results showed that with increasing w/c-ratio both the Cl- and OH-concentration in the pore water of samples with otherwise identical composition decrease continuously, however, at different rates. The Cl/OH-ratio was therefore the higher the lower the w/c-ratio. That would mean that the corrosion risk increases with decreasing w/c-ratio. This is in contradiction to all practical experiences probably because of the influence of other corrosion relevant factors such as oxygen diffusion, moisture, electrical resistivity etc.
Chloride-Binding in Cement and the CL/OH-Ratio of the Pore Solution

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1. Introduction - Chapter 1

The assessment of corrosion hazard, to which steel of reinforced or pre-stressed concrete is exposed, is generally based on the total chloride content in concrete. The latter is related to the cement content, as cement is capable of binding and thus rendering harmless small quantities of chloride. However, there is no generally accepted value about the amount of harmless chloride. In Europe a threshold value of 0.4 M% Cl - related to the cement mass - is often used. Yet in recent years this value has become more and more controversial. This mainly because practical experience has shown that even at much higher chloride contents corrosion sometimes have not occurred. Moreover the threshold values reported in literature differ widely.

In the following, first some threshold values from literature shall be compared. Then investigations on the composition of the pore solution of hardened cement pastes containing chloride are described. These experiments have been performed because it is only the unbound chloride which can interact with and possibly destroy the protective layer on the steel surface. Thus it is worthwhile to have precise information on the chloride concentration in the pore solution of hardened cement paste after binding as well as on the factors influencing chloride binding.

2. Threshold values of chloride in alkaline solutions and concrete in literature - Chapter 2

Table 1 shows some threshold values, which have been determined by electrochemical methods. Table 2 contains threshold values obtained from practical experience or derived from experiments performed under practical conditions as well as values which are not defined in detail.

It can be seen that the threshold values vary considerably. It is an interesting fact and perhaps one possible explanation for this phenomenon that the harmless chloride concentration is dependent on the pH-value of the solution as suggested for the first time by Hausmann /4/. As only the chloride ions cause corrosion, and the OH-ions, on the other hand, have a passivating effect, it appears highly plausible that the more chloride is harmless the more OH ions are present, i.e. the more strongly.
Table 1: Chloride threshold values in concrete and alkaline solutions (determined by electrochemical methods)

<table>
<thead>
<tr>
<th>Harmless chloride in % of cement mass (total chloride)</th>
<th>Concrete</th>
<th>Alkaline solutions</th>
<th>Author</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt;0.5% CaCl₂ (=&lt;0.32 % Cl)</td>
<td>&quot;after 40 hours&quot;</td>
<td>--</td>
<td>H. KAESCHE (1959) /1/</td>
</tr>
<tr>
<td>&gt;0.9% CaCl₂ (=&gt;0.57 % Cl)</td>
<td>no indication</td>
<td>&lt;35 mg CaCl₂/l (=&lt;22 mg Cl/l)</td>
<td>A. BÄUMEL (1959) /2/</td>
</tr>
<tr>
<td>max. 3.1% CaCl₂ (=&gt;2.0 % Cl) for PC 375</td>
<td>&quot;after longer hardening in humid wood shavings&quot;</td>
<td>&lt;35 mg CaCl₂/l (=&lt;22 mg Cl/l)</td>
<td>A. BÄUMEL H. J. ENGELL (1959) /3/</td>
</tr>
<tr>
<td></td>
<td>in pH range 11.6-13.2: Cl/OH ≤ 0.6</td>
<td>NaOH or Ca(OH)₂</td>
<td>D. A. HAUS-MANN (1967) /4/</td>
</tr>
<tr>
<td></td>
<td>in pH range 11.5-13.5: pH=nlogCl+ K</td>
<td>NaOH and Ca(OH)₂</td>
<td>V. K. GOUDA (1970) /5/</td>
</tr>
<tr>
<td></td>
<td>&gt;0.035 (&lt;0.040 m% Cl (= &gt; 350 &lt;400 mg Cl/l)</td>
<td>sat. Ca(OH)₂</td>
<td>H. POLSTER J. KEUCHER (1974) /6/</td>
</tr>
<tr>
<td></td>
<td>0.03 mol NaCl/l; presence of air 1.0 mol NaCl/l absence of oxygen</td>
<td>sat. Ca(OH)₂</td>
<td>H. BERMAN (1975) /7/</td>
</tr>
<tr>
<td>~0.4% -~0.8% Cl</td>
<td>28 days</td>
<td>--</td>
<td>C. E. LOCKE A. SIMAN (1980) /8/</td>
</tr>
<tr>
<td>&gt;1% &lt;1.5% NaCl at OPC (=&gt;0.61% &lt;0.91% Cl)</td>
<td>between 1 and 7 weeks</td>
<td>&gt;190 &lt;373 mg Cl/l</td>
<td>P. RECHBERGER (1980; 1982)/9/10/</td>
</tr>
</tbody>
</table>

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Table 2: Empirically determined and not further specified chloride threshold values

<table>
<thead>
<tr>
<th>Threshold value (total chloride)</th>
<th>Author</th>
</tr>
</thead>
<tbody>
<tr>
<td>Related to concrete mass</td>
<td></td>
</tr>
<tr>
<td>Related to cement mass</td>
<td></td>
</tr>
<tr>
<td>2% CaCl₂ (≈1.3% Cl) at a concrete porosity of max. 9.5 vol% and a min. cover of 1.5 cm</td>
<td>J.TOMEK and F.VAVRIN (1961) /11/</td>
</tr>
<tr>
<td>0.2% Cl</td>
<td>K.C. CLEAR and R.E.HAY (1973) /12/</td>
</tr>
<tr>
<td>approx. 11b Cl/yd³ (=approx 0.59 kg/Cl/m³; = approx. 0.02%)</td>
<td>R.F. STRAF-TULL et al (1975) /13/</td>
</tr>
<tr>
<td>500 ppm Cl</td>
<td>C.F.STEWART (1975) /14/</td>
</tr>
<tr>
<td>0.028% Cl at 330 kg PC/m³ concrete and a water soluble chloride content of 75%; 0.051% Cl at 390 kg PC/m³ concrete and a water soluble chloride content of 50%;</td>
<td>OECD-Road Research Group (1976) /15/</td>
</tr>
<tr>
<td>2% CaCl₂.2H₂O (≈1% Cl; at dense concrete; otherwise less)</td>
<td>working group &quot;Monoliet&quot; (1976) /16/</td>
</tr>
<tr>
<td>0.4% Cl</td>
<td>L.J.EVERETT et al. (1980) /17/</td>
</tr>
<tr>
<td>1.0% to 1.5% Cl (at dense concrete; otherwise less)</td>
<td>W.LUKAS (1980) /18/</td>
</tr>
<tr>
<td>0.035% Cl at concrete with a unit weight of 2300 kg/m³ and 400 kg PC/m³ concrete</td>
<td>0.2% Cl</td>
</tr>
</tbody>
</table>
alkaline the medium. According to Hausmann, there is no danger of corrosion up to a ratio of molar concentration of Cl/OH - approx. 0.6. According to this, there is a linear correlation between logarithm of chloride-threshold concentration and pH-value of the solution. This interaction has been confirmed by Gouda, however, with the reservation that the Cl/OH-ratio is somewhat below 0.6 and that the linearity only exists up to a pH-value of 13.5, and that the chloride concentration hardly increases any further at higher pH-values /5/.

In more recent publications there is repeated mention that the critical amount of chloride can vary from case to case and that the results obtained in aqueous solutions are not applicable with the protective effect being higher in mortar and concrete. According to C.L.Page et al /20/, this is, inter alia, due to the fact that Ca(OH)₂ is present in mortar in solid form, which quickly replenishes the OH-ions used up after the dissolution of iron as a result of origination of ferrous hydroxide. The solid Ca(OH)₂ is therefore counteracting acidification at the corrosion pit, and it facilitates repassivation or makes it possible at all. Moreover chloride diffusion to the corrosion pit is hampered by dense mortar due to its high diffusion resistance /20/21/. T. Yonezawa et al. suggest that the formation of voids at the steel/mortar-interface (e.g. an area with sedimentation water) is essential for the onset of active corrosion /22/. Furthermore, it is generally recognized that the rate of corrosion is strongly influenced by the oxygen diffusion through the concrete cover. The oxygen flux varies considerably depending on the concrete composition and moisture as does the electric resistance of the concrete, which is also of importance.

There are also doubts in various publications contained in Table 2 as to whether a uniform threshold value can be given for the situation in practice without taking into account other boundary conditions. That all shows that the corrosion hazard depends not only on the chloride content but also quite essentially on the OH concentration of the pore electrolyte (the pH value) as well as on the diffusion of the ions, the oxygen flux, i.e. the concrete quality in general (w/c ratio, compaction; storage conditions) and the microstructure of the cement paste in particular etc., thus on a whole series of different factors.

3. Chloride and hydroxide concentration in the pore water of cement pastes prepared with the addition of chloride - Chapter 3

3.1 Test method

In literature various potentially suitable methods have been described to determine the chloride concentration of pore water in hardened cement. However, the methods have led of differing results /23/24/25/. Therefore the methods had to
be examined for their suitability. This was done by specially prepared slabs of hardened cement paste of only one cm in thickness. The hardened slabs were stored in a solution of constant chloride concentration for up to one year. After that time the composition of the pore water corresponded to that of the storage solution due to equalization of diffusion. Further investigation showed that correct results can be obtained only by the expression technique (for details see /26/). Hence all further experiments were performed according to this method. The work was carried out within the COST 502 program of the EC with the cooperation of the Danish Corrosion Center (Mrs. C.M. Hannson) and the Swedish Cement & Concrete Research Institute (Mrs. K. Byfors).

3.2 Preparation of samples

The chloride was dissolved in the mixing water before mixing. Plastic bags were placed into cylindrical containers and the exposed top ends of the bags pulled down over the walls of the cylinders. The fresh pastes, which had been mixed in a glass with a spoon, were then filled into containers supported by frames that could be turned. After filling, the containers were covered with a metal plate to which a rubber seal was attached, and the plate was screwed tightly onto the containers. The samples were turned for one day, afterwards the plastic bags with the samples were removed from the cylinders, the bags tied up and then placed in a second plastic bag and stored at 20°C (68°F) until required for testing.

3.3 Test Results

The majority of the tests were not restricted to one type of cement but performed on several, sometimes up to 17 different kinds of cement from Austria, Sweden, Denmark and FRG. The correlations shown in the diagram were obtained with Austrian Portland cement "PZ 375". The type of cement contain no mineral additives (except for gypsum). The determination of the clinker composition according to Bogue showed a C₃A content of 10.6 M%.

3.3.1 Influence of the total chloride content and the type of added chloride salt

Figure 1 shows that the residual chloride concentration (sample age about 10 weeks) in the pore solution of the hardened cement increases continuously as the total chloride content rises. The addition of NaCl results in higher chloride and hydroxide concentrations than when calcium chloride is added. This is in agreement with results of other authors /27/28/29/30/. The OH-concentration, on the other hand, increases with the use of NaCl and decreases with CaCl₂.

Other results showed that different chloride compounds (NaCl, HCl, CaCl₂, MgCl₂) produce different residual chloride concentrations in samples with otherwise identical compositions (same cement, same w/c-ratio and same total chloride content) only, if they influence the hydroxide concentration differently /31/.
3.3.2 Influence of the w/c-ratio

Figure 2 demonstrates that both the chloride and the hydroxide concentration in samples with otherwise identical composition decreases continuously with increasing w/c-ratio, however with different gradients. At a w/c-ratio of 0.40 the concentration of the chloride ions was clearly higher than that of OH-ions, whereas at a w/c-ratio of 1.0 the situation was reversed.

Fig. 2: Chloride- and Hydroxide-concentration of the pore water of samples with different w/c-ratio
3.3.3 The Cl/OH-ratio

The ratio between molar concentration of chloride and hydroxide ions of the pore solution becomes larger as the total chloride content increases (left diagram in figure 3). At a given total chloride content, however, the Cl/OH-ratio depends on the w/c-ratio and decreases with increasing w/c-ratio (right diagram). The differences between samples containing NaCl and CaCl₂ are not very marked here, as the addition of CaCl₂ reduces both the chloride and the hydroxide concentration of the pore water so that the quotient of Cl/OH is not strongly influenced by the type of added chloride salt.

The criterium established by Hausmann of Cl/OH = approx. 0.6 was attained in the different Portland cement types tested not before a total chloride content of 1 to 1.5 m% Cl, yet was exceeded in sulfat-resistant cement already at values smaller than 0.6 m% Cl. This is probably one of the reasons why in practice corrosion frequently has not occurred in well-prepared concrete in spite of relatively high total chloride contents /11/16/18. Unfortunately, it is not possible yet to apply the results obtained from cement pastes, which have not been exposed to environmental influences, to practice as each concrete emits OH-ions into rainwater and, due to carbonatization, the bound chloride is at least partly dissolved again in the boundary zones.
4. Binding of subsequently added chloride - Chapter 4

4.1 Preparation and analysis of samples

To examine the binding of chloride added subsequently by diffusion, chloride-free cement slabs of 1 cm thickness and w/c-ratios of 0.50 and 0.70 were used, whose production is described elsewhere /26/. The slabs were immersed in the chloride-free storing solution for several days in order to fill the shrink pores with water prior to the addition of chloride. As storage solutions saturated Ca(OH)$_2$ solution (pH-value: 12.5), 0.1 NaOH solution (pH-value: 13.0) and 0.5 NaOH solution (pH-value: 13.7) each with five different chloride concentrations were used. From time to time the chloride concentration of the storage solution was measured and used-up chloride replenished. In that way the chloride concentrations were kept more or less constant over the whole test period. Although the chloride content of the samples and the storage solutions changed only insignificantly between 150 and 200 days the slabs were retained in the solutions for a total of 380 days in order to be sure that a state of equilibrium had been reached and that the composition of porewater had become equal to that of the storage solution. Only then the final determinations of the total chloride content of the hardened cement pastes were carried out.

4.2 Test results

As can be seen from Fig.4, the higher the chloride concentration of the storage solution the greater the amount of chloride is taken up of the hardened cement. The pH-value of the storage solution plays a major role as well; the higher the pH-value is, the less chloride will be bound. The influence of the w/c-ratio on the other hand, is rather insignificant - samples with a w/c-ratio of 0.70 take up slightly more chloride in relation to the cement mass than comparable samples with a w/c-ratio of 0.50.

![Diagram](image)

Fig. 4: Chloride uptake of cement pastes immersed in chloride solutions
5. Discussion of results - Chapter 5

The result shows that chloride is not bound in absolute quantities but rather in dependence on a distribution equilibrium which is established between solids and the liquid phase. The state of equilibrium and thus the residual chloride concentration, however, varies from case to case and depends on the composition of the cement as well as on the pore solution chemistry, particularly on its OH-concentration (the pH-value). With regard to the binding of chloride there is a strong dependence on the w/c-ratio only, if the samples are prepared with the addition of chloride but not if the chloride diffuses into the hardened paste. The pore water composition (its alkalinity) depends on the w/c-ratio and therefore there exists an influence of the "w/c-ratio" on chloride binding when the samples are prepared with the addition of chloride. During storing of chloride-free prepared samples with different w/c-ratios in the same chloride solution, however their initial differences in the composition of the pore solutions are equalized by diffusion so that about the same quantity of chloride is taken up regardless of the w/c-ratio (The influence on the pH-value of storage solution is shown in Fig.4).

The dependence of the OH-concentration of the pore solution on the w/c-ratio can be explained by the shift in the absorption/desorption balance with rising water content (rising w/c-ratio). The reason for the difference in pH-values of the pore solutions between samples containing NaCl and CaCl₂ is that at the high pH-values of the pore solution calcium is insoluble and precipitates as solid Ca(OH)₂ under reduction of the OH-concentration whereas sodium remains in solution (for details see /31/).

6. Conclusions - Chapter 6

The studies have shown that with regard to corrosion risk there is no definite critical amount of chloride (total chloride) that could be applied to all cases in practice. This can be attributed to the fact that the residual chloride concentration of the pore solution, which is relevant for corrosion, may differ considerably in samples taken from the same cement and with the same total chloride content. It varied between samples with 1% total chloride content in dependence on the w/c-ratio (between 0.4 and 1.0) and the used chloride compound (NaCl or CaCl₂) up to a factor of > 8. Therefore the total chloride content alone cannot be a reliable indicator.

Results of different authors on the other hand show that the Cl/OH-ratio of the pore water is the significant value. Although the Cl/OH-ratio allows undoubtedly a much better assessment than the total chloride content alone, it is not the indicator either which provides an absolutely reliable information. That because the results showed that, with increasing w/c-rat-
ratio, both the Cl- and OH-concentration in the pore water of samples with otherwise identical composition decreased continuously, however, at different rates. The Cl/OH-ratio was therefore the higher the lower the w/c-ratio. That would mean that the corrosion risk increases with decreasing w/c-ratio. This is in contradiction to all practical experiences probably because of the influence of other corrosion relevant factors such as oxygen diffusion, moisture, electrical resistivity etc.

Unfortunately, the influence of these parameters is not known sufficiently, moreover, variations of moisture already produce changes. The oxygen transport from outside to the steel surface is strongly influenced by the pore structure as well as by the water content of the concrete, and it is many times higher in air-dried concrete than in water-saturated one, in which, due to the lack of oxygen, corrosion is rather unlikely to occur. Moreover, conditions vary from place to place due to unsymmetrical drying and depend on the cover thickness as well as on any inhomogeneities and voids (cracks). Thus it is altogether impossible - even if it were technically feasible, for instance, to measure the oxygen flux at the construction site - to assess quantitatively the correlations applicable to a specific case in practice.

Hence the chloride content of concrete will continue to be the most important parameter for the assessment of the corrosion risk. However, the total chloride content alone does not provide a sufficient basis for this assessment. The present approach to decide on the necessary repairs of reinforced or prestressed concrete bridges on the basis of the total chloride content and an expert opinion is unsatisfactory both in terms of safety and economy. However, in order to find the best possible compromise between safety and economy, a much more precise assessment and thus a better understanding of the influences and mechanisms of corrosion will be required. Hence investigations into the Cl/OH-ratio in the concrete under practical storage conditions or into the microstructure of hardened cement, for instance, seem to be indispensable and will be the objective of further studies.
References

31. J. Tritthart, "Chloride binding in Cement - Part 2" accepted for publication in Cement & Concrete Research
Protection of Reinforcing Steel against Corrosion upon Chloride Impact

Gerhard Hartl
Univ-Doz
Dipl-Ing
Dr techn
Zementforschungsinstitut
Austria
Corrosion Protection of Reinforcement against Chloride Impact

Concrete structures and 64 sample concrete slabs were examined for the resistance of their concrete surfaces and for chloride penetration, upon de-icing salt impact, and 188 reinforced beams with State-II fissures were examined for corrosion of the steel reinforcement after several years of exposure, with and without chloride impact.

The results showed that fissures, of the width usual in reinforced concrete, did not affect the stability, even upon chloride impact, if concrete quality and thickness of the concrete layer were adequate.

When implementing such structures according to the provisions contained in Austrian standard UNORM B 4200, section 10, these must be dimensioned sufficiently.

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PROTECTION OF REINFORCING STEEL AGAINST CORROSION UPON CHLORIDE IMPACT

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Zementforschungsinstitut, Vienna, Austria

1. INTRODUCTION

Around the mid-1970s, bridges began to be inspected more thoroughly, both in Austria and its neighbouring countries. Some five years later, the first reports on these analyses appeared in the literature. To some extent, they contained very negative results, which caused major concern among scientists.

In 1980, the "Zementforschungsinstitut" (= Cement Research Institute) in Vienna received a grant for the research project "Corrosion Protection of Reinforcing Steel in Concrete against De-icing Salt Impact", sponsored by the Federal Ministry of Public Works and Technology. The project was implemented in cooperation with the Institute for Construction Materials and Material Testing at the Technical University of Innsbruck. The investigations and their results are described in detail in /1/. A summary of the findings is given below.

2. ANALYSIS OF TEST CORES FROM A CONCRETE HIGHWAY PAVEMENT

30 test cores (diameter 5 cm), including portions of the reinforcing steel bars, were removed from a reinforced concrete pavement, which had been built in 1958 and was closed for repairs after having been exposed to severe chloride attack during its entire service life of 20 years, in order to determine, as quickly as possible, what chloride penetration could be expected in concrete that complied with the standard (Austrian Standard OENORM B 4200 Part 10 /2/).

Fig. 1 shows the mean chloride-content values, as well as the maximum and minimum values, determined at different pavement depths. It also shows the 0.4 % threshold value according to Richartz /3/ and the 1.0 % threshold value according to Rechberger /4/. As is known, "threshold value" refers to that amount of chlorides which the hardening or hardened concrete can bind, in a stable and permanent manner, so that they no longer affect the corrosion protection of the reinforcing steel. As yet, there is no uniform opinion on the quantity of the threshold value, nor on its determining parameters. Frequently, the 0.4 % limit according to Richartz is used to assess the risk of corrosion.

More recent studies at the Cement Research Institute /4/ and at the Technical University of Innsbruck /5/ have shown that the threshold value can be assumed to amount to a minimum of 1.0 % Cl cement weight.

The maximum chloride penetration depth was 4.5 cm; beyond that value, no more chlorides were detected.

When taking the 0.4 % limit as a basis for determining a possible risk of corrosion, Fig. 1 indicates that this value is exceeded, to any major extent, to a depth of 3 cm only. On the basis of the 1.0 % threshold, which the Cement Research Institute holds to be applicable, this content can be found in the top 1.5-cm layer only.

The results indicate clearly that dense concrete of adequate thickness, produced in compliance with the relevant standard, is well suited to protect the reinforcing steel against chloride corrosion. After all, the concrete cover of 3.5 cm, required according to /2/ in case of frost/de-icing salt exposure, is more than double the value of 1.5 cm, i.e. the penetration depth determined for the 1 % threshold value of the chloride concentration. It should be mentioned though, that the pavement concrete in question had a very low water/cement ratio (0.41), was very stiff and had optimum compacting characteristics upon placing, which is the exception.
**Straßenbeton 1958**

$W/Z = 0.41$

$Z = 330 \text{ kg/m}^3$

**O.K. Betondecke**

![Diagram showing chloride contents and location of reinforcing bars](image)

**Fig. 1:** Chloride contents and location of reinforcing bars, determined on drilled cores removed from a highway pavement.

**Fig. 2:** Test slabs
rather than the rule. At the time of the chloride analysis, its strength was about 100 N/mm² (1000 kp/cm²).

3. INVESTIGATING CHLORIDE PENETRATION INTO CONCRETE

In the next step, we therefore studied the question what chloride penetration can be expected in concrete with frost/de-icing salt resistance, produced according to /2/.

The test specimens were test slabs (20/20/12 cm) in the embodiment shown in Fig. 2.

The slabs were reinforced with new bars (ribbed TOR steel 50, diameter 16 mm), placed in the shape of a cross. The specimens designated "O" were tested on the struck-off (screeded) top surface, the specimens designated "U" were tested on the bottom surface, formed when pouring the concrete into the mould (Fig. 2).

Chloride exposure of the test specimens complied with Austrian Standard OENORM B 3303 /6/. Accordingly, the tested top surface was surrounded by a profiled rubber strip (height: 15 mm). A 3 % NaCl solution was poured into the enclosure, with a depth of 4 to 5 mm. The test specimen was then exposed to freeze-thaw cycles, consisting of 16 hours of freeze and 8 hours of thaw conditions.

In deviation from /6/, the test specimens were submersed in water for one week at +20°C, followed by air storage for two weeks at +20°C and appr. 60 % relative humidity, after every 25 freeze-thaw cycles.

We used concrete with a water/cement ratio of 0,50 for the tests. For the sake of comparison, we used concrete with entrained air according to /2/ as well as concrete of the same mix but without air entrainment.

Fig. 3 shows the increase in penetration depth for 0,01 %, 0,4 % and 1,0 % Cl⁻ cement-weight front for Portland cement concrete with a 6 % air content, in relation to the freeze thaw cycles. The solid lines refer to the test specimens with the screeded top surface, the broken lines to the test specimens with the mould-formed bottom surface. Fig. 3 indicates that the threshold concentration of 1,0 % Cl⁻ cement weight - the critical value here - has penetrated to a depth of about 2 cm after 350 freeze-thaw cycles, thus not extending to the reinforcing steel, which has a minimum cover of 3,5 cm. In addition, we could detect not a single case of rust on the reinforcing steel, which was removed after the test.

The following conclusions can also be drawn from the above tests:

- The well-known root—time relationship \( x = a_n \sqrt{t} \) could be established between the penetration depths determined for the 0,4 % front, the 1,0 % front, as well as the 0,01 % front (= maximum penetration depth), and the time of exposure.

- The chloride contents of the concrete surfaces increase with the time of exposure, following the relationship \( C_o = a_o \sqrt{t} \), which demonstrates that the chloride contents increase more quickly initially, but with decreasing speed as time passes.

- In the above tests, the mould-formed surfaces proved to be less susceptible to penetration by de-icing salt solutions, as well as more resistant to frost/de-icing salt action.

- The relationship between effective chloride diffusion coefficients and time of exposure was found to be \( D = a_r \sqrt{t} \).

If, theoretically, the effective chloride diffusion coefficients approach zero, while the surface chloride contents and the chloride penetration depths grow at a decreasing rate with increasing duration of chloride exposure, this means - under practical conditions - that chloride
Fig. 3: Increase in penetration depth for the 0.01 %, 0.4 % and 1.0 % Cl⁻ cement-weight front, in relation to the number of freeze-thaw cycles.

Fig. 4: Interaction between number of freeze-thaw cycles in frost/de-icing salt tests and corresponding chloride exposure times in structures.

Nach einer Näherungslösung des 2. Fick'schen Gesetzes gilt:

\[ x_{\text{max}} = 4 \sqrt{D \cdot t} \]
penetration approaches an finite limit value, as is the case with carbonation front penetrations.

On completion of the tests we knew therefore that the depassivating 1% front had penetrated the air-entrained concrete, produced in compliance with the relevant standard, by about 2 cm after 350 freeze-thaw cycles and had not reached the reinforcing steel, with its 3.5 cm concrete cover, which means that the steel was protected sufficiently against corrosion.

The next question was how to translate the results into practice.

This complex issue was solved by calibrating the chloride penetration curves, obtained in the course of the laboratory tests, against curves determined on actual structures. If the chloride penetration curves from a frost/de-icing salt test and from actual investigations on structures correlate, as is shown schematically in Fig. 4, the number of freeze/thaw-cycles in a frost/de-icing salt test, i.e. the corresponding chloride exposure time of a structure, can be determined by means of a diffusion calculation or, as is shown in Fig. 4, by means of an approximation result from the application of Fick's second law \( x_{\text{max}} = 4\sqrt{D \cdot t} \).

In case of congruent curves \( x_{\text{max}}, \) frost/de-icing salt test equals \( x_{\text{max}}, \) structure. The relationship between chloride exposure time in the frost/de-icing salt test and the actual structure is therefore

\[
t_{\text{frost/de-icing salt test}} = \frac{D_{\text{structure}}}{D_{\text{frost/de-icing salt test}}} t_{\text{structure}}
\]

Fig. 5 shows that 6 freeze-thaw cycles, performed in the laboratory in keeping with the relevant standard, both with stagnant and trickling de-icing solution, result in the same chloride penetration on the structure, as caused by one winter season with de-icing salt exposure. The Figure also shows the ratio for the diffusion coefficients, which amounted to about 10 for the concrete types under review. This signifies that the standardized frost/de-icing salt test has ten times the intensity of a frost/de-icing salt exposure under practical conditions, as far as chloride penetration is concerned. In other words, the above 6 freeze-thaw cycles, performed during the standardized test in the laboratory, correspond to 60 freeze-thaw cycles under practical conditions. For splash-water exposure, the respective figure is 3 freeze-thaw cycles under standardized frost/de-icing salt test conditions, and for finely sprayed water, containing de-icing salt, the respective figure is 1 freeze-thaw cycle, which will result in the same chloride penetration as caused by one year, i.e. one winter season, of de-icing salt exposure under practical conditions.

The conclusion - with considerable practical relevance - is therefore that stagnant or trickling de-icing salt solutions have twice the effect of chloride saturation and penetration than spray water containing de-icing salt, and six times the effect than finely sprayed water containing de-icing salt. When knowing that 6 freeze-thaw cycles, performed in a standardized frost/de-icing salt test, are equivalent to one year of frost/de-icing salt attack by stagnant or trickling de-icing salt solutions under practical conditions, it can be derived that the 350 freeze-thaw cycles of the frost/de-icing salt test correspond to about 60 years of de-icing salt exposure under practical conditions, as far as chloride penetration is concerned.

The time-related evolution of the surface chloride content and of the diffusion coefficient, as obtained during the tests, was then entered into a diffusion calculation, in order to determine the time required until the depassivating 1% limit reaches the reinforcing steel, protected by a minimum concrete cover of 3.5 cm. The resulting figure was 110 years.

The above period, as well as the fact that after that period the protective effect only, i.e. passivity, is lost, justify the statement that a minimum concrete cover of 3.5 cm is generally sufficient provided, of course, that the concrete is resistant to frost/de-icing salt exposure.

4. THE INFLUENCE OF CRACKS

It is obvious that the reinforcing steel can be protected against chloride, oxygen or moisture
<table>
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<tr>
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<th>Spritzwasser</th>
<th>Sprühnebel</th>
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<td>6</td>
<td>3</td>
<td>1</td>
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<tr>
<td>Frost - Tau - Wechsel im Frost - Tausalzversuch gemäß ÖNORM B 3303</td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>$D_{FT}$-Versuch $D_{Bauwerk}$ (&quot;Intensitätsfaktor&quot;)</td>
<td>~ 10</td>
<td></td>
<td></td>
</tr>
<tr>
<td>d. h. 1 FTW im FTS-Versuch entspricht 10 FTW unter praktischen Verhältnissen</td>
<td></td>
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</tr>
</tbody>
</table>

Fig. 5: Interaction between a specific number of freeze-thaw cycles in frost/de-icing salt tests and its corresponding chloride exposure time in structures

Fig. 6: Exposure of test specimens
Fig. 7: Removal of material for analysis
Fig. 8: Chloride contents at crack origin (point A) in relation to crack width - Portland cement concrete; water/cement ratio = 0.50. (The symbols designate exposure periods of 0.5, 1.0, 1.5, 2.0 and 4.0 years. Plain symbols, e.g. •, mean "uncovered exposure", solid symbols, e.g. ○, mean "exposure under hood". Cracks with rust signs are marked by two symbols, e.g. ◦).
attack by means of a thick, dense “concrete jacket”, resulting in no or little exposure only. It is also obvious that air, water and harmful substances have easier access to the steel surface in case of cracks, which affect the density of the concrete cover.

In this connection, there are two issues:

- Is there an admissible crack width which - when not exceeded - prevents chloride penetration to the reinforcing steel, and therefore corrosion, during the service life of a structure?

- What, in general, is the influence of cracks upon the risk of corrosion due to chloride exposure?

The following tests were made, in order to study these extremely complex issues:

Fig. 6 shows the test arrangements. The beams, used for the tests, were provided with a main reinforcing bar (diameter 12 mm), a handling reinforcing bar (diameter 8 mm) and stirrups (diameter 6 mm). The beams were placed in a flexural tensile testing apparatus, converted to state II, and tensioned in pairs, allowing for permanently open cracks of different widths (0,05 to 0,8 mm). The pairs of beams were then placed in tanks. The cracked sides of the beams were submersed in different test solutions - a 3 % NaCl solution or a 3 % NaCl solution and covered with a plastic hood, in order to obtain a relatively high humidity, and in pure tap water (for the sake of comparison) - to half the beam height. The submersion periods were 2 weeks during the first half of the test and 3 weeks during the second half. At intervals of 2, or 3 weeks respectively, the beams were turned over.

After an exposure period between 3 months and 4 years the chloride contents were determined at the Technical University of Innsbruck, at point A (crack) and points B (along a longitudinal section in the reinforcing plane), as is shown schematically in Fig. 7.

Fig. 8 shows the results of the chloride-content determination at point A (crack origin in the steel) for the beam specimens, made of Portland cement concrete with a water/cement ratio of 0,50. The chloride contents were plotted in relation to the crack width, which was measured at the beam surface. The following conclusions can be drawn from the results shown in the diagram:

- The chloride contents, shown on the ordinate axis, increase in almost linear proportion to crack width.

- The five straight lines in the diagram refer to the chloride contents after an exposure time of 6, 12, 18, 24 and 48 months. The Figure shows that the increase in exposure time corresponds almost to a parallel displacement of the straight lines.

- This presentation of the test results illustrates clearly the solution to the issue “admissible crack width”. If it is defined as that crack width where, when not exceeded, the threshold value is not reached at the crack origin in the steel during the service life of a structure, we can see that, for the 1 % limit, admissible crack width decreases with the chloride content, which in turn increases in the course of time. While it was still 0,4 mm after 6 months of exposure, it was only about 0,1 mm after 2 years and zero after four years, i.e. the threshold value of 1 % chloride of the cement mass is exceeded for every feasible crack width. It is only a matter of time, and intensity of the chloride exposure, for the admissible crack width to become zero. As a result, there is no admissible crack width.

Major in-depth corrosion in the crack area occurred only in test specimens with a water/cement ratio of 0,70 and a concrete cover of 2 cm. The beams of concrete with water/cement ratios of 0,40 and 0,50, and a concrete cover of 3,5 cm, exposed under the same conditions, only showed a rust film in the crack area.
Fig. 9: Corrosion models for areas with impaired corrosion protection (according to /7/)

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In simplified model form (Fig. 9) this phenomenon can be explained as follows: in permeable concretes, a macro-element could appear, with the steel in the crack origin constituting the anode and the adjacent areas forming the cathode. As a result, the involved cathode surfaces will increase with the conductivity of the concrete and its oxygen permeability. Consequently, the ratio between anode and cathode decreases, which accelerates and enlarges anode erosion.

Whenever the concrete cover is sufficiently dense and thick, any possible corrosion is restricted to the area of the "bare and exposed" steel in the crack origin, for lack of conductivity and oxygen permeability of the concrete in the vicinity of the crack. When corrosion appears under such conditions, it is in the form of a normal, erosion-type of corrosion with tolerably little erosion, which will probably be preceded in most cases by depassivation of the crack origin, due to carbonation. This could be corroborated on newly removed reinforcing steels, by means of the phenolphthalein test.

It can be assumed, on the basis of results from investigations on structures and the above-described tests, that an adequately thick concrete cover of sufficiently dense concrete will protect the reinforcing steel against corrosion, also against higher chloride contents, which may be higher than any feasible threshold value, since under normal climatic conditions - the volume of voids, consisting mainly of gel pores, is usually saturated with water, which prevents oxygen access to a large extent, and since the existing water content provides insufficient electric conductivity only.

5. SUMMARY

- When investigating test cores from a concrete highway pavement, built in 1958, and analysing chloride penetration in laboratory tests, it could be established that concrete, complying with the relevant standard, providing a minimum cover thickness of 3.5 cm can protect reinforcing steel with reliability against chloride corrosion. Only in the event of severe de-icing salt exposure, an increase in concrete-cover thickness to 5 cm is advisable.

- The results of an investigation on the influence of crack width indicate that there is no admissible crack width.

- provided that the concrete cover is sufficiently dense and thick, cracks with a width of approximately 0.4 mm, as is usual on reinforced concrete, will not adversely affect the durability of the structure. The appearance of extremely wide cracks (> 0.4 mm) should be prevented by an adequately strong minimum reinforcement.

- On account of the interaction, described above, it can be said that the threshold value is not of such importance, as is assumed by some research workers. Chloride contents at the steel, exceeding the threshold value, may eliminate the corrosion protection; yet, corrosion will not occur, if the concrete cover is of sufficient quality, since this will not create the necessary electro-chemical conditions. Exceeding the threshold value at the reinforcing steel is a necessary, but not the only, prerequisite for corrosion.

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/7/ Untersuchungen zur Chlorideindringung in Beton und zum Einfluß von Rissen auf die
Protecting Concrete by Flexible Waterproofing Slurries

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PROTECTING CONCRETE BY FLEXIBLE WATERPROOFING SLURRIES
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Summary:
Flexible waterproofing slurries consist of sand, cement, water, an aqueous polymer dispersion, and various additives. Their polymer content is 7 - 9 weight %. They are applied in two to four coats of 2 - 4 mm total thickness onto wetted concrete. The bulk of the water evaporates, and the polymer forms a tough, rubbery film, which bridges over fine cracks and waterproofs the concrete without impeding water vapour diffusion, i.e. for drying of concrete to stop corrosion of the reinforcement. Concrete coated with flexible waterproofing slurries practically no longer carbonates, and chloride penetration is considerably suppressed. Laboratory studies and many years of practical experience have demonstrated that flexible waterproofing slurries are eminently suitable for additionally or subsequently protecting of concrete, e.g. on sections of road bridges exposed to spraying water and generally in the case of repair.
1. INTRODUCTION

Flexible waterproofing slurries consist of cement, sand and polymer dispersed in water. They are applied on wet concrete surfaces 2 - 4 mm thick. The polymer content is at least 40% by mass of the cement. As a result, the cement no longer forms a firm framework. In fact the polymer bonds the mixture of sand and cement to yield a soft and flexible structure, roughly comparable to that of ebonite.

The positiv experience gained in building constructions since about 1980 soon showed that flexible waterproofing slurries could also be used for protection of reinforced or prestressed concrete exposed directly to the elements, with the aim:

- to prevent penetration of water into the concrete to such an extent that further corrosion of reinforcement could be retarded,
- to hinder penetration of chlorides into the concrete, e.g. in zones of road bridges exposed to spraying water [1], and/or
- to prevent the progress of carbonation if the concrete cover is not sufficiently dense and/or is too thin.

The Institute of Building Materials of the Technical University of Munich supervises the performance and properties of rigid [2] and flexible waterproofing slurries [3]. In close cooperation with the Applications Department for Surface Coatings and Construction Chemicals in BASF, Ludwigshafen, extensive studies on the properties of flexible waterproofing slurries were carried out, guided by the compilation of the nine requirements imposed on coatings for concrete and listed in Fig. 1. These requirements had to be met by specimens in the laboratory and on concrete surfaces exposed outdoors on buildings [4]. According to the results obtained with slurries of slightly different consistence here the main properties are described.
2. BASIC MATERIALS

2.1 Composition and properties of the freshly mixed slurry

Manufacturers of construction chemicals supply flexible waterproofing slurries in two components. The powder component contains the fine sand (maximum grain size about 1 mm), cement, and various additives often including flame retardants. The wet component contains the polymer dispersion. The two components are mixed together with conventional stirring tools.

With a dispersion of acrylic styrene type suitable properties were obtained with a mass fraction of polymer in the slurry between 7 and 9 %, Fig. 2. Since the figure determined for the density of the fresh slurries was 1.6-1.9 kg/dm³, the proportion of polymer was roughly 110-170 kg/m³ or 11-18 vol.-%. The cement content was 350-400 kg/m³ (Portland Cement 35 F or 45 F). Thus the ratio of the water contained in the dispersion to the cement was about 0.7.

The density of the fresh slurry also depends on small air bubbles produced during mixing. The magnitude of total air content should be not too high in order to ensure an effective seal. For that reason the frothing due to mixing usually is reduced by additives.

Owing to the presence of the polymer dispersion the freshly prepared slurry is extremely tacky, with the consequence that the equipment, formwork, etc. must be cleaned with water at the earliest possible opportunity.

The slurries do not contain any additional chlorides. The highest chloride content (including organic chlorine [6]) determined was 0.05 wt. %.

The colour can be adjusted from white to dark grey depending on the cement and fine sand used.

2.2 Polymer dispersion

The polymer dispersion is milky white and contains the polymer as microscopic particles with a diameter of 0.05-5 μm, emulsifying agents and other additives, e. g. wetting agents and thickeners: The polymer and the additives govern the storing stability, the compatibility with cement, the film formation, and other properties of the dispersion as well as the important colloidal chemical interaction with the cement and the fine sand.

The stabilized polymer particles in the aqueous dispersion dry to form a coherent film, the properties
of which, e. g. the outdoor performance, the resistance to hydrolysis and the glass temperature, are largely governed by the chemical structure of the monomers.

It is important that the dispersion displays the following properties:

- storing stability for at least six months at temperatures between 5 and 30 °C,
- good compatibility with cement and rapid film formation,
- good resistance of the polymer film to outdoor exposure,
- low water absorption (as determined by the DIN 53 495 method [7], e. g. less than 28 wt. % in 24 hours and
- a glass temperature between -5 and -20 °C.

The dispersion used contained no solvents but only water, which evaporates or is bound to the hydrating cement. Consequently, slurries of that kind do not cause any environmental problems.

2.3 APPLICATION

The concrete to be coated should be sandblasted or swept by high-pressure water jets to remove loose layers of cement, dust, etc.. Any sharp edges and ridges must be rounded off to allow a flawless coat of uniform thickness to be applied. The concrete must be adequately wetted to avoid a too rapid water loss from the fresh slurry. The concrete surface must have a dull wet appearance when the first coat is being applied and its temperature should be higher than 5 °C.

The slurry is applied in 2-4 coats with a brush or a hard broom into the surface of the concrete. The thickness of the total coat should be 2-4 mm. The next separate coat can be applied when the previous coat has formed a film. The individual coats must be alternately overlapped. The fact that several coats are applied and that the thickness is higher compared to thinner painting systems imply that even larger pores of the concrete surface can be covered. Hence a previous grouting of the surface can usually be dispensed with.

It is not necessary to keep the freshly applied slurry moist, on the contrary it is important to ensure that it can dry slowly. Exposure to rain or excessive sunshine must be avoided in the first few hours or, even better, for the first day. Likewise, the wet slurry must not be exposed to frost.
Efflorescence may occur if water droplets are formed on the still young slurry, e.g. as a result of condensation, but will be washed off again by subsequent rewetting.

Damaged coatings can be repaired simply by applying fresh coats in the way described above.

4 MECHANISMS DURING HARDENING

Evaporation of the water is necessary for the polymer to form a film. If the slurry would be applied all at once in a very thick coat, the water at the surface would evaporate first, and the film formation beneath the surface would proceed much more slowly with the formation of a higher pore content.

Laboratory tests showed the effect of evaporation or hydration on hardening. A slurry freshly applied on a pane of glass was kept in a standard atmosphere of 20 °C and 65 % relative humidity, and the amount of water removed from it was determined by weighing at regular intervals. After different times corresponding specimens were dried at 105 °C and reweighed in order to determine the amount of evaporable water compared with that added during mixing, Fig. 3: About 90 % of the water added evaporated in 30 hours. The water bound to the cement was less than 5 %. Hence the effective water-cement ratio was less than 0.04, and the calculated degree of cement hydration was barely higher than 15 %. Accordingly, the bulk of the cement acts largely as a filler within the polymer film.

During drying the thickness of a coat decreases so that pores are hardly produced. That's the reason for the slurry should be applied in separate coats of no more than 1 mm thickness each.

5 PROPERTIES OF THE HARDENED SLURRY

5.1 Stress-strain diagram and strength

Values for the modulus of elasticity, the tensile strength and the ultimate elongation [8] depending on temperature are listed in Table 1. The glass temperature of the polymer lay between -5 and -10 °C. Thus cooling from 20 °C to -10 °C was accompanied by an increase in the modulus of elasticity and tensile strength and a decrease in the ultimate elongation. Even in the vitreous state, the polymer is still 100 times more ductile than concrete, as is evident from the figures for the modulus of elasticity and ultimate elongation. Hence the possibility of stresses induced between the slurry and the concrete, e.g. by
differential thermal expansion, shrinkage or swelling, can be dismissed.

5.2 Waterproofing

If used as water repelling coat against driving rain or splashing water, i.e. water without pressure, it is important how the capillary suction into the concrete can be reduced. It was investigated by applying a coat of slurry to the face of concrete specimens. For comparison some of the specimens remained uncoated. The specimens were first kept for 7 and 28 days in a standard laboratory atmosphere of 20 °C and 65 % relative humidity. Afterwards, they were subjected to capillary rise. The weight of the specimens was determined at various intervals of time. The figures thus obtained are compared in Fig. 4a with recommended limits (i.e. \( w = 0.5 \text{ kg/m}^2\text{Vh} \) for water repellent plaster [9], and \( w = 0.05 \text{ kg/m}^2\text{Vh} \), for coatings [10].

Although the coated specimens underwent a decidedly lower increase in weight than the uncoated blanks, they did suck a small amount of water.

It must be assumed that the coats of slurry themselves picked up at least some of this water. Consequently, their water absorption was also determined, Fig. 4b. It can be seen from the results that most of the water taken up by the coated concrete specimens was absorbed by the layer of slurry itself, at least in the first 4-9 days, and that hardly any water penetrated into the concrete.

In addition the moisture content of the concrete just below the slurry was determined after accelerated exposure to water spray cycles. Similar measurements were performed on specimens exposed outdoors to natural rainfall. It was thus demonstrated that the concrete dried continuously under the slurry. From this fact, it can be concluded that moisture from the concrete can pass through the slurry and that the moist concrete can thus dry (unless it sucks up more water from behind than can evaporate through the coating). This is an important point in protecting reinforcement from corrosion: if the concrete remains dry, the steel can no longer corrode even if the chloride content of the concrete is increased.

5.3 Permeability to water vapor

The permeability to water vapor which governs drying was determined for various humidity gradients [11].

The moister the environment, i.e. the more water the slurry contained, the greater was the amount of water
vapor that could diffuse through the slurry, as is true for many other materials, Fig. 5 a.

As the slurry ages it becomes denser – evidently as a result of continuing hydration of the cement, Fig. 5 b. Nevertheless, it still remains more permeable than many other polymer coating materials. The relative water vapor diffusion resistivities for the humidity gradient of 100 % to 50 %, was $\mu = 100-400$ compared with air. Thus the slurries are only slightly denser than concrete in bridge constructions.

5.4 Efficiency in "arresting carbonation"

Although acrylic coatings are known to be permeable to water vapor they are very impermeable to atmospheric carbon dioxide (CO$_2$) which on penetrating the concrete promotes its carbonation. At present for the determination of resistance to carbon dioxide diffusion a method described in [12] is used. This method necessitates bone-dry conditions, which never occur in practice and certainly not in a European climate with an average relative humidity higher than 80 %. The diffusion of any gas through a porous material is reduced - in some cases quite considerably - by the moisture content of the material. Hence the values determined by this method cannot be fully adopted as a criterion for a coat of flexible waterproofing slurry's efficiency in arresting carbonation. Despite these objections, experiments were performed. They revealed that a single coat of 2 mm thickness had the same resistance to diffusion as air of 10 to 100 m thickness.

More realistic experiments were run to determine the actual progress of carbonation in concrete underneath a flexible waterproofing slurry. Concrete specimens of high porosity, i. e. water cement ratio of 0.7, were kept in air with a normal carbon dioxide content. The relative humidity was 45 - 65 %, a condition under which concrete carbonates rapidly. After more than 3 1/2 years, carbonation had proceeded to a depth of 10-12 mm into the unprotected concrete. In contrast to this, no carbonation could be detected under the flexible waterproofing slurry as well as inside the slurry itself.

5.5 Sealing against chlorides

The ability of flexible waterproofing slurries to hinder a penetration of chlorides is an important factor if structures are exposed to deicing salts or sea water.
Preliminary investigations consisted of exposing coated concrete to brine for a period of four weeks. No measurable change in the chloride content of the underlying concrete occurred during this period. Since wet-dry-cycles are said to promote the penetration of chlorides, tests under more severe conditions were performed. Coated concrete disks of 3 cm thickness and produced with a water-cement ratio of 0.7 were subjected to the following cycles: drying for seven days at 30-40 °C followed by 28 days of exposure to the brine.

The chloride contents determined in the concrete just below the coating are listed in Table 2. While no chloride penetration was determined for slurry A, slurries B and C, which had a somewhat higher polymer and air content, considerably retarded, but did not completely prevent the penetration of chlorides under these severe conditions.

Practical tests were also performed on aged concrete exposed to deicing salt on a motorway. Their aim was to determine the change in the chloride content of the concrete after it had been coated and further exposed to chlorides. Thus bridge piers were coated with slurries A and B in the autumn of 1983. Under both slurries the existing chloride distribution no longer has been changed. As was to be expected, seasonal changes down to a depth of a few centimetres occurred in the uncoated concrete in the immediate vicinity of the test area of coatings.

5.6 Aging and outdoor performance

Test coatings applied in 1983 on road and motorway bridges as well as later test-coatings being observed did not undergo any adverse changes. Also accelerated exposure tests were performed in the laboratory to determine conformance with Requirements 3-6 and 8 in Fig. 1. Concrete slabs coated with slurries A and B were exposed to cycles as follows:

1. nine days in a standard atmosphere of 20 °C and 65 % relative humidity with exposure to ultraviolet and infrared radiation from a 4500-watt xenon lamp on the second and fourth days,

2. ten days exposure to water spray on parts of the coated surface and

3. one day exposure to a temperature of -20 °C.

The conditions were made more severe by damaging the coatings slightly, by perforations and cuts, to allow local access of water.
No damage in the form of detachment of the coatings, cracks, or discolouration was observed, neither in the coating itself nor in the vicinity of the artificial damages. The only defect consisted of small blisters between the last 2 of the 3 separate coats of slurry B, which had a somewhat higher polymer content.

Adhesion tests with steel stamps always ended with failure in the slurry and never in the bond to the concrete. It is evident from Table 3 that the strength increases with the age of the slurry and the duration of exposure. Tensile tests on specimens taken from free layers that had been immersed for some months in water demonstrated that the ductility also decreases slightly as a result of prolonged exposure to water. Both these facts indicate that the cement can still hydrate.

5.7 Bridging over of cracks

The ability to bridge over cracks is tested with a special device where the crack opens from 0 to 0.6 mm at a temperature of +20°C. As a rule only a white discoloration originating from the high local strain thus appears in the surface of the coating along the crack.

Of course in practice, more factors must be taken into consideration. For instance, the cement hydrates somewhat more in the course of time and thus the slurry becomes more rigid, and cracks are often widened at lower temperatures. Hence it can be assumed that cracks can be bridged over only if they open very slightly, e.g. less than 0.2 mm, and if the movements that they undergo are very slow. Wider cracks that exist at the instant when the coating is applied may more surely be bridged over, also provided that the width subsequently changes only slightly and/or very slowly.

The sole source of information on the long-term performance of coatings for bridging over cracks is practical observation. One example that can be called to mind is that a flexural crack of about 0.3 mm width in a crossgirder of a bridge slab underwent roughly ±10 % movement as a result of the traffic. This crack was successfully bridged over for more than 3 years by a flexible waterproofing slurry.

6 FURTHER PRACTICAL EXPERIENCE

The good results obtained on test coatings and the experiences with similar slurries in Switzerland and Austria for more than 8 years gave rise to the decision of the German Federal Railways and Motorways Authorities to coat concrete structures with flexible waterproofing slurries:
- to avoid penetration of water,
- to suppress carbonation,
- to hinder chloride penetration.

The Echelsbach Bridge which spans over the River Ammer gorge in Southern Bavaria with an arch of 130 m width was erected in 1928-1929 and had to be repaired in 1985 - 1986. It was not possible to replace all concrete parts where chloride has penetrated. A coat was applied on the whole bridge to keep the concrete dry. Not only rainwater and chloride had to be excluded but also the condensation formed by warm air fronts and high relative humidities.

Areas of inadequate concrete cover have been found in a number of new reinforced concrete structures. They have had to be coated with flexible waterproofing slurries to protect against carbonation whenever the thickness of the concrete cover was still at least 1 cm.

A test coating was also applied to a much-frequented footway deck on a bridge and was still intact after about 3 years hard wear.

However, practical experience has shown that two important limitations must be observed:

1. If the concrete can suck water from behind, e.g. at badly sealed joints, at a greater rate than it can evaporate through the coating, the coating may be detached locally and frost damage may subsequently occur.

2. Although flexible waterproofing slurries are alkaline, they do not protect steel from corrosion if they are applied to it directly. It would appear that their high polymer content prevents the steel from adequately being passivated [13]. Hence, if there is no concrete cover, the steel should be coated with a cement mortar of at least 5-10 mm thickness before the flexible waterproofing slurry is applied.

Accelerated exposure tests in the laboratory and five years field observations of exposed test coatings have demonstrated that flexible waterproofing slurries offer high resistance to weathering and that local mechanical damage does not produce further detrimental effects. Coatings applied to large bridges have confirmed the material's robustness. There have been no signs as yet of any damage mechanism that could shorten the life of the flexible slurry if the instructions given in this report are taken into consideration.
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/13/ Volkwein, A.: Anstriche als Korrosionsschutz der Bewehrung bei Sanierungen? In: (see /10/)
<table>
<thead>
<tr>
<th>Test temperature in °C</th>
<th>−20</th>
<th>−10</th>
<th>20</th>
<th>50</th>
</tr>
</thead>
<tbody>
<tr>
<td>Min. value from 3 experiments</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Modulus of elasticity, N/mm²</td>
<td>460</td>
<td>405</td>
<td>14</td>
<td>10</td>
</tr>
<tr>
<td>Tensile strength, N/mm²</td>
<td>6.5</td>
<td>4.8</td>
<td>0.65</td>
<td>0.39</td>
</tr>
<tr>
<td>Ultimate elongation, %</td>
<td>1</td>
<td>2.5</td>
<td>28</td>
<td>12</td>
</tr>
</tbody>
</table>

Table 1. Mechanical properties of a coating obtained from a flexible waterproofing slurry (age of specimens: 56 days)

<table>
<thead>
<tr>
<th>Slurry</th>
<th>Conditioning before test commenced</th>
<th>Chloride content of concrete, wt.%¹ after 1st cycle</th>
<th>. . .</th>
<th>2nd cycle</th>
<th>. . .</th>
<th>3rd cycle</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>28d 20 °C/65% r.h.</td>
<td>not measured</td>
<td>0.047</td>
<td>0.057</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>28d water</td>
<td>0.060</td>
<td>0.046</td>
<td>0.068</td>
<td></td>
<td></td>
</tr>
<tr>
<td>B</td>
<td>28d 20 °C/65% r.h.</td>
<td>0.830</td>
<td>1.330</td>
<td>1.470</td>
<td></td>
<td></td>
</tr>
<tr>
<td>C</td>
<td>28d 20 °C/65% r.h.</td>
<td>0.780</td>
<td>not measured</td>
<td>1.660</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

¹ Intrinsic chloride content of the concrete: 0.045 — 0.050 wt.%.

Table 2. Chloride content at a depth of 0–1 cm in concrete coated with three different slurries after exposure cycles consisting of drying followed by wetting

<table>
<thead>
<tr>
<th>Test performed after following exposure conditions</th>
<th>Slurry A</th>
<th>Slurry B</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial state (age 56 days)</td>
<td>0.13</td>
<td>0.15</td>
</tr>
<tr>
<td>4.5 months at 20 °C and 65% r.h.</td>
<td>0.44</td>
<td>0.40</td>
</tr>
<tr>
<td>Accelerated exposure¹</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Areas with and without wetting by simulated driving rain</td>
<td>0.43</td>
<td>0.44</td>
</tr>
<tr>
<td>after 1st cycle</td>
<td></td>
<td></td>
</tr>
<tr>
<td>after 5th cycle</td>
<td>0.73</td>
<td>0.61</td>
</tr>
<tr>
<td>Wetted area only</td>
<td>0.88</td>
<td>0.74</td>
</tr>
<tr>
<td>Not wetted area only</td>
<td>0.59</td>
<td>0.48</td>
</tr>
</tbody>
</table>

¹ cf. Para. 4.7

Table 3. Tear off strength of two flexible waterproofing slurries after various preconditioning (tensile failure inside the slurry in each case)

1. Water + salt (Cl, SO₄ …) Impermeable to water
2. Water vapour Permeable to water vapour
3. Corrosive substances Ultra-violet radiation Resistance
4. Water Forces causing detachment Adhesion — even to moist concrete
5. pH > 12 Hydrolisis … chemical mechanical Compatibility
6. Heat/Cold Stresses Shrinkage/Swelling αₜ, E, ultimate elongation, shrinkage swelling
7. Bridging cracks
8. Water No osmosis
9. CO₂, O₂ … Impermeable to gases

Fig. 1 Requirements imposed on protective coatings for concrete
Fig. 2. Composition of a flexible waterproofing slurry

Fig. 3 Water loss during drying in a standard atmosphere of 20 °C and 65% relative humidity and at a temperature of 105 °C
a) Water absorbed by slurry plus that entering the concrete:

Concrete water/cement = 0.5
Concrete water/cement = 0.7

With slurry (age)

E (28d)  B (1,6 mm)
E (28d)  1,9 mm

Days

Square root scale

b) Water absorbed by free layer of slurry:

Slurry:

B (28d; 1,6 mm)
A (28d; 1,9 mm)

Resistance to water vapor diffusion ($\mu'_L$ expressed in terms of air) for various humidity gradients and slurries of different age:

Fig. 5

a) Relationship to average humidity:

Slurry Kept at 20 °C/65% r.h. until age of

B 33 d
B 60 d
B 150 d
A 33 d
A 60 d
A 150 d
C 33 d

Average humidity in %

Average humidity gradient in %

b) Relationship to age (humidity gradient 100/80%)

Fig. 4 Water absorption by coated concrete specimens (a) and by free layers of slurry (b)