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# Blacktop Resurfacing of Bridge Decks

FINAL REPORT  
June 2001

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# **1 Review of the Background of Bridge Deck Deterioration**

The delamination (spalling) of concrete in bridge decks has become a serious problem [1.1]. Durability of hydraulic-cement concrete is defined, as ability to resist weathering action, chemical attack, abrasion, or any other process of deterioration. Durable concrete will retain its original form, quality, and serviceability when exposed to its environment. More important causes of concrete deterioration include: freezing and thawing, aggressive chemical exposure, abrasion, corrosion of metals, chemical reactions of aggregates. The principal cause of deterioration is reinforcing steel corrosion, which is largely due to the use of deicing salts. The formation of the corrosion products produces an expansive force, which is largely due to the use of deicing salts, that causes the concrete to spall above the steel. Ample cover over the steel and use of a low-permeability, air-entrained concrete will assure good durability in a great majority of cases, but a more positive protection, such as epoxy-coated reinforcing steel, cathodic protection, chemical corrosion inhibitors, or other means, is needed for very severe exposures. We are increasingly aware of the fact that durability of concrete is important, and in some cases it is even more important than its compressive strength. Concrete is durable if no changes appear in it with time, despite being subjected to an action of aggressive actions of freezing and thawing, especially in a combination with a salt action.

Reinforced-concrete structures have long durability in normal environmental conditions. However, if they are exposed to an environmental "overload", which has not previously been taken into account, the durability and function of the reinforced-concrete structure may be reduced so much that the safety of the structure may come into question. Although there is a great number of structures whose properties have, even after only 20 years of service, degraded so much that the safety and function of the structures is endangered, the problem of reinforced-concrete durability still does not receive the proper attention in design, realization, and maintenance. Consequently, the number of the structures with a shorter life span than possible and necessary has increased. Furthermore, structures affected by reinforcement corrosion have been repaired with inadequate materials and procedures, which may, despite

great expense, hasten deterioration. This is why the decision of repair strategy of reinforced-concrete structures is of exceptional economic and technical significance. This is also the reason why guidelines for the repair of reinforced-concrete structures affected by reinforcement corrosion are a part of this report.

Durability of reinforced concrete is primarily determined by the penetration of aggressive substances into concrete. The substances penetrate into a concrete structure, degrading both concrete and reinforcement. Degradation mechanisms are processes or reactions, which change properties of structural elements, or the whole system; consequently, the system does not behave as anticipated. It is very important for a civil engineer to have good understanding of degradation mechanisms, if proper design and performance are to be achieved, including the life-span expectation of the materials and structure as a whole.

Damaged residential, industrial, transportation and other structures can be repaired, but often at high cost. If damage is to be repaired at a stage when it is significant, it often becomes an unsolvable economic problem. Some damaged industrial structures may cause environmental pollution, adding ecological problems to already existing economic ones. Damage in reinforced concrete structures is caused by many factors [1.2]. The most common cause of damages to reinforced concrete structures is reinforcement corrosion. The influences (actions) that cause reinforcement corrosion may be observed in two phases of a reinforced-concrete structure's life span, (Table 1.1 [1.3]). In the initial phase, agents (substances) penetrate into concrete. Carbonization, penetration of chlorine ions, sulfates, that later on is accelerated by drying and wetting processes, are examples of harmful agents' penetration in the initial phase.

**Table 1.1** Causes of damages of reinforced-concrete structures

Alkali-aggregate reaction	(6%)
Construction fault (design error), cause of reinforcement corrosion	(33%)
Influence of chloride cause of reinforcement corrosion	(19%)
Geotechnical problems	(23%)
Other	(19%)

The service life and utility of concrete depends strongly on its transport properties, i.e. permeability, sorptivity and chloride permeability. The ingress of potentially deleterious materials, such as: chlorides, sulfates and water by diffusion and capillary transport, can lead to corrosion of steel reinforcement or to a reduction of strength due to cracking by frost or sulfate attack. Clearly, a variety of transport mechanisms play an important role in degradation of concrete and must be better understood to mitigate such processes. Depending on its mix design, preparation and environmental exposure, material properties of concrete can be highly variable. To make assessment of the quality and condition of materials; standard experimental test methods are needed to measure key transport properties, such as: permeability to fluids, sorptivity and the diffusion of ions. Without standard measurement methods it is difficult to develop objective criteria needed for prediction and assessment of the service life of concrete as a construction material and barrier.

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## **2 Classification of the Environmental Loads Based on Reinforced Concrete Performance**

Many other environmental effects can, in addition to dead and live loads, affect stability and safety of reinforced concrete structures. Since it is possible to simulate mechanisms of these influences mathematically, i.e. quantify their effects, they can be considered as a special type of loading that can be incorporated into reinforced concrete structures design procedures. That way, instead of using current general recommendations, environmental loads could also take part in definition of the size of a cross-section, reinforcement bar size, and of material properties. To work out a simple concept for a design procedure that includes environmental loads, it is suggested to classify them in four basic sets: chemical, physical, biological and mechanical, with corresponding subsets. Each type of the environmental load corresponds to a certain characteristic of a structure based on reinforced concrete reaction to their activity, and each characteristic corresponds to a certain testing method [2.1,2.2]. Results from performed tests can be used as input parameters in mathematical models for environmental effects, as well as for quality control of reinforced concrete structures.

Every structure is to be designed for dead, live and special loads so that its bearing capacity is not exceeded. Poor condition of numerous existing offshore and under-water structures points out a requirement for developing a design procedure that will consider environmental and durability conditions [2.1]. Therefore, it is proposed to include environmental loads into design procedures. Environmental loads are those that initiate or accelerate deterioration processes of materials, especially the corrosion of steel in concrete [2.3]. Besides cracking and spalling of the cover, environmental loads cause reduction of the active reinforcement bar of cross section that can result in a loss of the section bearing capacity. Influence of environmental loads has not been considered in the design of reinforced concrete structures so far, except in part in cracking limit state design criteria. Such an approach in the design and construction of durable concrete structures does not meet requirements of modern engineering

construction of increasingly more sophisticated and demanding structures. The result of disregarding environmental loads may be failure of parts or the entire structure.

## 2.1 Classification of Environmental Loads

Regourd [2.4,2.5] RILEM committees [2.6], Mehta [2.7], Moskvina [2.8], Bjugovic [2.9] and others proposed classification of aggressive environmental actions on reinforced concrete structures. Because of their extensive descriptiveness, those classifications are not practical for civil engineering purposes. A simple classification of environmental loads that can be included into reinforced concrete design procedures is presented in the report. The classification divides environmental loads into four sets: chemical and electrochemical, physical, biological and mechanical, and their corresponding subsets (Fig. 2.1).

***Chemical and electrochemical environmental loads*** consist of four subsets (Fig. 2.2):

1. *Chemical and electrochemical actions or active environmental loads:*

Corrosion of reinforced concrete, AAR, seawater, sulfate attack, mineral water, acids, bases, salt solutions and water with large amount of free CO<sub>2</sub>.

2. *Reactions to chemical and electrochemical actions or active environmental loads:*

Cover splitting, cracks, spalling, swelling of solid components, solid components' transformation, hydration products solubility.

3. *Characteristic of structures exposed to chemical and electrochemical actions or active environmental loads:*

Depth, volume, mass, reinforcing bar diameter, cover depth, Cl<sup>-</sup> concentration through cover, reinforcement corrosion, penetrability of cover, pH of concrete cover.

4. *Methods for testing performance of the structures exposed to chemical and electrochemical actions or active environmental loads:*

Meter, balance, reinforcement locator, Cl<sup>-</sup> identification, phenolphthalein test, pathfinder (potential measurement techniques), moisture meter, corrosion rate meter, initial



surface absorption test, water permeability, air permeability, gas permeability, O<sub>2</sub> diffusion, Cl<sup>-</sup> diffusion.

**Physical environmental loads** consist of four subsets (Fig. 2.3):

*1. Physical actions or active environmental loads:*

Volume changes because of temperature difference, salt pressure crystallization, reinforced corrosion, extreme temperature as freezing and thawing, fire.

*2 Reactions to physical actions or active environmental loads:*

Spalling, crumbling, cracks, structure decay, splitting.

*3. Performance of the structures exposed to physical actions or active environmental loads:*

Depth, volume, mass of decay, width, depth, frequency of cracks, changes of modulus of elasticity, strength decreasing, porosity increasing, loss of binder, loss of integrity.

*4. Methods for testing performance of the structures exposed to physical actions or active environmental loads:*

Meter, balance, microscope, ultrasound, load testing on samples, chemical analyses of samples, pull-off.

**Biological environmental loads** consists of four subsets (Fig. 2.4):

*1. Biological actions or active environmental loads:*

Shells, bacteria.

*2 Reactions to biological actions or active environmental loads:*

Growing development, structure decay, solid components' transformation.

*3. Performance of the structures exposed to biological actions or active environmental loads:*

Change of the modulus of elasticity, strength decrease, porosity increase, loss of binder, loss of integrity, penetrability of cover, depth.

*4. Methods for testing performance of the structures exposed to biological actions or active environmental loads:*

Ultrasound, load testing on samples, microscope, chemical analyses on samples,

pull-off, initial surface absorption test, water permeability, air permeability, gas permeability, O<sub>2</sub> diffusion, Cl<sup>-</sup> diffusion.

***Mechanical environmental loads*** are divided into four subsets (Fig. 2.5):

*1. Mechanical actions or active environmental loads:*

Erosion, overloading, cyclic loading, impact.

*2. Reactions to mechanical actions or active environmental loads:*

Surface wear cracking, deflection, and fracture.

*3. Performance of the structures exposed to mechanical actions or active environmental loads:*

Depth, volume, mass of surface wear, width, depth, number, frequency and size of cracks.

*4. Methods for testing performance of the structures to mechanical actions or active environmental loads:*

Meter, balance, microscope, ultrasound, mechanical strain gauge, electrical resistance (metal and alloy) inductive displacement.

# ENVIRONMENTAL LOADS - PERFORMANCE OF RCS

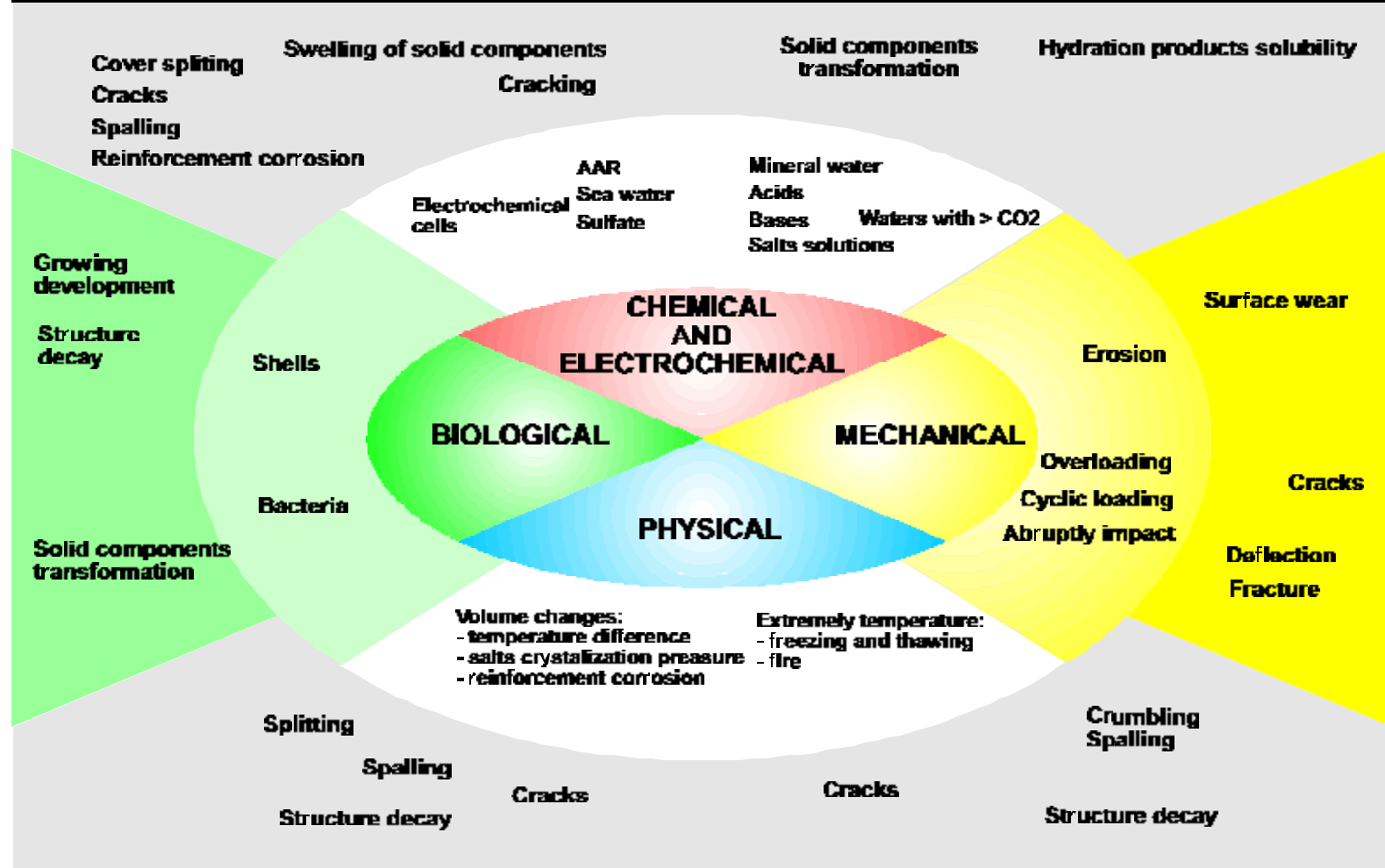


Fig. 2.1 Environmental loads

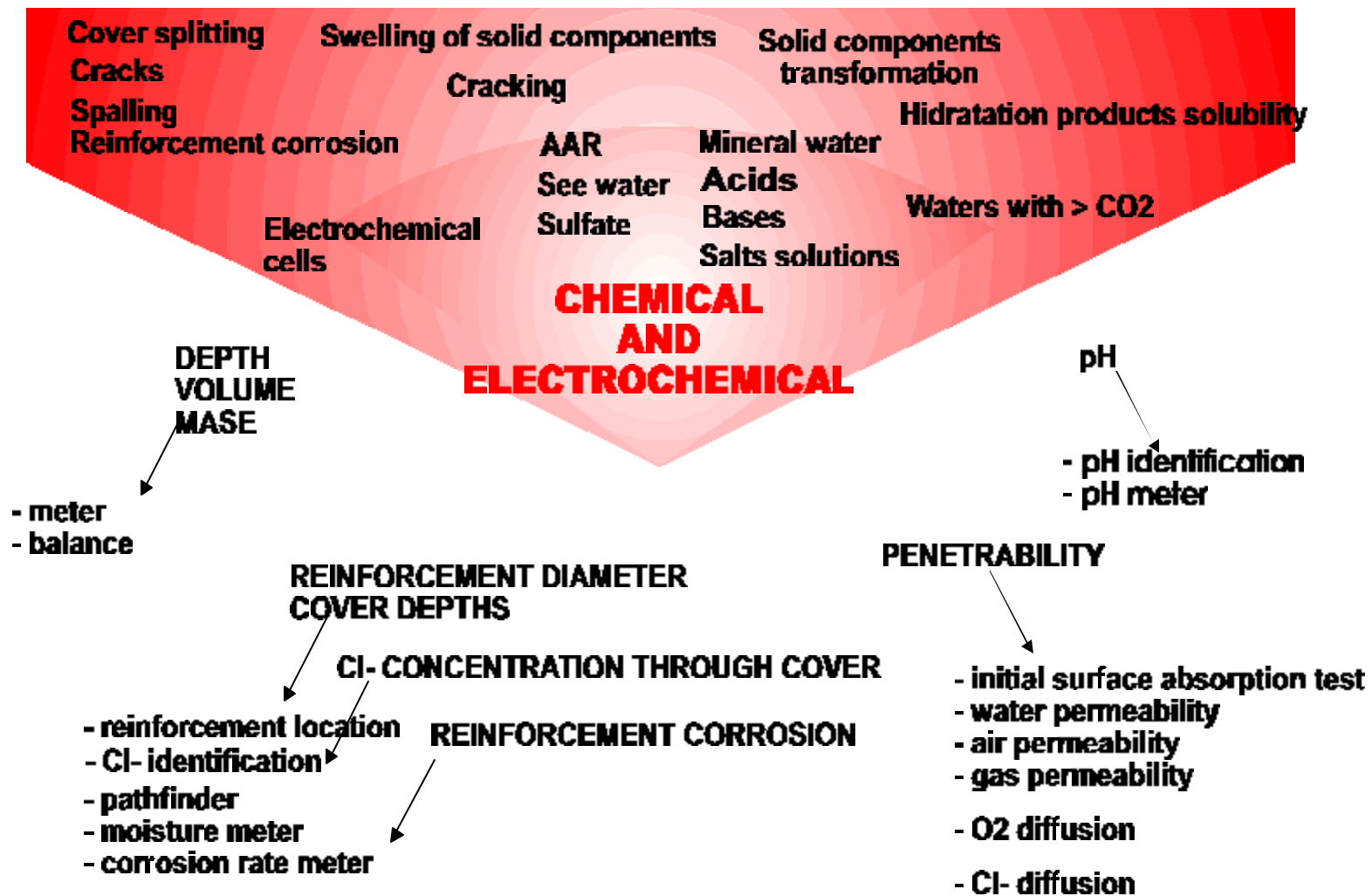


Fig. 2.2 Chemical and electrochemical environmental loads

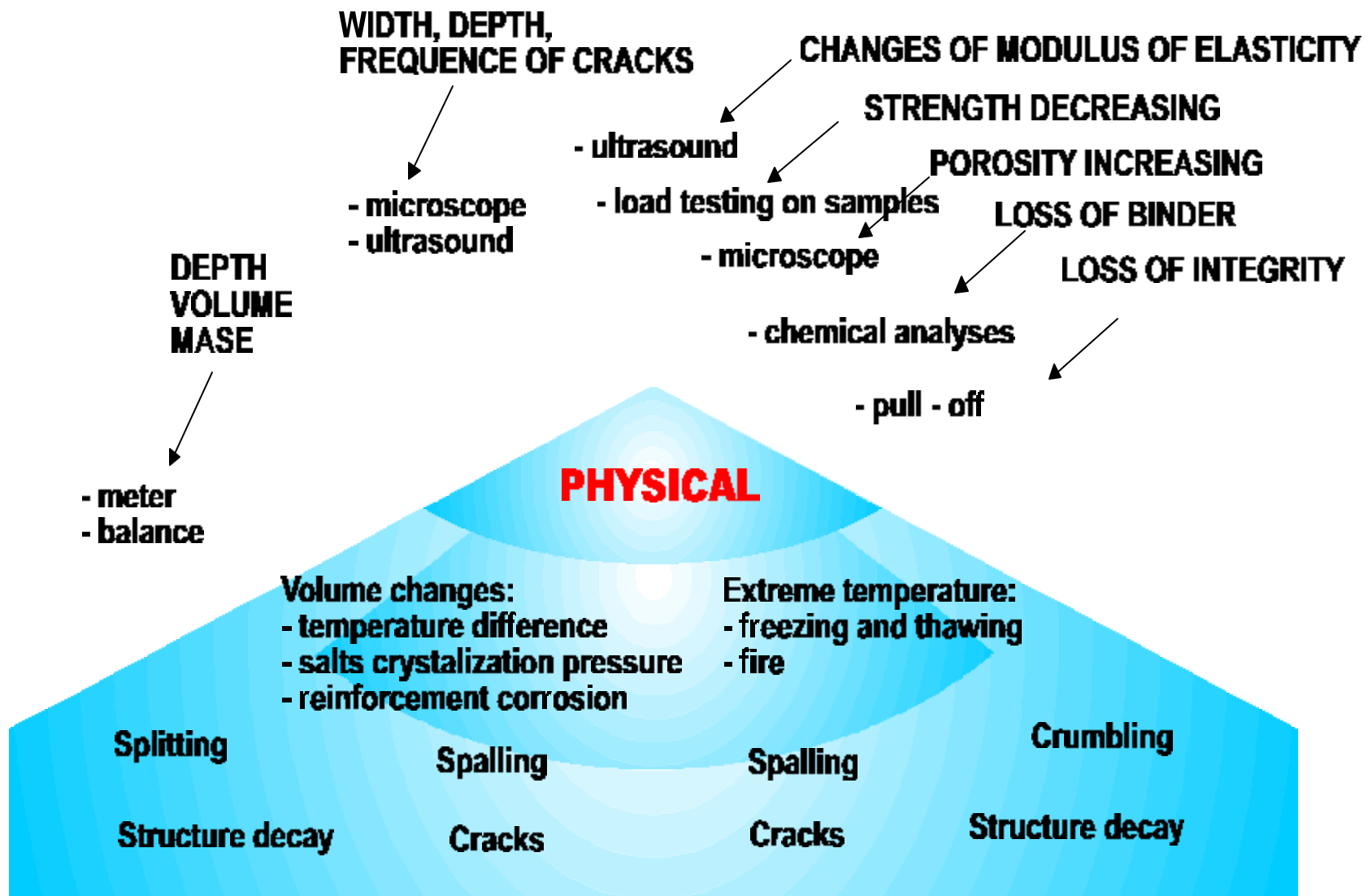


Fig. 2.3 Physical environmental loads

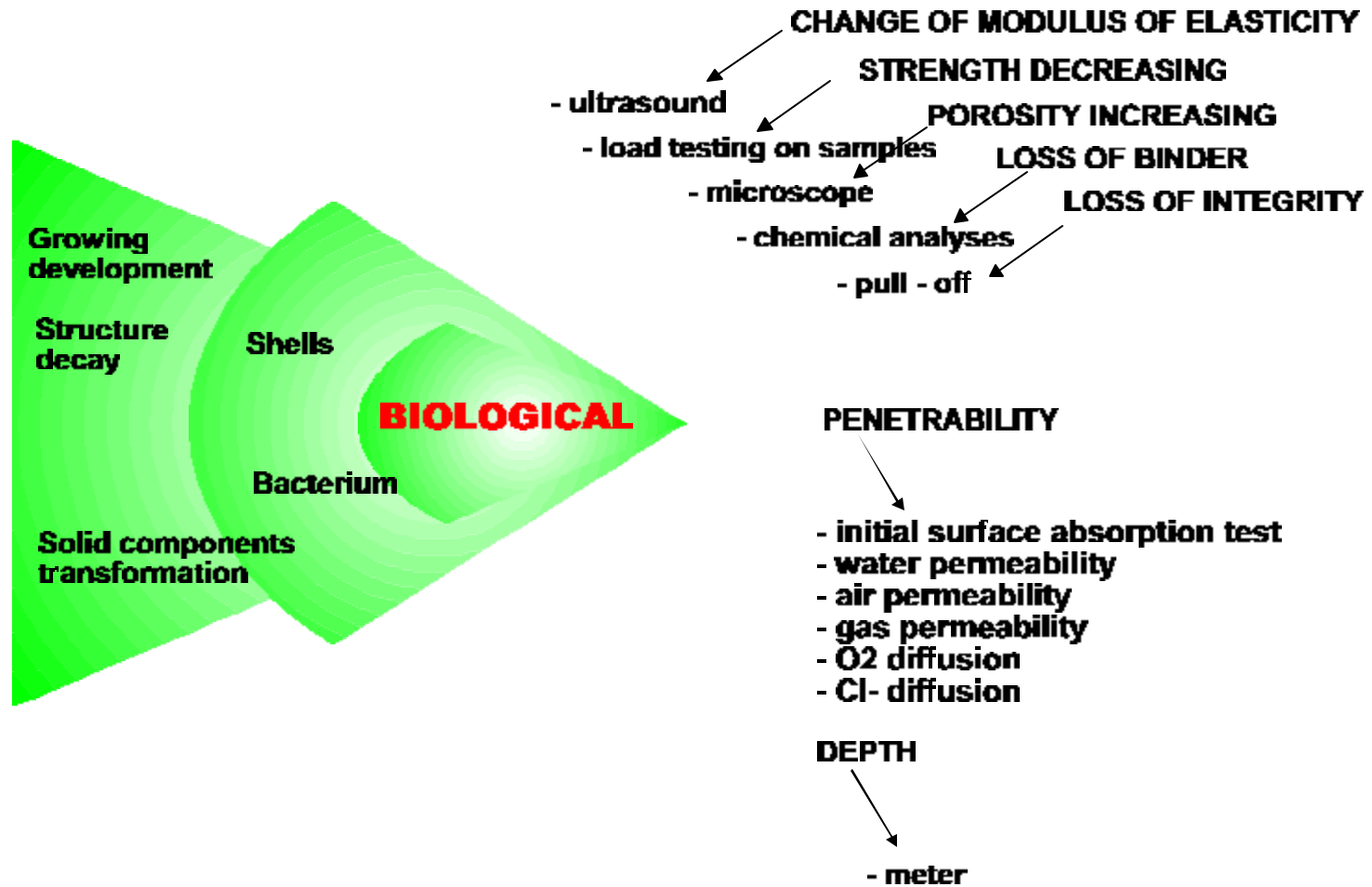


Fig. 2.4 Biological environmental loads

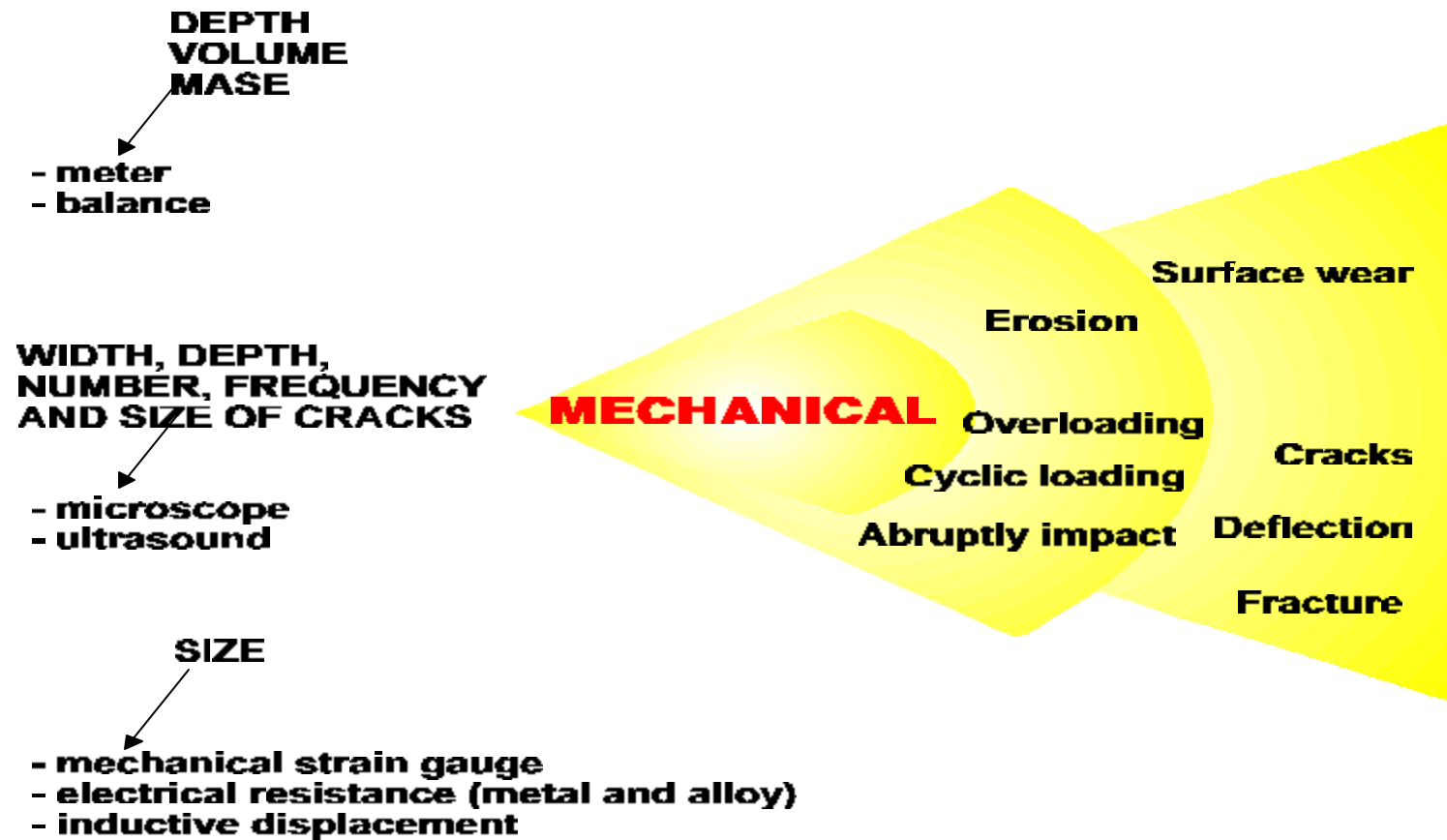
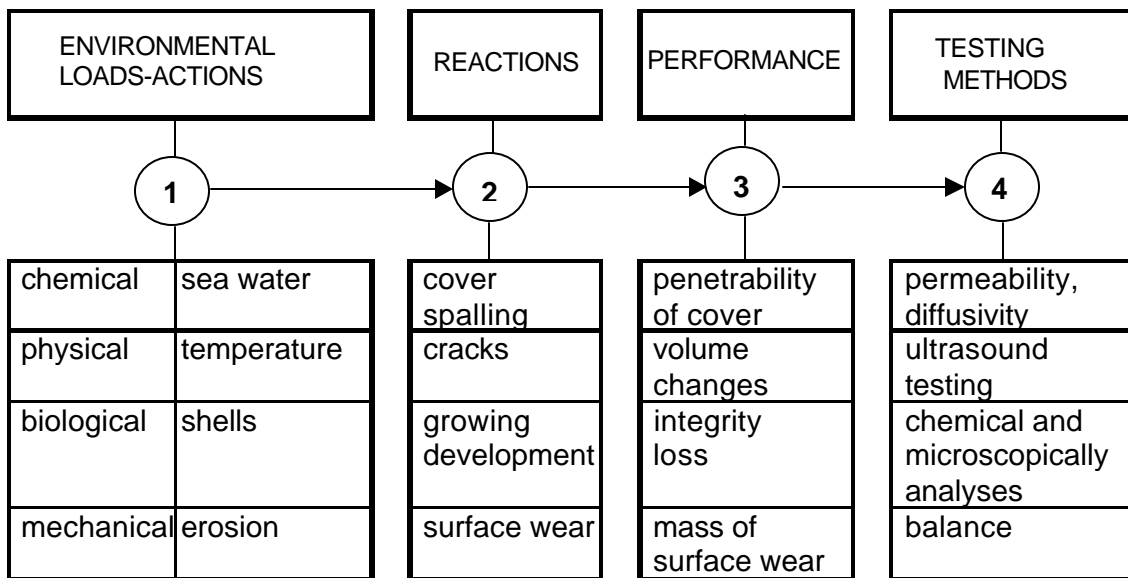


Fig. 2.5 Mechanical environmental loads

## 2.2 Example

Proposed classification of the environmental loads is applied in an example of a bridge abutment submerged in seawater. Active environmental loads, reactions, material performance and corresponding testing methods are given in Fig. 2.6.

### STRUCTURE: Bridge Abutment



**Fig. 2.6** Classification of the environmental loads for the bridge abutment

Parameters obtained from the results of performed tests can be finally used in a design procedure to affect the size of the cross section, quantity and size of reinforcement, material properties, and parameters for quality assurance and quality control.

A service life of a structure is determined largely by its durability properties and existing environmental conditions. Environmental loads can be included in the existing design procedures and thus affect material and geometrical properties to insure an adequate service life of the structure. The proposed classification of environmental loads in four sets: chemical, physical, biological and mechanical, and their corresponding subsets contribute to the



reinforced concrete design procedure by providing the basis for development of additional tools for determination of influences of the environment on the structure and its performance. Testing methods for evaluation of material and structural performance provide parameters for design models and properties relevant for quality assurance.

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### **3 Mathematical Model for Durability Design of Reinforced Concrete Structures**

After twenty or thirty years of service, a reinforced concrete structure may be in a condition that can lead towards its failure. It is not due to design or construction flaws, but due to severe environmental conditions that may cause considerable concrete and reinforcement damage. A procedure (model) for durability design of reinforced concrete structures in aggressive environment has been proposed [3.1]. According to this proposal, the damage is anticipated already during the design phase. Thus, the impact of environmental influences is considerably reduced during the service life of the structure. In this way, the least probable structural damage resulting from the effect of special environmental loads can be provided already in the design phase.

Today, reinforced concrete structures are designed according to the ultimate limit state or the serviceability limit state. In that design, general conditions of the environmental impact are considered only within the serviceability limit state, resulting in a prescribed size of ultimate crack width. Strict criteria of a limiting crack width often result in an increased need for reinforcement. At the same time, cracks wider than those calculated or defined by standards cannot be completely eliminated. To avoid uncertainties about the actual crack width, a new criterion of the serviceability limit state is proposed. This new criterion makes the crack width limit less strict, and also considers numerous other durability factors.

#### **3.1 Corrosion Limit State**

It was proposed that durability factors be called "environmental loads" [3.2]. Environmental loads stimulate or accelerate material degradation, especially the process of reinforcement corrosion in concrete. Consequences of the reinforcement corrosion in concrete are cracking of the concrete cover, and reduction of the surface of steel bars. In the long run this may result in a structural failure. Taking this into consideration, a new criterion is suggested to be called

the "criterion of the corrosion limit state". It is suggested to control the criterion within a regular serviceability limit state design.

According to the criterion of the corrosion limit state, it is necessary to prove that the calculated reinforcement diameter  $\phi_c$  is greater or equal to the limit value,  $\phi_u$ :

$$\phi_c \geq \phi_u, \quad (3.1)$$

or that the calculated service life of a structure  $t_c$  is greater or equal to the design life  $t_d$ :

$$t_0 + t_1 = t_c \geq t_d \quad (3.2)$$

where:

$t_0$  is period of initiation of reinforcement corrosion in concrete

$t_1$  is period of propagation of reinforcement corrosion in concrete [3.3].

The proposed design procedure, requires long enough  $t_0$  and  $t_1$  periods of service life under a condition of simultaneous action of dead, live loads and environmental loads.

### **3.2 Period of Initiation of Reinforcement Corrosion in Concrete**

In the period of initiation of the reinforcement corrosion in concrete,  $t_0$ , the following environmental loads are considered:

- chlorides,
- carbonization,
- sulfates,
- temperature.

It is assumed that environmental loads can initiate the corrosion process in a sufficiently long period of time [3.2]. For structures in a marine environment and objects on roads where the pavement is strewn with salt in conditions of continental winters, chloride is a critical environmental load. The period of initiation of reinforcement corrosion in concrete stimulated by chloride can be determined by a diffusion analysis of chloride ions in a reinforced concrete structure.

The solution of the diffusion problem in a semi-infinite, anisotropic and non-homogeneous medium (reinforced concrete slab) for a constant value of the initial concentration of chloride ions and time-dependent diffusion coefficient is given by the following equation (3.3):

$$C(x,t) = C_o \left[ 1 - \operatorname{erf} \left( \frac{x}{2\sqrt{t}} \right) \right] \quad (3.3)$$

where:

$C(x, t)$  is concentration of chloride ions at depth  $x$  in time  $t$ ,

$C_o$  is initial concentration of chloride ions,

$\operatorname{erf} \left( \frac{x}{2\sqrt{\tau}} \right)$  is error function,

$\tau = \int_0^t D_{Cl^-}(s) ds$ , and

$$D_{Cl^-} = D_{w/c} \cdot D_o \cdot t^{-\alpha.1} [3.4].$$

$D_{Cl^-}$  = diffusion coefficient of chloride ions,

$D_{w/c}$  = diffusion coefficient of chloride ions depends on water/cement ratio.

It is suggested that the value coefficient  $D_o$  be taken according to Table 3.1 [3.2].

**Table 3.1** Coefficient  $D_o$

<b><math>D_o</math></b>	<b>INFLUENCE</b>
1 - 1.35	Sulfates
0.5 - 1	Increase of $C_3A$ quantity
0.4 - 0.9	Addition of fly ash
0.08 - 0.12	Addition of silica dust
0.8	Addition of superplasticisers
2	Permanent temperature difference of $10^\circ\text{C}$ compared to an average value of $20^\circ\text{C}$
1 - 1.3	cracking of concrete under the basic loads
0.04	casting in controlled permeability formwork

The actual initial concentration of chloride ions in concrete surface layer of newly built structures changes with time, from zero to some maximum value,  $C_{\max}$  [3.2,3.5]. Therefore,

it is necessary to modify the solution of diffusion problem defined by equation (3.3) by taking into account the boundary conditions in the following form:

$$C(0,t) = \varphi(t), \quad 0 \leq C_0 < C_{\max} \quad (3.4)$$

For the assumption of a linear increase of the initial concentration of chloride ions with time, the boundary conditions in equation (3.4) can be expressed as:

$$C(0,t) = kt, \quad 0 \leq C_0 < C_{\max}$$

where k is a coefficient of a linear increase of the initial chloride ion concentration. Values of coefficient k are given in Table 3.2 [3.2,3.4].

**Table 3.2** Increase coefficient value of the initial chloride ion concentration (k)

ENVIRONMENTAL CONDITION	k
Wetting zone	10
Tidal zone	1
Atmospheric zone – for structures up to 10 m away from the sea	0.8
Atmospheric zone – for structures at 10 - 50 m away from the sea	0.5
Atmospheric zone – for structures at 50 - 250 m away from the sea	0.2
Atmospheric zone – for structures more than 250 m away from the sea	0.1
Continental area – application of deicing salt	0.1

On the basis of a computational experiment, the diffusion coefficient and the initial concentration of chloride ions are obtained (for a continuous diffusion process of chloride ions in a reinforced concrete structure for a time-varying coefficient k) [3.2]:

$$C(x,t) = [C_0 + k(t-1)] \cdot \left( 1 - \operatorname{erf} \frac{x}{2\sqrt{t}} \right) + k \cdot \left[ \left( 1 + \frac{x^2}{2t_1} \right) \left( 1 - \operatorname{erf} \frac{x}{2\sqrt{t_1}} \right) - \frac{x^2}{\sqrt{p \cdot t_1}} e^{-\frac{x^2}{4t_1}} \right], \quad (3.5)$$

$$0 \leq C_0 < C_{\max}$$

where:

k is the coefficient of linear increase of initial concentration (Table 3.2),

$\tau$  is the substitution by which variation of  $D_{Cl^-}$  with time is taken into account [3.4], and

$$\tau_1 = \tau(t=1).$$

After reaching the maximum initial concentration  $C_0 = C_{\max}$  the equation (3.3) is valid again. The period of initiation of the corrosion process is finally determined from the condition of critical chloride ions concentration at the reinforcement level:

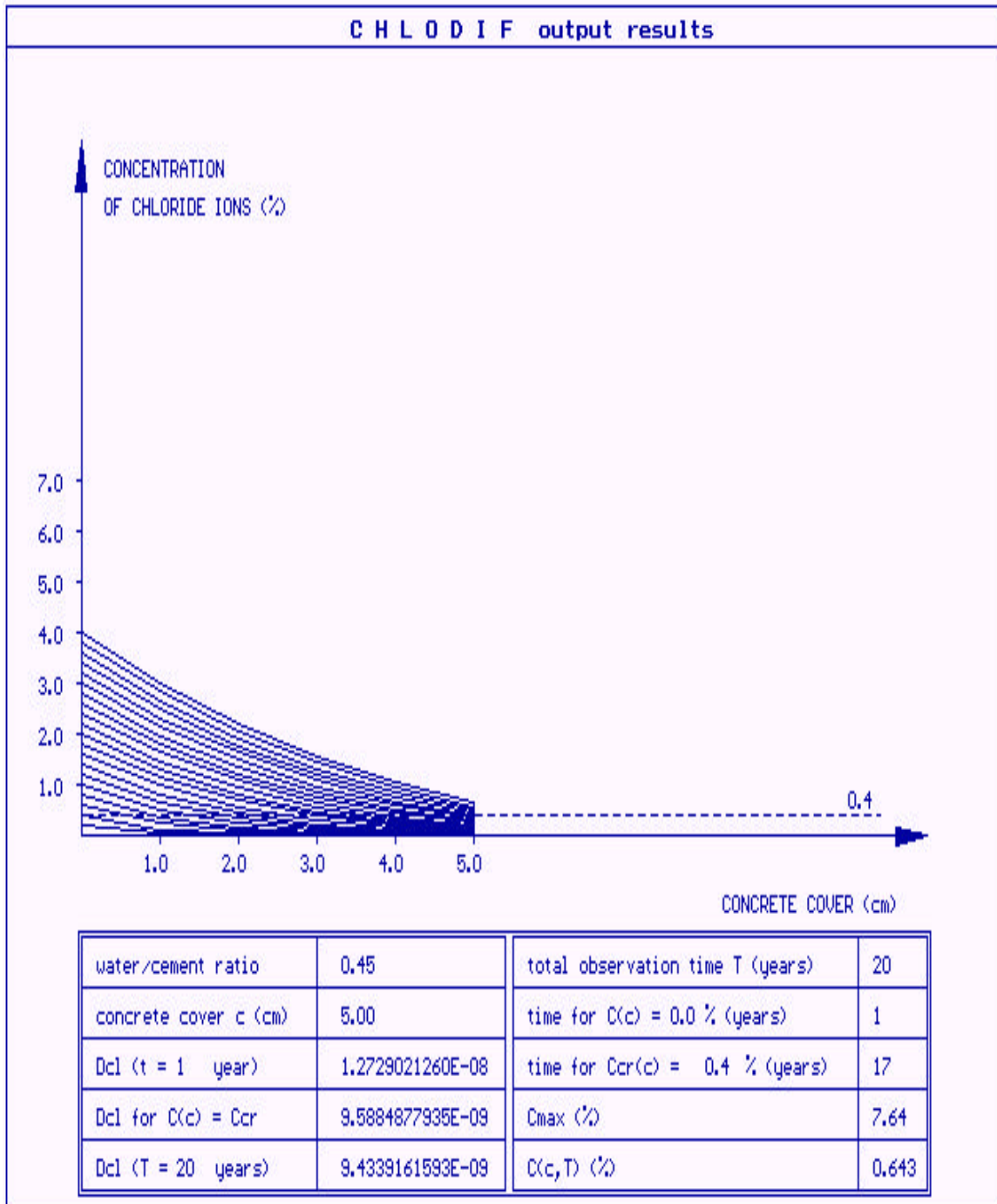
$$C(c, t_0) = C_{cr} \quad (3.6)$$

where:

$c$  is thickness of concrete cover,

$C_{cr} = 0.4\%$  of cement weight [3.6].

For the purpose of calculating the period of initiation of a corrosion process of reinforcement in concrete, a computer program CHLODIF was developed [3.2,3.5]. The program computes the required initiation period (Fig. 3.1) through the analysis of chloride ions diffusion in a semi-infinite medium.



**Fig. 3.1** Resulting of computing corrosion process obtained by CHLODIF program

### 3.3 Period of Propagation of Reinforcement Corrosion in Concrete

The period of propagation of reinforcement corrosion in concrete,  $t_1$  has previously been seldomly taken into account in the life-span analyses of reinforced concrete structures, especially of those structures where the reinforcement corrosion was initiated by chlorides.

The period of propagation calculated according to the criterion of the corrosion limit state can be significantly longer than what has been considered to be in the past. It can be computed using the algorithm for evaluation of the corrosion rate. There are numerous mathematical and empirical models that describe the corrosion rate. The model adopted in the corrosion limit state design is one of the most commonly used ones. It can be defined as [3.7]:

$$\phi(t) = \phi_i - 0.023 \cdot i_{\text{cor}} \cdot t \quad (3.7)$$

where:

$\phi(t)$  - the reinforcement diameter at time  $t$  (mm),

$\phi_i$  - the initial reinforcement diameter at time  $t = 0$  (mm)

0.023 - conversion coefficient ( $\mu\text{A}/\text{cm}^2$ ) in (mm/year),

$i_{\text{cor}}$  - corrosion current density ( $\mu\text{A}/\text{cm}^2$ ); for newly built structures values are given in

Table 3.3 [3.8-3.10].

$t$  - time passed since the corrosion process initiation (years).

**Table 3.3** The corrosion current density values

ENVIRONMENTAL HUMIDITY (%)	100	70-100	70
$i_{\text{cor}}$ ( $\mu\text{A}/\text{cm}^2$ )	10	1	0.1

Stated values of corrosion current density take into account only the influence of the environmental relative humidity, while the actual value of  $i_{\text{cor}}$  depends also on numerous other



factors. Therefore, it is proposed to modify equation (3.7) by introducing the correction coefficient of corrosion current density in the following way [3.2]:

$$\phi(t) = \phi_i - 0.023 \cdot p \cdot i_{\text{cor}} \cdot t \quad (3.8)$$

where:

$p$  - correction coefficient of corrosion current density,  $p = p_i$  where values  $p_i$  for various factors are given in Table 3.4 [3.2].

**Table 3.4** Values of the corrosion current density coefficient

$p_i$	INFLUENCE
2	Sulfates
1 - 15	Carbonation
1 - 20	Cracks

The correction coefficient of corrosion current density takes into account effects that directly contribute to the reduction of the reinforcement bar cross-section.

There are many other factors that do not directly affect the corrosion process, but contribute to an increased failure probability of the bearing capacity of the design cross-section where the corrosion process is in progress. These factors cannot be quantified, but they can be included in a factor of safety which modifies equation (3.8) into the following final form [3.2]:

$$\phi(t) = \phi_i - \gamma_{\text{gsk}} (0.023 \cdot p \cdot i_{\text{kor}} \cdot t) \quad (3.9)$$

$\gamma_{\text{gsk}}$  - corrosion limit state safety coefficient.

Although the exact influence of the factors included in the factor of safety cannot be expressed numerically, their contribution can be estimated. The fuzzy set theory has proved to be an adequate procedure for that estimation [3.2].

The fuzzy sets theory was established by Zadeh [3.11] in 1965, because of lack of a classical mathematical apparatus for algorithm definition of certain systems' responses to input stimuli (see Appendix A). Application of this theory allows for regulation of the system's control by series of the rules, which simulate human (experienced) behavior. This theory of fuzzy sets reached its peak in practice in eighties and its quick development to date has resulted in amazing achievements concerning supervision of even the most complex processes. Since the description of several additional factors' influences is qualitative (greater or minor), the calculation according to the fuzzy sets theory is taken as the most appropriate procedure in the safety coefficient definition.

### **3.4 Sets of Influences and Consequences**

The elements of fuzzy sets influences are verbal expressions about certain influences on a designed section's bearing failure. The elements of fuzzy sets consequences are categories that include significance of corrosion consequences on the bearing capacity of the same section. Fuzzy sets of influences and consequences are described in Table 3.5. For a simpler analysis these set elements are grouped according to common properties in sets and subsets. Assessment of an appropriate selection of influences of elements, as well as definition of the respective consequences is a very complex procedure which requires a final property expertise or more of them, if necessary. Therefore, only a suggestion is provided in the report on how the functions could possibly look like and what membership of the consequence categories to the selected set of element influences is possible.

To define the membership functions, first it is necessary to define a relevant variable  $x$ . The bearing failure occurs due to a real reduction of the reinforcement area by corrosion (with a greater or minor probability, enhanced by additional factors). Therefore, the variable  $x$  is suggested to be taken as a ratio of the reduction of the cross-section area and the initial area of reinforcement (For a single bar the condition is that the same reinforcement profile is used in the observed section). The value of up to 10% is cited in literature as the greatest allowed section area reduction (as in a consequence of corrosion). Additional factors that contribute to the

bearing failure, are considered in the report as well. The reduction limit due to them may be increased, but up to a value of 25% at most [3.6].

In all further calculations, a value of 20% [3.3] for the greatest possible area reduction of the reinforcement section is adopted from the bearing failure safety conditions. To facilitate calculation and application of the formerly calculated values, the variable  $x$  is finally presented as:

$$x = \frac{\Delta\phi}{\phi_i}, \quad (3.10)$$

where

$\Delta\phi$  - reduction of reinforcement diameter,

$\phi_i$  - initial value of reinforcement diameter.

The reinforcement section area reduction of 20% corresponds to a diameter reduction of approximately 10%. Then, the variable  $x$  takes values within an interval from 0 to 0.1 (including values 0 and 0.1). To make calculation of the safety limit corrosion state possible by means of the fuzzy sets theory, the variable  $x$  will be discretized and hence, the following values will be observed: 0.00, 0.01, 0.02, 0.03, 0.04, 0.05, 0.06, 0.07, 0.08, 0.09, 0.10. It is assumed that all relevant fuzzy sets of influences and consequences may be expressed by means of the variable  $x$ , just like fuzzy sets for which the universal set  $X = \{0.00, 0.01, 0.02, 0.03, 0.04, 0.05, 0.06, 0.07, 0.08, 0.09, 0.10\}$ . The membership functions of the observed fuzzy sets are given in Table 3.6. The membership function values are suggested on the basis of experience acquired to date.

**Table 3.5** Elements of fuzzy sets of influences and consequences

<b>INFLUENCES - U</b>		
<b>CONSTRUCTION PARAMETERS</b>		
STRUCTURE SIGNIFICANCE	COMPUTING ADOPTED MODEL	TYPE OF STRUCTURE ELEMENT
KZ1 – secondary KZ2 – main KZ3 – primary	KM1 - adequate KM2 - simplified KM3 - approximate	KE1 - non bearing KE2 - group bearing KE3 - single bearing
DESIGNED CROSS-SECTION	TECHNOLOGY OF CONSTRUCTION	ERRORS IN DESIGN AND CONSTRUCTION
KP1 – distant KP2 – near KP3 – critical	KT1- prefabricated KT2- in form work, continuous KT3- in form work, construction joints, slipform	KI1 - hardly probable KI2 - probable KI3 - almost certain
MATERIAL SIGNIFICANCE	SPECIAL ENVIRONMENT CONDITION	ACTION OF OTHER SDL*
G1 - very well G2 – well G3 – average G4 – bad G5 - very bad	O1 - very favorable O2 - favorable O3 - average O4 - unfavorable O5 - very unfavorable	SLD1 - without load SLD2 - a little load SLD3 - average load SLD4 - great load SLD5 - very great load
	<b>CONSEQUENCES - P</b>	
	P1-without consequences P2 - a little P3 - significant P4 - great P5 - critical	

\*SLD - special durable loads

**Table 3.6** The membership functions of the observed fuzzy sets of influences and consequences

ordinal number	1	2	3	4	5	6	7	8	9	10	11
X	0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09	0.10
$m_{KM1}(x)$	1	1	0.9	0.7	0.5						
$m_{G1}(x)$	1	1	0.9	0.8	0.7						
$m_{O1}(x)$	1	1	0.9	0.8	0.6						
$m_{PTO1}(x)$	1	1	0.9	0.8	0.6						
$m_{P1}(x)$	1	1	0.9	0.7	0.5						
$m_{KZ1}(x)$	0.8	0.9	1	0.9	0.8	0.6					
$m_{KM2}(x)$		0.9	1	0.9	0.8	0.6					
$m_{KE1}(x)$	0.8	0.9	1	0.9	0.8	0.6					
$m_{KP1}(x)$	0.8	0.9	1	0.9	0.8	0.6					
$m_{KT1}(x)$	0.7	0.9	1	0.9	0.8	0.6					
$m_{KI1}(x)$	0.7	0.9	1	0.9	0.8	0.6					
$m_{G2}(x)$		0.9	1	0.9	0.8	0.5					
$m_{O2}(x)$		0.8	1	0.9	0.7	0.6					
$m_{PTO2}(x)$		0.7	0.8	1	0.8	0.7					
$m_{P2}(x)$		0.9	1	0.9	0.8	0.6					
$m_{KM3}(x)$			0.6	0.8	0.9	1	0.9	0.6	0.4	0.2	0.1
$m_{KE2}(x)$			0.6	0.8	0.9	1	0.9	0.6			
$m_{KT2}(x)$			0.6	0.8	0.9	1	0.9	0.6			
$m_{KI2}(x)$			0.6	0.8	0.9	1	0.9	0.6			
$m_{G3}(x)$			0.5	0.7	0.9	1	0.9	0.7			
$m_{O3}(x)$			0.5	0.6	0.8	1	0.8	0.6			
$m_{PTO3}(x)$			0.4	0.5	0.7	0.8	1	0.9			
$m_{P3}(x)$			0.6	0.8	0.9	1	0.9	0.6			
$m_{KZ2}(x)$					0.6	0.7	0.8	0.9	1	0.9	
$m_{KP2}(x)$					0.6	0.7	0.8	0.9	1	0.9	
$m_{KT3}(x)$					0.6	0.7	0.8	0.9	1	0.9	0.8
$m_{KI3}(x)$					0.6	0.7	0.8	0.9	1	0.9	0.8
$m_{G4}(x)$					0.6	0.7	0.8	0.9	1	0.9	
$m_{O4}(x)$					0.5	0.6	0.7	0.9	1	0.8	
$m_{PTO4}(x)$					0.5	0.6	0.7	0.8	0.9	1	
$m_{P4}(x)$					0.6	0.7	0.8	0.9	1	0.9	
$m_{KZ3}(x)$							0.7	0.8	0.9	1	1
$m_{KE3}(x)$							0.7	0.8	0.9	1	1
$m_{KP3}(x)$							0.7	0.8	0.9	1	1
$m_{G5}(x)$							0.7	0.8	0.9	1	1
$m_{O5}(x)$							0.5	0.7	0.8	1	1
$m_{PTO5}(x)$							0.5	0.7	0.8	0.9	1
$m_{P5}(x)$							0.7	0.8	0.9	1	1

### 3.5 Probability Bearing Failure Sets

The mentioned coefficient  $\gamma_{gsk}$  (corrosion limit state safety coefficient, equation (9)) is taken as a relevant variable  $y$  in determination of the corrosion limit state probability when discretization is performed using values 1.0, 1.2, 1.4, 1.6, 1.8, 2.0. With their bearing failure probability described as: V1 - very small, V2 - small, V3 - significant, V4 - great, V5 - very big, the fuzzy sets V1, V2, V3, V4, V5 are expressed by means of variable  $y$ . It means that fuzzy sets' universal set  $Y = \{1.0, 1.2, 1.4, 1.6, 1.8, 2.0\}$ . The respective membership functions are presented in the Table 3.7 on the basis of the acquired experience, the same as for Table 3.6.

**Table 3.7** The membership functions of the fuzzy sets of corrosion limit state safety coefficient

ordinal number	1	2	3	4	5	6
<b>Y</b>	1	1.2	1.4	1.6	1.8	2
$m_{V1}(x)$	0.1	0.3	0.5	0.7	0.9	1
$m_{V2}(x)$	0.2	0.3	0.4	0.5	0.6	0.7
$m_{V3}(x)$	0.9	0.8	0.7	0.6	0.5	0.4
$m_{V4}(x)$		1	0.8	0.6	0.4	0.2
$m_{V5}(x)$		1	0.9	0.8	0.6	0.1

### 3.6 Safety Coefficient Calculation

For any concrete case, firstly the relevant influences  $U_i$  ( $i=1, \dots, m$ ) and the respective consequences  $P_j$  ( $j=1, \dots, n$ ) (according to the Table 3.5 and Table 3.6) are selected and the corresponding relationship  $Z_{ij}$  between them is established. This is obtained in a way that the membership function of relationship  $Z_{ij}$  is defined as follows:

$$m_{Z_{ij}}(x_1, x_2) = \min \{m_{U_i}(x_1), m_{P_j}(x_2)\}, \quad x_1, x_2 \in X \quad (3.11)$$

To take the mutual effect of all the selected elements into consideration, all the relations will be combined, that is the fuzzy set  $Z$  will be obtained from the following respective membership function:

$$m_Z(x_1, x_2) = \max_{ij} \{ m_{Z_{ij}}(x_1, x_2) \}, \quad x_1, x_2 \in X \quad (3.12)$$

To establish a relationship between the selected sets of consequences and sets of bearing failure probabilities, the relationships  $R_{jk}$  ( $k = 1, p$ ) are defined. Hence, their respective membership functions are as follows:

$$m_{R_{jk}}(x, y) = \min \{ m_{P_j}(x), m_{V_k}(y) \}, \quad x \in X, y \in Y \quad (3.13)$$

The global effect is obtained by combining all of the above mentioned relationships, e.g. by observing fuzzy set R using the following respective membership function:

$$m_R(x, y) = \max_{j,k} \{ m_{R_{jk}}(x, y) \}, \quad x \in X, y \in Y \quad (3.14)$$

A limit state corrosion safety coefficient will be expressed like fuzzy set K obtained from a composition of relations Z and R ( $K=Z \circ R$ ), using the following respective membership function:

$$m_K(x, y) = \max_{x_2 \in X} \{ \min \{ m_Z(x_1, x_2), m_R(x_2, y) \} \}, \quad x \in X, y \in Y \quad (3.15)$$

On the basis of the obtained membership function of set K, it can be stated that the safety factor is very small, small, mid, great or very great. This statement certainly anticipates that the membership functions respective to the mentioned descriptions are determined in advance. Hence, the obtained membership function will be compared to the determined ones, and the final judgement about the safety factor will be given. If concrete value  $\gamma_{gsk}$  for the limit corrosion state safety factor is sought, the  $y_M \in Y$  will be taken for which the following relation is relevant:

$$m_K(x, y_M) = \max_{x,y} \{ m_K(x, y) \} \quad (3.16)$$

It may happen that several different values  $y_M$  are obtained. This is in accordance with the fuzzy sets theory.

The ratio of the reduced ( $\Delta\phi$ ) and initial values of the bar diameter is adopted as a variable in the procedure to define function of the corresponding fuzzy sets elements.

$$x = \frac{\Delta\phi}{\phi_i}, \quad (3.17)$$

where:

$$\Delta\phi = \phi_i - \phi(t), \text{ and}$$

$\phi(t)$  is derived from equation (3.8).  $\phi(t)$  takes into account direct factors that reduce the area of the reinforcement cross-section. Concerning the factor of safety value, the following possible values are proposed [3.2]:

$$\gamma \in \{1, 1.2, 1.4, 1.6, 1.8, 2\}$$

Finally, to define the propagation period of reinforcement corrosion processes in equation (3.9), it is necessary to determine the maximum allowed reduction of the bar diameter,  $\phi_u$ . The failure condition of the designed cross-section is fulfilled when the maximum bar diameter reduction is reached. According to data presented in [3.8-3.10], a reduction of the reinforcement bar cross-section area of 20% can be considered as the ultimate one, with the respective bar diameter reduction of 10%. This leads to the following expression:

$$\phi_u = 0.9 \phi_i \quad (3.18)$$

The period of propagation can be finally determined from the condition of equality between the bar diameter at time  $t_1$  and the ultimate diameter:

$$\phi(t_1) = \phi_i - \gamma(0.023 \cdot p \cdot i_{\text{corr}} \cdot t_1) = \phi_u = 0.9 \cdot \phi_i$$

$$t_1 = \frac{0.1 \cdot \phi_i}{\gamma(0.023 \cdot p \cdot i_{\text{corr}})}. \quad (3.19)$$

The calculated service life of a structure,  $t_s$ , is finally obtained from the sum of the period of propagation,  $t_1$ , and the period of initiation,  $t_0$ .



### 3.7 Design Procedure

The design procedure, according to the criterion of the corrosion limit state, is described below. The period of initiation and the period of propagation of reinforcement corrosion are computed and compared to the designed service life of a structure. If the condition:

$$t_0 + t_1 = t_c \geq t_p \quad (3.20)$$

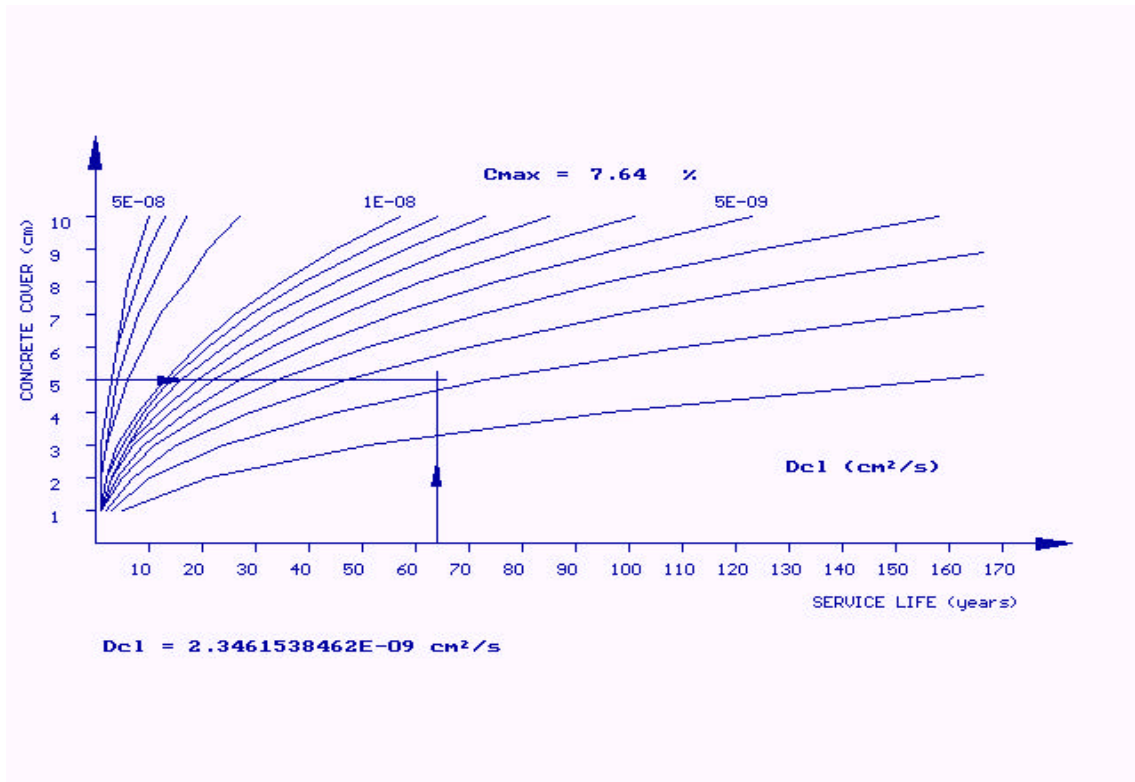
is fulfilled, the design procedure is completed. Otherwise, it is necessary to design the section of interest in one of the following ways:

*a) By extending the period  $t_1$*

This can be achieved by either reducing the factor of safety (reevaluating elements of fuzzy sets of influences), or by reducing the correction coefficient of corrosion current density (modifying the way of reinforcement development, size of a chosen bar diameter, or the spacing between bars, that may result in a reduction of the crack width in concrete). Most of the time a change in the period  $t_1$  will not be sufficient to satisfy the required condition.

*b) By extending the period  $t_0$*

This can be achieved by improving the quality of the materials, primarily of concrete, or by changing the dimensions of the design concrete cover. If the corrosion is caused by chlorides, it is necessary to increase the diffusion coefficient  $D_{Cl^-}$ . Afterwards, concrete quality and the thickness of the concrete cover,  $c$ , can be determined from a nomogram, called C-D-c-t nomogram [3.4]. For a required period  $t_0$ , values of  $c$  and  $D_{Cl^-}$  can be determined, or the period  $t_0$  can be determined for  $c$ ,  $D_{Cl^-}$ , or for both known (Fig. 3.2). If intervention in material properties or in the concrete cover thickness still cannot satisfy the condition given by the corrosion limit state criterion, the diffusion process of chloride ions can be additionally slowed down by placing silicone coatings. This will reduce the time necessary for the change of the initial concentration of chloride ions. Finally, the process can be completely discontinued by applying the cathodic protection or corrosion inhibitors [3.12,3.13].



**Fig. 3.2** C-D-c-t nomogram

The final result of the design process, according to the corrosion limit state criterion, is the time schedule for the maintenance of a structure. An example is given in Table 3.8. The procedure is applied in this case to the design of flanges of a bridge arch. The procedure secures the service life of the structure within the designed period of 100 years [3.14-3.16].

**Table 3.8** Time schedule of the structure's maintenance

<b>t (years)</b>	<b>ACTIVITY</b>
0	completion of structure construction
21	concrete surface spraying with mobile corrosion inhibitors
37	initiation of mobile inhibitors' effect
39	reaching the critical chloride ions concentration on reinforcement, applying of second layer of mobile inhibitor
64	end of mobile inhibitors' effect, corrosion initiation if the second layer of mobile inhibitor is not applied
100	fulfilled conditions of ultimate corrosion limit state, if the second layer of mobile inhibitor is not applied

### 3.8 Conclusions

A mathematical model for the durability design of reinforced concrete structures that insures bearing conditions during the entire service life of a structure, is proposed. Unlike the existing methods, the time framework of the calculation is extended to the period of propagation of the reinforcement corrosion process in concrete. This contributes considerably to an increase of the service life. The model is based on the proposal of supplementing the existing design procedure, based on the criteria of serviceability limit states. The new criterion insures that the bearing capacity of the cross-section under a condition of simultaneous action of traditional and environmental loads is retained. Environmental loads incorporate durability factors, which can be expressed either qualitatively or quantitatively, and thus are included into the design procedures. Environmental loads influence mostly changes in the active bearing capacity of the reinforcement cross-section. The critical variable that controls the structural failure in marine conditions is corrosion due to chlorides. The result of the proposed design procedure, according to the corrosion limit state criterion, are the initiation and propagation periods of corrosion process, i.e. the structure's service life and its maintenance time schedule within the entire designed service life. Finally, dimensioning according to the corrosion limit

state criterion often reduces the amount of reinforcement, in comparison to the amount necessary according to the conditions of the cracking limit state.

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## **4 Concrete Production by Means of Knowledge Supported Mix Design**

There are two characteristic phases in the mix design of concrete: laboratory tests and redesign of concrete proportioning on the basis of control testing. In the first phase, the concrete composition is predicted on the basis of the fundamental rules of concrete technology, and then the properties of a fresh mixture and hardened concrete are tested in the laboratory. Proportions of the components necessary for the beginning of production are obtained by application of mathematical methods on the basis of experimental data [4.1].

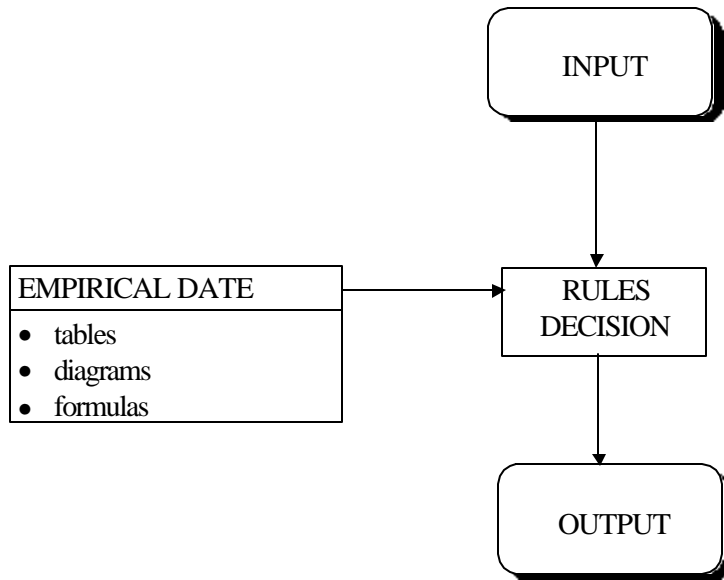
Concrete is certainly one of the most important construction materials. Many properties of concrete structures depend significantly on the properties of the concrete placed. On the other hand, concrete is a complex material, produced out of different components according to specific mix designs and technologies. It is therefore the basic task of a concrete technology specialist to design such a composition and concrete production technology to meet already determined concrete properties. In that process, main difficulties occur due to the fact that physical and chemical links between properties of the individual concrete components, external conditions during the production process, and the ultimate concrete properties, are extremely complex and intricate. Thus, the actual scientific knowledge cannot present them in a form of exact mathematical formulas [4.2-4.5], i.e. numerical algorithms that would predict concrete properties on the basis of already known properties of components and the technological process. That is why the design of concrete composition is presently based on a large number of empirical facts (various tables and graphs), actual laboratory tests, and significantly on the knowledge and experience of a concrete technology expert.

Due to numerous data obtained by the components' control testing and due to complexity of their influences, a decrease in quality is sometimes observed during a concrete quality control, but only after 28 days. During a production in progress, properties of components and climate conditions significantly change and affect the concrete's workability and the compressive strength. On the basis of components' control testing, the concrete production is controlled in

a way that the quality of components must be in the range of determined and required limits. However, a variation of the properties of a fresh and a hardened concrete, although within the required limits, sometimes results in the compressive strength failure, or in an overly high, non-economic, average design compressive strength. Taking into account so many variable data, that require quick changes in the composition, it is very difficult to control the quality. Recently, several authors have suggested a formulation and solution of the procedures for the concrete mix design by means of a computer aided mathematical modeling [4.2-4.7]. As a result, different programs have been developed [4.3,4.6], which enable more exact, more reliable and quicker decision making in individual stages of the mix design and concrete production. Namely, the algorithmic programs make the empirical knowledge (tables, formulas, etc.) memorized in the computer, and transform a part of the expert's knowledge into formal decision rules. In that way, an approximate solution may be reached, and even improved later by laboratory tests.

#### **4.1 Computer Aided Mix Design of Concrete (CAMDOC)**

A theoretical model of the computer aided mix design of concrete has been developed on the basis of basic concrete properties. The concrete mix design is based on assurance of project conditions, required workability, concrete compressive strength, and durability. This resolving way has directed the research and resulted in sophistication of the information system CAMDOC for the computer aided mix design. The advantage of using a computer in daily activities is in simple procedures. The ability to store data for a later use represents a special advantage. Computer Aided Mix Design of Concrete runs on a personal computer. The software tools used in the development of the CAMDOC system are Clipper and Turbo-Pascal. For all the computing, the information system includes 30 empirical databases and decision rules in the solution of the problem (Fig. 4.1). Basic steps of the mix design of concrete are described in the flowchart in Fig. 4.2. On the basis of a project design, structural elements information, environment, concrete reinforcement, bar spacing, and than about the contractors, equipment and workmanship are to be collected.

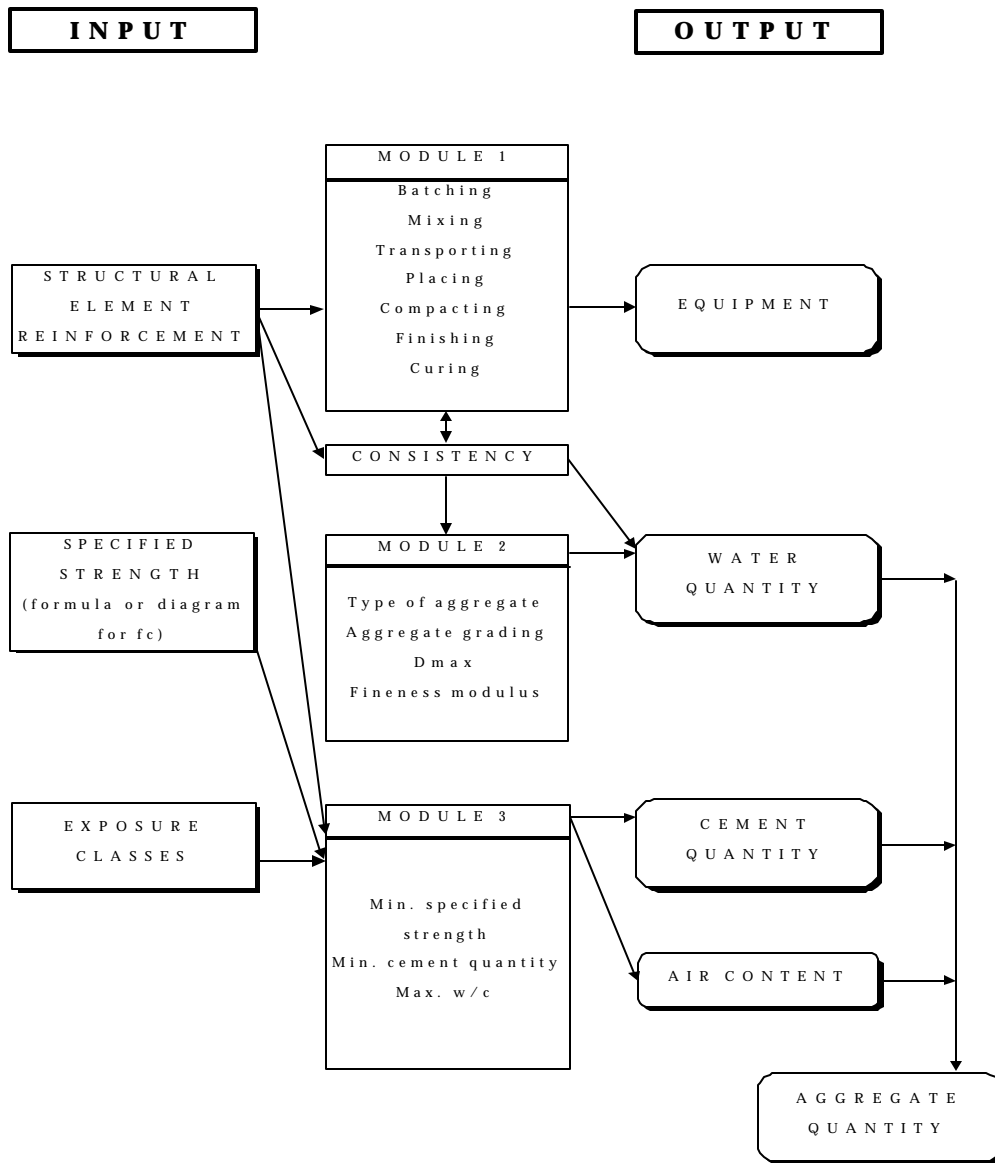


**Fig. 4.1** Flowchart 1 of program CAMDOC

From research and empirical testing results relationships between the properties of concrete components and the criteria of concrete quality control were established in a form of databases [4.2]. The implementation of these relations is presented in Fig. 4.2, while details of these relations are described by modules 1, 2 and 3.

All properties can be considered at three levels: required, designed and real (experimental). The aim presented in this model (Fig. 4.1, 4.2) is to obtain desired properties. The workability of concrete is designed by the aid of two modules. Module-1 describes equipment for concrete placement (Fig. 4.3) while module-2 describes a computer method to achieve a desired grading. According to these two modules, the database of a current project is expanded by an adequate consistency, the type of aggregate fractions, proportions of aggregate fractions, type of grading curve, fineness modulus, and then by the required water content.





**Fig. 4.2** Flowchart steps in CAMDOC

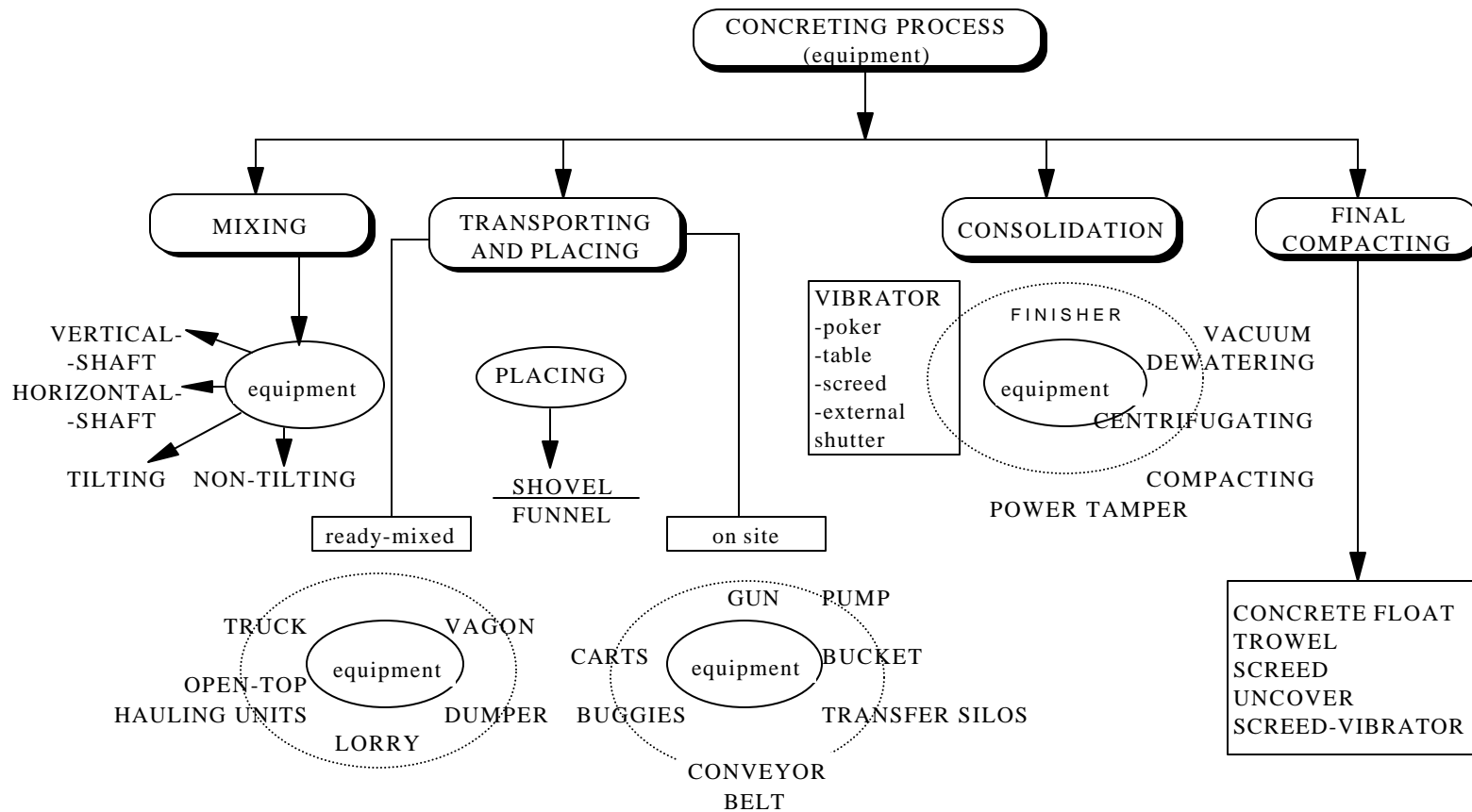


Fig. 4.3 Flowchart 3 MODULE 1

A computer method to achieve a desired grading is a combination of various fractions of aggregate as to get optimum grading of aggregate. It is an important step in mix design of concrete. Module-2 of CAMDOC is a computer routine by which the number and quantity of aggregate fractions are combined in the best way to achieve the desired grading composition.

Design of compressive strength and durability of concrete are included in Module-3. From the structures design, the specified strengths, and criteria for concrete components, as well as criteria for designed compressive strength of concrete are known. The assurance of concrete durability parameters is implemented through criteria where exposure classes are connected to a specified strength, cement content and water cement ratio. Popovics [4.9] formula or Walz [4.13] present the relation between the water cement ratio and characteristic compressive strength curves. The cement class and quantity can be chosen out of them. Then, by subtracting water content, cement content and air content from the whole volume, the aggregate content is calculated.

## 4.2 Basic Relations

The procedure for a concrete mix design that has been used as a model for CAMDOC [4.3], and is described herein, is based on three fundamental relationships:

- Relationship between the water required and the desired consistency
- Relationship between the compressive strength and the water cement ratio
- Relationship between the required concrete durability and CEB-FIP Model Code 90 requirements.

Kluge has demonstrated [4.7,4.8] that the quantity of water requirement in concrete consists of the quantity required for cement standard consistency and the quantity to keep the surface of the aggregate wet.

$$W = SC \cdot C + A_t, \quad A_t = \sum \frac{a_{ti}}{100} \cdot A_{ti} \quad (4.1)$$

where:

$W$  - water requirement [4.2],

$SC$  - standard consistency of cement (percentage),

$C$  - cement quantity (kg),

$a_{ii}$  – aggregate retained between two sieves (kg) (proportion of each fraction in cumulative aggregate),

$A_{ii}$  - water requirement fraction of aggregate on each sieve in percentage of aggregate weight.

Kluge has experimentally determined quantity of water requirements of each fraction ( $A_{ii}$ ) for the corresponding aggregate fineness modulus (Table 4.1). Although these quantities are not the same for all the aggregates, because they depend on the shape, texture and mineralogical composition, they can be used as a reference for all the aggregates.

**Table 4.1** Kluge's experimentally results of water quantity requirements of each fraction ( $A_{ii}$ ) for the corresponding aggregate fineness modulus

SIEVE (mm)	0-0.25	0.25-0.5	0.5-1	1-2	2-4	4-8	8-16	16-32
$A_{ii}$ (%)	21.89	10.19	6.42	4.53	3.77	2.98	2.23	1.55

Popovics extended and modified Abrams formula ( $f_c = \frac{A}{B^{W/C}}$ ) by means of multiple nonlinear regression model and obtained the following [4.9]:

$$f_c = \frac{A}{B^{W/C + 0.00063T + 0.279t}} \quad (4.2)$$

where:

$f_c$  - 28-day compressive strength,

$W/C$  - water cement ratio by weight,

$C$  - cement content,

$a$  - air content volume,

$A, B$  - constants depending on aggregate and cement.

The equation clearly shows, how the strength rate is increased as the water cement ratio is decreased due to an increase of cement (cement paste) quantity. Analyzing a given data set, Popovics got  $A=353.72$  and  $B=23.66$  [4.9]. By supplying data for specific cements, aggregates, admixtures and compressive strengths, the values of the equation parameters can be by feedback, step by step corrected. The third basic relation for CAMDOC is based on the tables from CEB-FIP Model Code 90 [4.9]. There, environmental conditions for concrete are classified in 5 classes and 9 subclasses (1-dry; 2: a-humid (without frost), b-humid (with frost); 3-humid (with frost and de-icing agents); 4: a-sea water (without frost), b-sea water (with frost), 5: a-slightly aggressive chemical environment, b-moderately aggressive chemical environment, c-highly aggressive chemical environment. Besides, a new class 6: aggressive mechanical concrete exposure, has been added to CAMDOC. For each exposure class in CAMDOC there are conditions for the minimum characteristic compressive strengths, cement quantity, the maximum water cement ratio and required entrained air. If concrete mixes obtained by the laboratory procedure (one-month process) are compared to the CAMDOC ones (several minutes), no significant differences in the concrete mix component quantities are obtained.

Design by means of the CAMDOC program have made the mix design of concrete quicker and better because in several minutes the type and quantity of concrete components, and equipment for concrete placing, can be obtained. Every change in the mix can be quickly evaluated at each step of the process by CAMDOC, new composition of concrete can be quickly calculated. Preference of computers is in a quick calculation, as well as in the possibility of connecting the concrete component data with the composition and the properties of concrete itself. All the data are memorized and stored on magnetic media, and thus will be of major help in the future mix design. Properties of the components are connected with the

compressive strength. Popovics formula of interdependence of concrete strength to w/c ratio, cement quantity and quantity of air, is used.

During the concrete production, many changes occur and the production is permanently based on the composition obtained earlier by means of the laboratory tests, which sometimes means a deviation from the reality. Due to that fact, the concrete mix should be constantly redesigned during production with respect to any change in the production process (types and quantities of the components). This is the great advantage of using the Computer aided mix design of concrete (CAMDOC). Mix design of concrete was based on the by assurance of the conditions for the structure's design, assurance of the required workability and concrete strength, and durability assurance. CAMDOC gives a possibility of simple and quick reaction to all the variable changeable values in concrete proportion, equipment and concrete production, and all of that to assure the concrete quality control. Its application reduces the scope of preliminary laboratory tests, as well as time necessary for their performance, and it enables quick implementation of quality control results to the correction of components' proportion during concrete production. The future of such mix design of concrete is in connection of decision and knowledge bases, and databases of cement, aggregate and concrete as well [4.14].

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## 5 Concrete Transport Properties

### 5.1 Concrete Structure

Concrete is a heterogeneous composite material, a mixture of cement, water and aggregate, where chemical reactions between the cement and water harden and bind the inert aggregate. Additional constituents are concrete admixtures and air, either already present or artificially entrained. In the hardened concrete, the cement paste has two main tasks: to fill the space between the aggregate grains and to create an impermeable mass with them, and to bind the aggregate grains and give the necessary strength to the concrete. The aggregate is just an inert fill in the cement paste matrix.

Mutual interaction of concrete constituents and interaction with the environment can be distinguished on various levels of structural complexity. Three observation levels are commonly used for concrete: micro, mezzo, and macro level [5.1]. The most elementary, or the primary level, is the micro level (atomic-molecular) of the structural composition of concrete constituents. On this level properties of a hardened cement paste are observed. Kondo and Daimon (1980) proposed a model of a hardened cement paste, distinguishing pores between the gel particles, pores between the crystallites, and pores in the crystallites. The following cement properties can be noticed: solubility in water, ability to react with the water action or to resist it, thermal stability within working temperatures, and attraction of various chemical compounds. These are in fact chemical properties of the material, defining its resistance to the aggressive components of the environment. On the mezzo level, the composite structure of the concrete is considered. The real structure can be presented as a porous structure and the matrix with randomly distributed aggregate grains. The hardened cement paste is a porous material with a wide range of pore size distribution. Within a porous matrix there are grains of a very variable size and shape. On the mezzo level, pores and aggregate should be modeled. On the mezzo level, concrete properties determine its ability to resist numerous aggressive influences, including freezing and thawing. Results obtained on



diffusion and the

process. In the case of diffusion, there is a combined influence of the following gradients:

- various solution concentrations causing osmotic pressure

- moisture concentration, influencing surface forces
- temperature differences

## 5.2 External Influences

*Water* can be considered as an integral part of concrete (pore water) and as an external influence (rain, etc.). In the hardened cement paste, water can be found in several states. Although there is no sharp line, the following classification is suitable:

- Chemically bonded water is a part of water bound by hydration process into hard compounds, making up a cement gel.
- Interlayer water penetrates between layers of a hard gel or intercrystalline space, like in clays. The depth of such interlayer is only about a nanometer. By removing the water these spaces close and the hardened cement paste contracts.
- Adsorbed water is the water tied to the gel surface by surface forces. In the first layer, which is several water molecules thick, these forces are very strong, so that this part is under a high pressure. With an increased distance these forces rapidly decrease.
- Free water is found in the capillaries and in large gel pores. It is sufficiently distant from the gel surface and is free from surface forces.
- Besides water, concrete voids contain air with a certain amount of vapor, depending on the temperature and air pressure.

*Temperature.* Concrete deterioration due to low temperature results from a change in the water volume when water freezes. In larger capillary pores, freezing starts at 0 °C, but in smaller pores, the temperature needs to be much lower before the water starts freezing. For example in the smallest cement gel pores, water freezes at -78 °C.

*Salt.* To de-ice the concrete surface, salts used are chlorides of sodium, magnesium or calcium, as well as urea. The salts have a two-fold action on the cement matrix: physical and chemical. Crystallizing pressure, which has a destructive effect on the cement matrix, is the physical action. The chemical action is expressed by a chemical bond of the cement mineral (tricalcium aluminate) and the chloride, resulting in Friedl salt, which by crystallizing,

changes the pore structure of the concrete. It is generally agreed that the salt action is a physical rather than a chemical phenomenon. Salt concentration in concrete plays an important role. The most dangerous concentration is a one of a moderate range (2-4%).

### **5.3 Mechanisms in Concrete**

Concrete behavior during freezing and thawing depends on the concrete pore structure, water content, and temperature, presents schematically causes of damage and the resulting mechanisms. When water freezes, the volume increases by 9% and the capillary water comes under hydraulic pressure. If the hardened cement paste does not contain air pores (air compresses easily and this absorbs capillary pressure), water pressure acts on the hardened cement paste structure and expands it. When the pressure is too great, the bonds break and the hardened cement paste structure is destroyed. Thawing of concrete provides for the absorption of new water quantities in the concrete, and its freezing further destroys the concrete. Increased ice volume is accompanied by shrinkage of hard material, cement gel and aggregate, due to a lower temperature. Obviously, concrete destruction appears only when concrete is saturated. If concrete is fully saturated ( $S=100\%$ ), the hydraulic pressure formed by an ice forming in the capillaries cannot be absorbed within the material and will inevitably bring about damage due to freezing.

Accordingly, a water-saturated concrete will be quickly damaged by frost, and a relatively dry concrete will be not harmed by a low temperature. This phenomenon is often encountered in practice. It has been mentioned that air pores in the hardened cement paste can serve to absorb the pressure appearing in the capillaries due to ice expansion. This phenomenon is utilized in the air-entrained concrete technique. It enables water to penetrate into air pores near the edge of capillaries or voids due to a pressure. The pressure is released fast enough, unless the hardened cement paste is too impermeable, the air pores too distant, and freezing is too rapid. In such a favorable case, instead of expansion, there is a contraction of concrete due to temperature. The entrained air pores have a favorable effect because they break the

capillaries, and thus increase the water impermeability of concrete. The decrease of concrete volume is the consequence of the contraction of the hardened cement paste and aggregate due to temperature. It is also influenced by the gradual passage of water from the cement gel into capillaries, or the capillary effect, a process identical to concrete shrinkage due to drying. But in this case, the water movement is due to a loss of the thermodynamic balance when the capillary water freezes and thus increases the surplus of free water energy in the gel pores. The water movement increases also because of an osmotic pressure, caused by a salt concentration in a still unfrozen capillary water (only clean water freezes). This is one of the main reasons why the presence of salt reduces freezing. This, however, is not the only form of the harmful effect of deicing salts. There is a series of other physical and chemical mechanisms and the reactions accelerating the concrete damage. There is a pressure caused by salt crystallization in larger pores above a definite solution concentration. On the other hand, dissolving of some salts, such as sodium chloride and urea cause additional cooling of concrete, i. e. a temperature shock, accompanied by an increase in the temperature gradient and stresses in the concrete.

The natural process of ice thawing without salt or a frozen salt solution has no temperature shocks, just like a natural freezing of water and salt solution. An increase in the chloride content lowers the water freezing point. Different concentrations at different depths result in subsequent freezing of the inside layer, so this is commonly called a “freezing layer by layer”. With respect to results related to the influence of the shock-caused temperature gradient on concrete freezing and thawing, the following conclusions can be given:

- A high temperature gradient caused by a temperature shock can be expected, as a rule, in the first five minutes after application of deicing salts, meaning its action is very soon.
- Destructive stresses increase only in a relatively narrow outer concrete zone, several millimeters thick.
- Internal tensile stresses can range between  $1-4 \text{ N/mm}^2$ , that is equal to the concrete tensile strength. Such repeated stresses can lead to micro cracks in concrete and to a further damage of the concrete surface.

An additional cause of concrete damage can be the difference in deformation of the hardened cement paste and aggregate due to temperature. The greater the difference of their expansion, the greater the danger of concrete damage.

## **5.4 Transport**

### **5.4.1 Microstructure of Concrete**

A close examination of concrete reveals a very rich and complex microstructure [5.2]. It is comprised of a variety of phases, with diverse material properties. For simplicity, concrete can be thought composed of cement paste, sand and aggregate. Each phase can be characterized by different length scales, which correspond to a typical pore diameter, the size that may span over many decades from nanometers to millimeters. Firstly, the cement paste is considered at the smallest scale, where it is largely composed of a calcium silicate hydrate (C-S-H) gel that is comprised of nanometer size C-S-H particles and pores. At the micrometer scale, the cement paste and aggregate (sand and stones) form a composite material, which, in addition, may contain air voids (at the millimeter scale). Another feature that may be present in concrete is the so-called interfacial zone. The interfacial zone is a region of an order of 10-40 micrometers near the cement paste/aggregate boundary, where there is an increase in porosity. The increase in porosity is probably a result of an inability of cement particles to densely pack near the paste/aggregate boundary. The interfacial zone can dramatically enhance the transport properties of concrete. For instance, when there is a large volume fraction of aggregate; it is easy for the interfacial zone to percolate through the system forming a highly permeable path.

The complexity of concrete's microstructure makes the theoretical and experimental investigation of its transport properties a great challenge. Transport of fluids and materials depend on a large number of factors, such as porosity, pore size distribution, connectivity, and

tortuosity. These factors, in turn, depend on the volume fraction of materials used, details of cement hydration, and the processing of concrete. In addition, new high-performance concrete (HPC-s) mixtures are being developed. HPCs are composed of a wide variety of materials, such as silica fume, blast furnace slag, fly ash, and plastisizers, to name but a few, that can greatly alter concrete's transport properties by modifying its pore structure.

Transport mechanisms include permeability, diffusion, electrical conductivity, and capillary transport.

*Fluid Permeability.* A random porous medium is a network of pores of varying size. When this network forms a connected path through the medium, it is capable of transporting fluids. The permeability of concrete characterizes the ability of a fluid-saturated porous medium to transport fluids when subjected to an applied pressure gradient and is defined by the Darcy's law. Permeability is a major factor that determines durability of concrete structures. If the permeability is low, durability of a structure is improved because the transport of aggressive substances, such as oxygen, chlorides and carbon dioxide, is disabled.

*Diffusion-based predictions of permeability.* Another promising approach towards permeability estimation has been to consider diffusion bounds. Ideally, such methods would be useful for determining the permeability of concrete with an extremely low connected porosity and thus a very low permeability. To understand how diffusion might be related to permeability, consider a set of uniformly distributed particles that are allowed to diffuse in a porous medium. The average time it takes the particles to reach a wall is called the mean lifetime, of the particles.

*Air Permeability.* The study of air permeability is important for understanding the ability of HPC to encapsulate hazardous materials or act as a barrier to gasses such as radon. Air permeability studies may also aid understanding the ingress of carbon dioxide that results in the carbonation of concrete. The air permeability method is based on creating vacuum on the surface of concrete and in measuring the rate at which pressure is decreasing. This method is applicable for measurement in-situ. Main problem of method is definition of volume in concrete from which gas flowing.

*Gas permeability.* Gas permeability is property by which material allows gas to pass through it under a pressure difference. The pressure difference between the two opposite sides of a test specimen is kept constant. The gas permeability test is a laboratory method.

*Water permeability.* Instead of gas, water flow through material can be measured under pressure. Water has lower viscosity than gas, and calculation of permeability coefficient is more complicated, but method gives us a good comparison with real situation at concrete structures.

#### **5.4.2 Chemical Reactions of Aggregates**

*Types of reactions.* Chemical reactions of aggregates in concrete can affect the performance of concrete structure [5.3]. Some reactions may be beneficial; others may result in a serious damage to the concrete by causing abnormal internal expansion which may produce cracking, displacement of elements within larger structural entities, and loss of strength.

Alkali-silica reaction – the reaction that has received the greatest attention and which was the first one recognized, involves a reaction between the  $\text{OH}^-$  ion associated with the alkalis ( $\text{Na}_2\text{O}$  and  $\text{K}_2\text{O}$ ) from the cement and other sources, with certain siliceous constituents that may be present in the aggregate. This phenomenon was referred to as “alkali-aggregate reaction”, but is more properly designated as an “alkali-silica reaction”.

*Mechanisms increase spacing* -Alkali-silica reaction can cause expansion and cracking of concrete structures and pavements. Silica can be dissolved in solutions of a high pH. The initial reaction product at the surface will be a non-swelling calcium-alkali silica gel approaching C-S-H. For a reaction to continue safely, the amount of reactive material must either be negligible or more than a pessimum (means worst, the opposite of optimum) amount, depending on the amount of alkali and the fineness of the reactive material. Formation of a non-expansive product is desirable and will occur, if the reactive particles present are sufficiently numerous or sufficiently fine. Alkali-silica reactive materials of high fineness are in fact pozzolanic materials and blast-furnace slag. If properly made and used, the

materials may transform the reactions to become beneficial. If the amount of alkali is large, with respect to the reactive aggregate surface, the interior alkali-silica gel with unlimited expansive potential will form, imbibe, water, and exert a potentially destructive force.

*Carbonation.* When concrete or mortar is exposed to carbon dioxide, a reaction producing carbonates takes place, which is accompanied by shrinkage. Virtually all the constituents of a hydrated portland cement are susceptible to carbonation. The results can be either beneficial or harmful, depending on the time, rate, and extent to which they occur, and the environmental exposure. In one hand, intentional carbonation during production may improve the strength, hardness, and dimensional stability of concrete products. In another, carbonation may result in deterioration, and a decrease in the pH, of the cement paste leading to corrosion of reinforcement near the surface. Exposure to carbon dioxide during the hardening process may affect the finished surface of slabs, leaving a soft, dusting, less wear-resistant surface. During the hardening process, the use of unvented heaters, or exposure to exhaust fumes from equipment or other sources, can produce a highly porous surface subject to further chemical attack. The source of the carbon dioxide can be either the atmosphere or water carrying dissolved CO<sub>2</sub>.

Atmospheric carbonation – a reaction of a hydrated portland cement with carbon dioxide in the air is generally a slow process. It is highly dependent on the relative humidity of the environment, temperature, permeability of the concrete, and concentration of CO<sub>2</sub>. Highest rates of carbonation occur when the relative humidity is maintained between 50 and 75 percent. Below a 25 percent relative humidity, the degree of carbonation that takes place is considered insignificant. Above a 75 percent relative humidity, moisture in the pores restricts CO<sub>2</sub> penetration. A relatively permeable concrete undergoes a more rapid and extensive carbonation than a dense, well-consolidated, and cured concrete. Lower water-cement ratios and good consolidation also serve to reduce permeability and restrict carbonation to the surface. Concrete in industrial areas with higher concentrations of CO<sub>2</sub> in the air is more susceptible to attack.



Carbonation by ground water – Carbon dioxide absorbed by rain enters the ground water as carbonic acid.

**Table 5.1** Factors influencing chemical attack on concrete

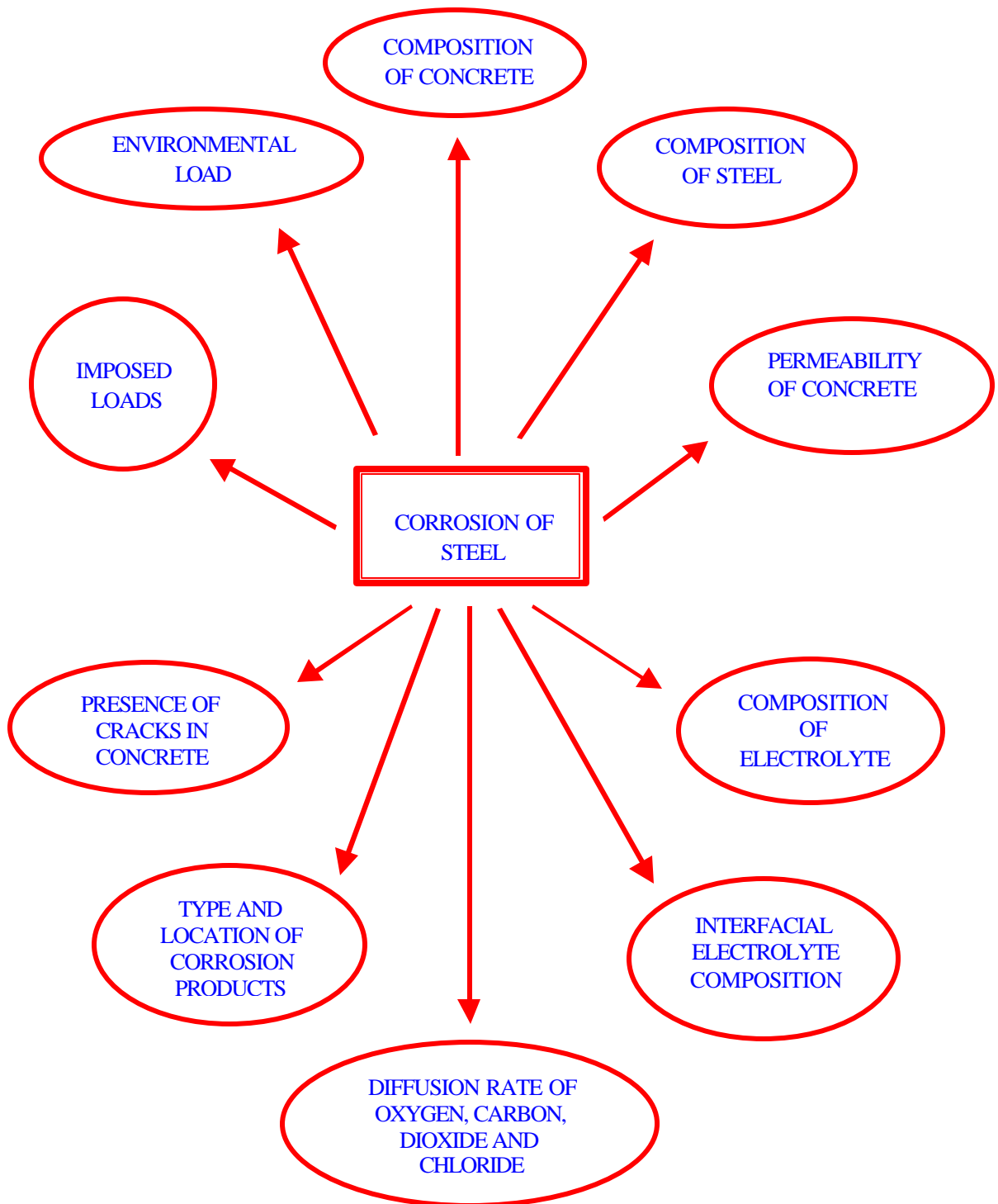
Factors which accelerate or aggravate attack	Factors which mitigate or delay attack
<p>⇒ High porosity due to:</p> <ul style="list-style-type: none"> <li>• High water absorption</li> <li>• Permeability</li> <li>• Voids</li> </ul> <p>⇒ Cracks and separations due to:</p> <ul style="list-style-type: none"> <li>• Stress concentrations</li> <li>• Thermal shock</li> </ul>	<p>⇒ Dense concrete achieved by:</p> <ul style="list-style-type: none"> <li>• Proper mixture proportioning</li> <li>• Reduced unit water content</li> <li>• Increased cementitious material content</li> <li>• Air entrainment</li> <li>• Adequate consolidation</li> <li>• Effective curing</li> </ul> <p>⇒ Reduced tensile stress in concrete by:</p> <ul style="list-style-type: none"> <li>• Using tensile reinforcement of adequate size, correctly placed</li> <li>• Inclusion of pozzolan (to suppress temperature rise)</li> <li>• Provision of adequate contraction joints</li> </ul>
<p>⇒ Leaching and liquid penetration due to:</p> <ul style="list-style-type: none"> <li>• Flowing liquid</li> <li>• Ponding</li> <li>• Hydraulic pressure</li> </ul>	<p>⇒ Structural design</p> <ul style="list-style-type: none"> <li>• To minimize areas of contact and turbulence</li> <li>• Provision of membranes and protective-barrier systems to reduce penetration</li> </ul>

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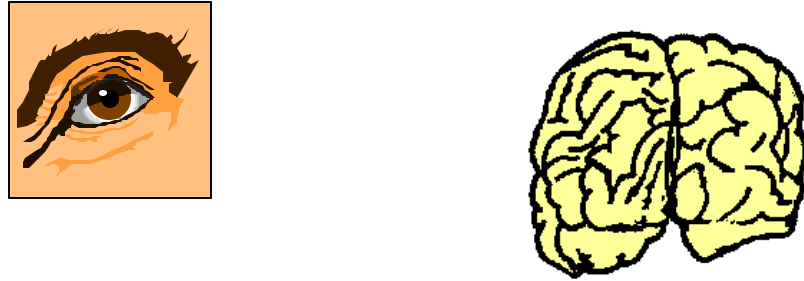
## 6 Overview of Testing Methods

Corrosion of reinforcing steel in concrete is causing damage in concrete structures all round the world, and problems associated with corrosion are growing [6.1]. Corrosion induced deterioration is related to the use of thawing salts, influence of the marine environment, and influence of the atmospheric carbon dioxide. All kinds of civil engineering facilities, such as bridge, transportation infrastructure, tunnels, buildings, are in danger, so that corrosion monitoring and condition monitoring in general are becoming essential. Various electrochemical and nonelectrochemical methods are developed for this purpose (Fig. 6.1). Nonelectrochemical methods include various approaches: from visual inspections of concrete structures to infrared thermography and acoustic emission methods. Electrochemical methods can be divided into static measurements and polarization measurements. The methods can be also categorized as in situ and laboratory measurements. Each of these approaches has certain advantages and disadvantages, and the best approach is to combine them. Determination of the reinforcement corrosion in concrete is a complex problem, which requires experienced specialists. Successful measurement of corrosion requires good understanding of corrosion processes. Otherwise, misinterpretations are frequently made.



**Fig. 6.1** Factors Influencing the Rate of Corrosion of Steel in Concrete Structures

There are many devices for corrosion investigation, but there is no single technique which will tell the engineer what precisely to look for, what is the extent of damage, and how rapidly the damage will grow with time? Visual inspection is always the first step in the structural evaluation., where the main equipment is the human *eye & brain* (Fig. 6.2).



**Fig. 6.2** Visual inspection

The depth of cover cracks, spalling and delamination of concrete are major indicators of structural performance, and of interest in the condition assessment of the structure. Evaluation of the degree of corrosion is a preliminary step in the corrosion identification, that needs to be further investigated. A *profometer* (Fig. 6.3) is a rebar locator which assists visual inspection in locating reinforcing bars and measures the depth of concrete cover and diameter reinforcement depths by means of magnetic induction.



**Fig. 6.3** Profometer

There are two principle causes of corrosion: a chloride attack on rebars and carbonation of concrete. The distinction between these two is important, because the type of repair may be determined by the cause of corrosion. *Carbonation* is easily detected and measured by exposing fresh concrete and spraying phenolphthalein indicator (Fig. 6.4) on it.

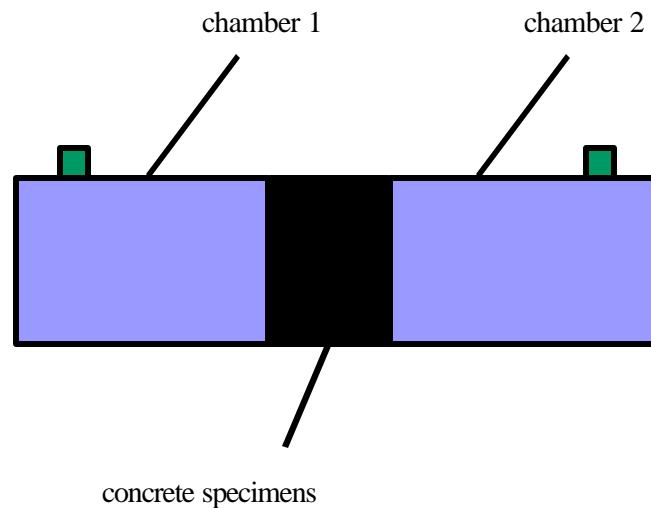


**Fig. 6.4** Phenolphthalein indicator

*Chloride Content Examination.* The method involves drilling small holes in the concrete at the level of reinforcement, taking out powder and the analysis of the powder in the laboratory to find the chloride content [6.2].

### 6.1 Measurement of the Diffusion Coefficient

The technique utilizes a steady state condition of diffusion to measure the diffusion coefficient [6.3]. The setup includes two chambers (Fig. 6.5): the first contains a solution of the diffusant under study, while the second chamber contains the sample and the solution (initially free of the diffusant). The concentration of the diffusant in chamber two is measured periodically and plotted.



**Fig. 6.5** Page cell

### 6.2 Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration

The test method covers determination of the electrical conductance of concrete to provide a rapid indication of its resistance to penetration of chloride ions [6.3]:

$$I = \frac{U}{R}[A], \quad R=0.43W$$

$$Q = 900 (I_0 + 2I_{30} + 2I_{60} + \dots + 2I_{300} + 2I_{330} + 2I_{360})$$

$I$  = current (amperes)

$U$  = voltage (volts)

$R$  = resistance (ohms)

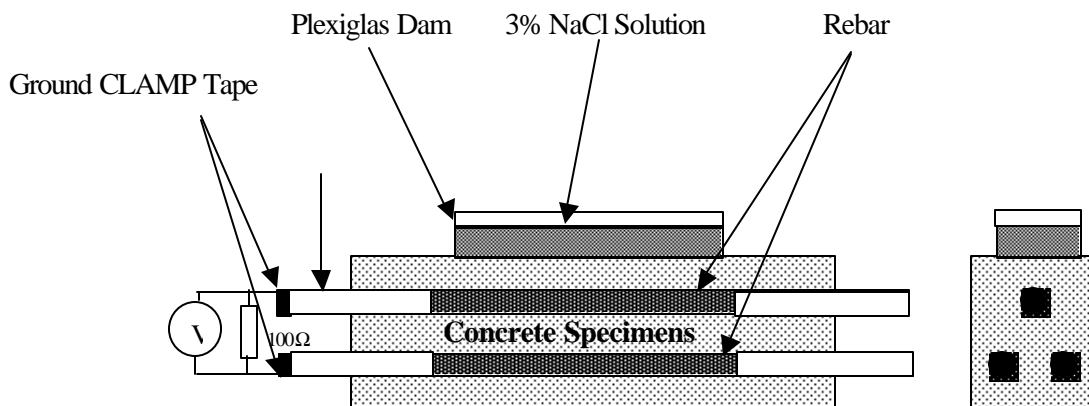
$Q$  = charge passed (coulombs)

$I_0$  = current (amperes) immediately after voltage is applied, and

$I_t$  = current (amperes) at  $t$  min after voltage is applied.

### 6.3 Determination of Effects of Chemical Admixtures on the Corrosion of Embedded Steel Reinforcement in Concrete Exposed to Chloride Environments

The ASTM standard test method [6.4] is useful in evaluation of efficiency of corrosion inhibitors to be used in concrete. Three concrete mini beams are cast with one rebar at the top and two at the bottom (Fig. 6.6). Each bar is attached to a ground clamp. A grounding wire connects two bottom bars. A  $100\Omega$  resistor is placed between the bottom bars and the top bar through the ground connectors. The voltage across the resistor is measured till the microcell current increases to  $10\ \mu\text{A}$  for at least half of specimens. That point is indication of reinforcement being totally corroded.



**Fig. 6.6** Diagram (side view) of concrete minibeam specimen

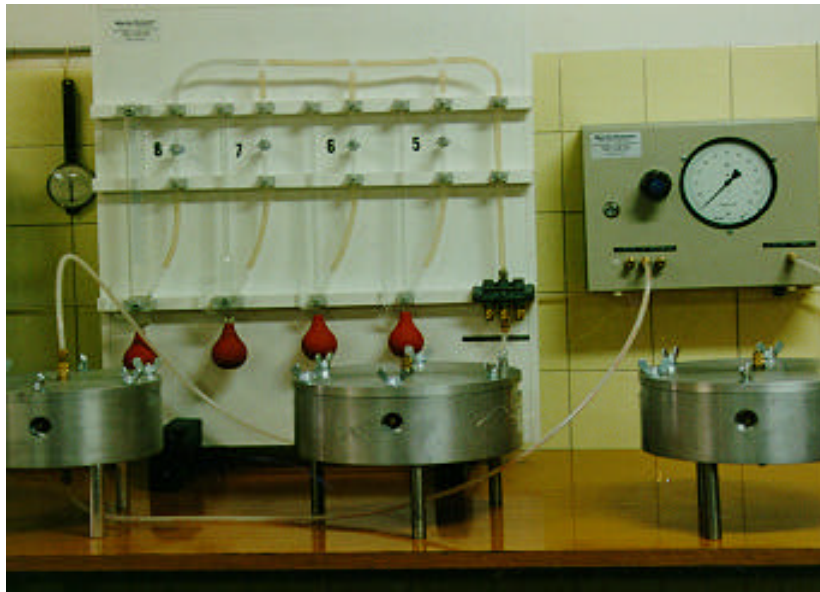


## 6.4 Concrete Structure

### *Gas - Permeability*

a) Method of gas permeability at constant gas pressure difference

The machine used to measure concrete permeability (nitrogen under constant pressure) [6.5] is shown in Fig. 6.7.



**Fig. 6.7** A machine used to measure concrete permeability

The theory behind the method of gas permeability at a constant pressure difference is based on the Hagen-Poiseuille relationship. The relationship is for laminar streaming of a compressible fluid through a regularly shaped model with a series of narrow parallel capillaries under stable conditions. The solution of the system points to a specific gas permeability coefficient  $K_s$ , given by equation (6.1):

$$K_s = \frac{2 \cdot \eta \cdot L \cdot Q \cdot p_2}{A \cdot (p_1^2 - p_2^2)} \quad (\text{m}^2) \quad (6.1)$$

where,

$K_s$  - specific gas permeability coefficient  $(\text{m}^2)$

$Q$  - flow  $(\text{m}^3/\text{s})$

$p_1$ - input pressure (absolute)	$(\text{N/m}^2)$
$p_2$ - output pressure (atmospheric)	$(\text{N/m}^2)$
$L$ - thickness of the sample in the flow direction	$(\text{m})$
$\eta$ - dynamic viscosity	$(\text{Ns/m}^2)$
$A$ - area of the sample	$(\text{m}^2)$

Methods of gas permeability at a variable pressure difference are non-destructive methods for in-situ testing. Instead of gas, a measurement of water flow under pressure through a material can be used. This method compares well to real situations in concrete structures.



**Fig. 6.8** Waterproof testing

## 6.5 Resistance to Frost

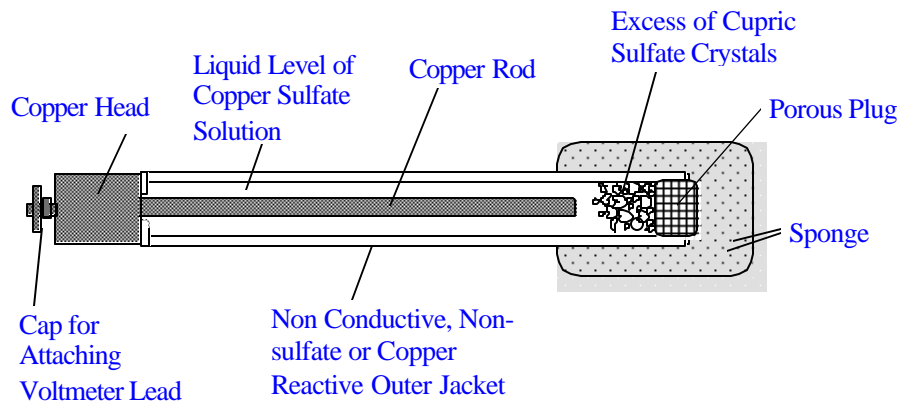
Evaluation of concrete resistance to frost and salt action is done to predict its service life, to determine the suitability of concrete composition, and to detect a possible need for implementation of protection measures. The difficulty lies in the fact that there is a no single testing method that provides information about the durability, for each concrete composition, for all placement methods and for all environmental conditions appearing in practice (Table 6.1 [6.6,6.7]).

**Table 6.1** Choice of Optimum Method

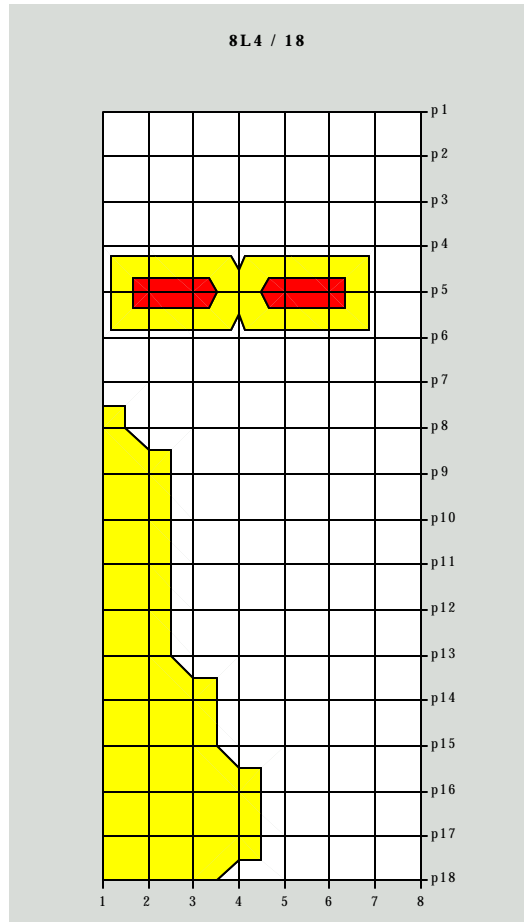
Method	Resistance Criteria	Type of Structure and Exposure
Resistance to Frost	Decrease in compressive strength. Decrease in dynamic modulus. Decrease in flexural strength. Change of resonance frequency.	Facades, columns, slabs and other structural elements not constantly exposed to water. Hydraulic structures and structural elements exposed to water.
Resistance to frost and deicing salt	Surface damage	All structures where salt is applied (roads, runways, platforms, industrial objects).
Linear deformations during freezing	Dilation factor	All types of structures
Microscopic determination of air voids	Distance factor $\leq 0.2$ mm	All types of structures
Critical saturation - $S_{cr}$	$S_{cr} = 0.80-0.95$	Only laboratory method, for preliminary testing

## 6.6 Half Cell Potentials

The principle of the method [6.8] is to develop electrical half-cell potential between the reinforcing steel and a copper/copper – sulfate reference electrode (Fig. 6.9). The half-cell is moved in a grid pattern over the concrete surface to be investigated, and the electrode potentials are measured at each point using a high impedance voltmeter. The measured potential is mapped in a form of equipotential contours to identify the corrosion areas (Fig. 6.10).



**Fig. 6.9** Sectional View of a Copper-Copper Sulfate Half Cell



Measured potential  $E_{kor}$  (Cu/CuSO<sub>4</sub> EQU)

Probability of Corrosion

	-350 (mV)	> 90 %
	-201 -350 (mV)	≅ 50 %
	-200 (mV)	< 10 %

**Fig. 6.10** Equipotential Contour Map Plotted using Half-Cell Readings

*GECOR6* [6.9,6.10] is the latest corrosion rate meter developed under Eureka/Eurocare project EU-401, and has three major components: a rate meter that automatically controls the system, a sensor for corrosion rate and half-cell potential measurements, and a sensor for concrete resistivity, ambient temperature and relative humidity measurements. The linear polarization technique for corrosion rate measurement provides a quantitative information on the deterioration rate.

## **6.7 Portable Seismic Pavement Analyzer (PSPA)**

The PSPA is a device for nondestructive evaluation of material properties and defects in bridge decks and the top pavement layer using seismic techniques [6.11].

*Use of the PSPA:*

- Quality control / quality assurance
- Evaluation of bridge deck delamination
- Detection of voids within structural elements
- Monitoring of concrete curing

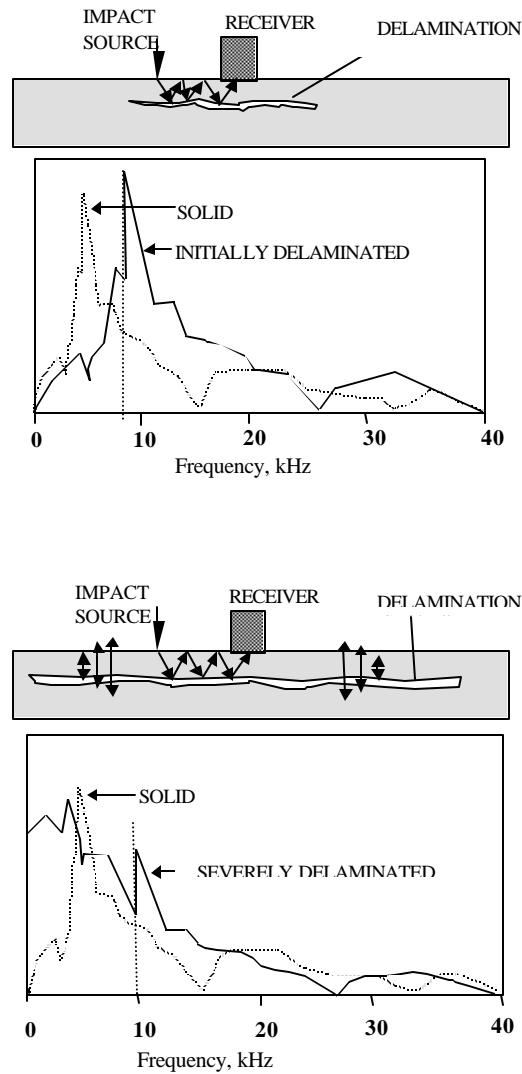
*Parameters Measured*

- Portland Cement Concrete
  - Elastic moduli of the surface layer
  - Poisson's ratio
  - Slab thickness or position of the delamination
- Asphalt Concrete
  - Elastic moduli of the surface layer
  - Poisson's ratio
  - Thickness

*Implemented Seismic Techniques*

- Ultrasonic Body Wave (UBW)
- Ultrasonic Surface Wave (USW)
- Impact Echo (IE)

Evaluation of elastic moduli and layer thickness by UBW and IE methods is presented in Fig. 6.11.



**Fig. 6.11** Evaluation of elastic moduli and layer thickness by UBW and IE methods

In the first part of the test, the UBW method is used to measure the velocity of the compression wave. In the second part, the IE test is used to measure the frequency of reflections from a reflector, the return frequency. From the return frequency and the compression wave velocity the depth of the reflector can be calculated. If there is an initial

delamination, the return frequency is higher than the return frequency for reflections from the deck bottom (upper figure). For a severely delaminated deck, a significantly lower return frequency is observed. This is because the dominant response is coming from oscillations of the whole upper delaminated portion of deck, instead of wave reflections from the delamination (lower figure).



## 6.8 Example - Bridge Deck Overlay Evaluation Plan

*Purpose.* This outline provides an overview of procedures required in evaluation of bridge deck overlays [6.12].

Task 1. Evaluate conditions of each installation before an overlay is placed using condition evaluation procedures similar to those used in the Strategic Highway Research Program to the extent it is practical. Include the following:

- Electrical half-cell potentials (ASTM C876)
- Chloride ion content profiles
- Crack and patches maps
- Permeability to chloride ion (AASHTO T277)
- Preinstallation photographic records

Task 2. Document the specifications used for each installation.

For hydraulic cement concrete, record (or test) and report:

- Site preparation and preoverlay repairs
- Surface preparation (include moisture condition of deck surface)
- Overlay technology selected
- Overlay design thickness
- Overlay design life
- Mixture proportions and characteristics of components (including moisture content of aggregates); include silica fume if used
- Curing method and time (follow State standard practice)
- Bond strength (VTM-92, ACI 503, or equivalent)
- Compressive strength (ASTM C39)
- Grout--follow State standard practice unless EKI bonding process is being used

For polymer overlays record (or test) and report:

Site preparation and preoverlay repairs

- Surface preparation
- Overlay technology selected
- Overlay design thickness
- Overlay design life
- Mixture proportions
- Binder-to-aggregate ratio
- Pot Life (ASTM C881)
- Tensile strength (ASTM D638)
- Tensile elongation (ASTM D638)
- Viscosity (ASTM D2393)
- Minimum compressive strength at 3 hrs (ASTM C109)
- Minimum compressive strength at 24 hrs (ASTM C109)
- Minimum adhesion strength at 24 hrs (VTM-92, ACI 503, or equal)
- Curing shrinkage

Task 3. Record results of job control testing or quality assurance testing for each site.

Tests for hydraulic cement concrete:

- Mixture proportions and characteristics of components including admixtures
- Record the placement time
- Record climatic conditions during the placement, including temperature, wind speed, relative humidity, and rainfall
- Measure rate of evaporation during the placement
- Compare actual mix properties to design specifications
- Water-to-cement ratio
- Drying shrinkage of the overlay concrete (ASTM C157)
- Thermal coefficients of the deck and overlay concrete
- Slump
- Compressive strength of the overlay (1 day and 28 days) (ASTM C39)
- Concrete temperature

- Aggregate gradations and moisture content
- Entrained air
- Curing time and type

Tests for polymer overlays:

- Mixture proportions and characteristics of components
- Record the placement time
- Record climatic conditions during the placement
- Binder-to-aggregate ratio
- Thermal coefficients of deck and overlay concrete
- Pot Life (ASTM C881)
- Tensile strength (ASTM D638)
- Tensile elongation (ASTM D638)
- Viscosity (ASTM D2393)
- Min. compressive strength at 3 hrs (ASTM C109)
- Min. compressive strength at 24 hrs (ASTM C109)
- Min. adhesion strength at 24 hrs (VTM-92; modified ACI 503)

Task 4. Evaluate initial conditions after placement of each installation.

For either hydraulic cement concrete or polymer overlays:

- Locate delaminations before opening to traffic using chain drag or other methods
- Measure roughness before opening to traffic using the straight edge
- Perform skid tests after 4 to 8 weeks of traffic; follow the State standard practice; report the procedure and results. If possible, tests should be performed at 40 and 50 mph using both a bald and a ribbed tire.
- Measure electrical half-cell potential (ASTM C876) on one lane of one span before opening to traffic
- Conduct three VTM-92 (modified ACI 503) or any reproducible tensile adhesion tests approximately six weeks after the installation.

- Conduct three AASHTO T277 permeability tests on cores approximately six weeks after the installation. The cores should be taken no earlier than 14 days after the installation. Use test cores to determine the thickness of the overlay.
- Post installation photographic record
- Map cracks, patches, and test locations
- Report cost of the overlay

Task 5. Evaluate condition of each installation annually.

- As a minimum, conduct a visual site inspection once a year and document changes, if any
- Collect weigh-in-motion data if possible, traffic volumes, 80kN ESALs
- Summarize climatic conditions, number of salt applications

Task 6. Evaluate final condition of installation. Repeat tests in Task 4 as warranted during the last field inspection prior to the final report.

- Chloride ion content profile
- Review and summarize bridge deck condition data from previous inspection reports

Task 7. Submit draft and final report to the FHWA. Include an estimate of the remaining service life of the overlay and an evaluation of its cost-effectiveness. Include an assessment of how well the project met the objectives of Section 6005 (e) 7, which states:

As a part of the program under this subsection, the secretary shall carry out projects to assess the state of technology with respect to thin bonded overlays (including inorganic bonding systems) and surface lamination of pavement, and to assess the feasibility of, and costs and benefits associated with, the repair, rehabilitation, and upgrading of highways and bridges with overlay. Such projects shall be carried out so as to minimize overlay thickness, minimize initial laydown costs, minimize time out of service, and maximize lifecycle durability.

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## **7 Repair of Reinforced Concrete Structures Damaged by Corrosion of Reinforcement**

### **7.1 Overlays**

This chapter deals with overlays placed on a cured bridge deck as a protective shield against water, chemicals, abrasion, or slipperiness [7.1].

#### *Need for overlays*

*Waterproof barrier.* The primary reason for the use of overlays is prevention and repair of spalling on concrete bridge decks. Such spalling is a result of expansive forces built up within the deck concrete by the products of corrosion of reinforcement steel. Such corrosion is induced by a presence of moisture and chlorides. Cracks over the reinforcement or pores can accelerate the rate of deterioration. Thus, where cracks or porous concrete are evident and deicers are used, some type of a waterproof barrier should be provided or spalling may be anticipated.

It should be reemphasized that a careful attention to good design and construction practices, as set forth elsewhere in this standard practice, should significantly reduce the propagation of cracks and prevent the acceptance of poor quality concrete. However, where repair costs have become excessive or where good practice is known to have been compromised, an overlay may be a cost-effective means of extending service life.

*Slipperiness.* Bridge decks, like all roadway surfaces, must be adequately skid-resistant. Occasionally, rapid surface wear, due to construction deficiencies and inadequate skid-resistant aggregates, induces slipperiness. Overlays provide means for correcting this deficiency.

*Wearing course.* The use of studded tires has markedly increased the abrasive wear on some bridges. Consequently, overlays may be considered as a sacrificial wearing course, since the loss through abrasion of an overlay would not reduce the section modulus or critical clear cover over reinforcing steel in the structural slab. Overlays can be replaced with relative ease and low cost.

*Reduction of wheel load effect.* Asphaltic concrete (AC) overlays are commonly used to provide wheel load distribution and a smooth riding surface which helps reduce impact. They are also used as a riding surface over waterproofing membranes.

## **7.2 Required Properties of Overlays**

The required properties of overlays depend on their intended purpose, as discussed above.

***Properties required for all overlays.*** Several properties are generally required for all overlays, regardless of the reasons leading to their use.

*Adhesion* to concrete or bond is a fundamental requirement for most overlays. Without adhesion, overlays soon delaminate which, at best present an unsightly appearance and, at worst, require extensive repair.

*Cohesion* or resistance to shear within the overlay itself is necessary to resist the stresses induced by the turning and braking of the heaviest vehicles. This resistance may be relevant when considering the use of unreinforced thermoplastic materials, such as asphalt.

*Skid resistance* is a fundamental requirement of an overlay, whether or not that is the purpose for which it was intended, because the overlay becomes the road surface. This property requires addition of an abrasion-resistant aggregate to most of the polymer-type materials currently marketed as overlays. Grooving (diamond blade saw cut of hardened concrete) or texturing (of plastic concrete) is usually required when placing concrete overlays.

*Durability*, used here as resistance to abrasion, deformation and decay, is another important property. Many materials, such as bitumens, soften under high temperatures and become subject to rutting. Such rutting may be imperceptible in the roadway, but creates an undesirable bump at bridge joints. Other products may become brittle with age or when oxidized, and thus may not retain the properties for which they were intended. Extended service histories should be investigated for any proposed overlay.

***Properties required for waterproof barriers.*** In addition to the properties listed above, waterproof barriers should be designed so that the conditions which could lead to the intrusion of moisture and chloride ions are considered.



*Impermeability* is an important property of waterproof barriers. Materials may be impermeable in lab test conditions, but may be affected by ultraviolet rays or by the heat from asphalt paving. Introducing aggregates for skid resistance or as bulk fillers may also create interconnected voids that admit water. Some construction techniques induce foaming and porosity which may increase water intrusion.

*Crack* resistance is another important requirement for a waterproof barrier. Development of cracks in concrete is one of the conditions leading to the use of a waterproof barrier. Hence, barrier materials must be capable of bridging such cracks in the underlying deck and remaining waterproof. Reflective cracking in bridge decks is a much greater problem on long-span, cast-in-place decks.

*Bridge decks* expand and contract with temperature change, and overlays placed on them must do likewise without loss of bond. Where thermal incompatibilities exist between the concrete and the membrane, shear stresses will be created by temperature change. These stresses are proportional to the membrane thickness. Such stresses may exceed the bond strength of the membrane or the shear strength of the concrete, and the resulting failure will destroy the membrane's effectiveness. Thus, the coefficient of expansion of any membrane material is an important property where substantial temperature changes occur.

### **7.3 Types of Overlays**

Overlays can be grouped into three categories (Fig. 7.1):

Type I – Thin overlays

Type II – Concrete – based overlays

Type III – Combined - system overlays

***Thin overlays.*** Thin overlays have thickness of ½ in. or less and therefore add minimal dead load to structures. Their primary function may be to increase the skid resistance on slippery decks or to act as surface membranes to minimize penetration of water and chloride ions. They must generally be applied to dry concrete surfaces. Thin overlays usually involve durable, abrasion – resistant aggregates glued together by various binders including asphaltic

emulsions, polymer resins, and polymer-modified cements. Thin overlays are generally not recommended for badly spalled or deteriorated decks. Specialized expertise may be needed to properly apply these systems.

**Concrete overlays.** The thickness of this type of overlay varies from 1 in. to about 2<sup>1</sup>/<sub>2</sub> in. These overlays include latex-modified concrete, polyester-modified concrete, low-slump dense concrete, fast setting concrete, and some variations involving steel fiber or silica fume, or high-range water-reducing mixtures or cathodic protection. The primary function of these systems is to replace deteriorated concrete or asphalt wearing surfaces with an economical, durable, crack-resistant, low-permeability material without significantly increasing the dead load on the structure. Relative advantages and disadvantages of the systems may vary from one region to another, depending on local economic, climatic and design factors.

The selection of an overlay system should be a result of consideration of particular structural and site conditions. Shrinkage and surface cracking of concrete overlays are likely to be significant factors in cold climates where deicing salts are used, as compared to a little use of deicing salts. Shrinkage cracking is also a significant factor in dry and windy climates. High-slump mixes (higher than 4-in. slump) are not recommended for decks with longitudinal grades exceeding 2 percent. Cathodic protection systems should be routinely monitored to insure continued performance. The use of steel fibers, or admixtures such as silica fume or superplasticizers, is generally intended to improve crack resistance and impermeability. Prior to use, field experience of any particular system should be investigated.

**Membrane and AC overlays.** This overlay type includes a waterproofing membrane covered with one or two courses of asphaltic concrete. The total thickness range is usually from 2 to 4 in. The economics of asphalt makes this a good option, in addition to a good riding and shock absorbing qualities of the material.

There are many types of membranes, including hot applied, rubberized membranes; sheet membranes; and liquid-applied, polymer membranes. The membranes should be capable of bonding to concrete, bridging cracks, waterproofing, and bonding to AC overlays without being affected by possibly a 300 °F hot asphalt. Some membranes require protection boards and two passes of asphaltic concrete in order to minimize damage during compaction. These

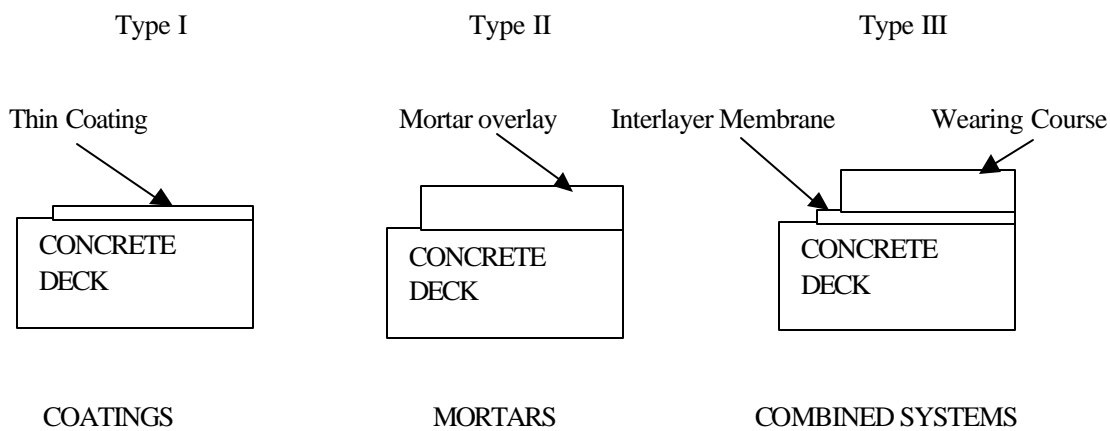
systems may not be suitable for repair of existing bridges that were not designed for an extra dead load. Some sheet membranes may not bond well to concrete, or may debond at later dates if exposed to heat and sunlight, which create vapor pressure and weaken the bond due to temperature. Liquid-applied membranes may require a special expertise. Membranes are not recommended for a repair of badly delaminated decks with corroded reinforcing bars close to the surface.

***Asphaltic concrete overlays.*** Asphaltic concrete overlays are used on bridge decks to provide a smooth riding surface and help reduce damaging impacts to deck [7.2]. They are also commonly used as a protective-wearing surface for penetration asphalt, membrane waterproofing system, or other deck sealers. Prior to the use of any overlay or increasing the thickness of an existing overlay, the ability of the structure to carry the added load should be investigated. As a rule, however, asphaltic concrete overlays are relatively porous and, by themselves, do not provide an effective seal. This porosity entraps salt-laden moisture which, in the absence of an effective deck sealer, can promote deck deterioration. As precautionary measure a multiple-course penetration asphalt surface treatment, membrane, or other deck sealer should always be applied prior to an asphaltic concrete overlay. When placing asphaltic concrete overlays, end dams should be provided at expansion joints to protect the overlay next to the joint and keep the overlay material out of the joint.

Existing asphaltic overlays on concrete bridge decks should be inspected periodically for cracking and debonding from the concrete. A hammer or rod may be used to locate unbonded areas in the overlay. These areas are more commonly found along curbs, expansion joints, and at locations where the overlay has cracked. Once located, the overlay in these loose areas should be removed and replaced. Attempts should also be made to determine the condition of the concrete beneath to overlay. If the concrete deck is deteriorated, all unsound concrete should be removed and replaced prior to replacing the asphaltic overlay. Care should be taken to finish the concrete patch flush with the existing deck. Dormant cracks in the concrete should be filled and active cracks should be sealed with crack-sealing material to prevent entry of water.

To insure good adhesion, the concrete deck must be dry and primed with an effective sealer and a bonding agent before the asphaltic overlay is placed. Care should be taken to assure that the overlay is thoroughly compacted.

Wearing courses are generally asphaltic concretes. The design of such courses is beyond the scope of the mentioned standard practice. An AC overlay should not be used directly on a portland cement concrete deck without a waterproofing membrane. All AC mixtures are inherently porous and readily conduct water and chlorides to the portland cement concrete deck where they cannot be flushed off. Such impounded brine greatly accelerates bridge deck deterioration, which is then difficult to observe or measure below the asphalt. Also, the permeability of AC greatly increases with age.



**Fig. 7.1** Several types of overlays showing wearing courses and/or interlayer membranes

#### 7.4 Positive Protective Systems

Repair costs for repairing corrosion-caused damage are typically very high [7.3,7.4]. Many protective systems have been proposed, some of which have been shown to be effective while others have failed. It is beyond the scope of this guide to discuss all possible systems. However, the most successful system are listed in the following paragraphs.

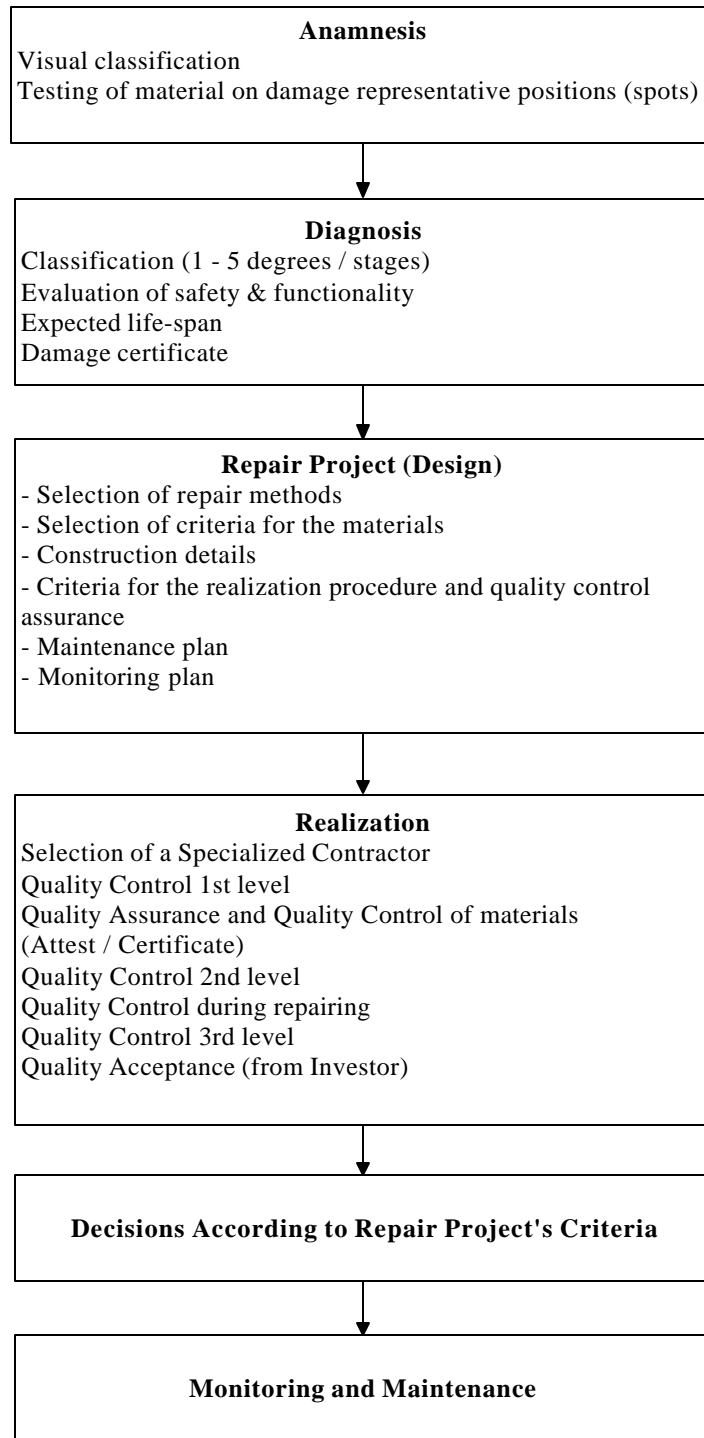
- Overlays and patches of very low water-cement ratio (0.32) using conventional low-slump concrete, latex-modified concrete overlays, concrete containing silica fume, and concrete containing high-range water reducing admixtures.
- “Waterproof” membranes.
- Surface protective-barrier systems produced from selected silanes, siloxanes, epoxies, polyurethanes, and methacrylates.
- Cathodic protection.
- Polymer impregnation.
- Replacement of the existing concrete with concrete containing a corrosion inhibitor.
- Overlays of Asphalt Concrete with corrosion inhibitors (Appendix C).

## **7.5 Steps Required in Repair of Bridge Decks**

The systematic approach to the repair of a reinforced-concrete structure [7.6], damaged by reinforcement corrosion, consists of a number of separate activities presented by a flow diagram in Fig. 7.2. The systematic approach to the repair of reinforced –concrete structures consists of a number of separate activities: anamnesis, diagnosis, repair design, repair execution, handing over, monitoring and maintenance.

To take a proper attitude concerning the degradation factors, it is necessary to carry out the following activities:

- A detailed research on climatic and microclimatic conditions, as well as of technological purposes (functions) of the structure
- A detailed research of the initial state a structural element (properties of the materials, dimensions) on a representative position (spot). This is in fact the first step, or the *anamnesis* of the structure's condition.



**Fig. 7.2** Method flow diagram in repair of reinforced-concrete structures

The next step is the *diagnosis*, which includes definition of causes and consequences of the damage identified. On the basis of visual estimates and the results obtained from testing of elements at representative positions in the structure (including testing performed directly on the structure, and of the samples taken from the structure), the damage is classified into 5 categories (degrees), according to the criteria given in Table 7.2 [7.6,7.7]. Accordingly, the final classification of the damage into the categories is a visual classification supplemented by criteria of boundary values for the parameters tested on the structure, or on the samples in the laboratory.

The decision of a strategy of reinforced concrete structures' repair design may be based on different principles with respect to either total prevention, or allowing of aggressive substances to penetrate into concrete. The selected principle (method) should be consistently carried out on the basis of a good knowledge of corrosion electro-chemical processes on the reinforcement surface, and chemical and physical processes in the concrete protective layer. Besides the principle (method) selection, the repair-work design should also provide criteria for repair materials, execution, quality control, and the allowed change of the repaired structure or of the state of a structural element within the warranty period. Repair-work can be assigned only to the Contractor with the references for similar works. Corrosion of the reinforcement depends on the transit (transport) of the substances from the environment into the protection layer of the concrete. This means that in repair of reinforced-concrete structure damaged by reinforcement corrosion, quality and the thickness of the protective layer hold the greatest significance.

Quality control should be carried out in three stages:

- I stage - quality assurance and quality control of materials
- II stage - quality control during the repair execution
- III stage - acceptance of the quality by the investor.

The handing over date of the repaired structure does not mean that the repair project is successful. It is necessary to continue with monitoring and maintenance consistently as determined by the repair project.



**Table 7.1** Criteria for the final classification of damage

Final category of damage	Visual estimate	Criteria for boundary values				
		Gas permeability /m <sup>2</sup> /	Measurement of potentials /mV/	Chemical analysis Cl <sup>-</sup> /%/ pH	Corrosion Rate /μA/cm <sup>2</sup> /	Pull off [7.8] /N/mm <sup>2</sup> /
I Surface imperfections	air-bubbles on <10% of the surface, cracks < 0.1 mm	>10 <sup>-16</sup>	>-200	<0.4 >11	<0.20 no corrosion	<5% of design strength
II Initial	Cracks <1mm, Shear steel visible	1-5 · 10 <sup>-16</sup>	-200 to -350	0.4 –1.0 11-10 depth <5mm	0.20 – 1.0 corrosion for 10 – 20 years	<10% of design strength
III Advanced	Cracks 1-2mm, concrete delamination up to 1 mm,	5-10 · 10 <sup>-16</sup>	<-350	>1 10-9 depth <10mm	1 – 10 corrosion for 2 – 10 years	<25% of designed strength
IV Active	Concrete delamination up to 30 mm, concrete splitting from reinforcement	< 10 <sup>-15</sup>	<-500	>2.0 <9 depth <30mm	>10 corrosion for <2 years	-
V Critical	Damage t depth of 50 mm, concrete breaking off from the reinforcement, reinforcement scaling	-	-	-		-

Cl<sup>-</sup> is concentration of Chloride ions

pH is measure of acidity and alkalinity

## References

- 7.1 ACI Committee 345: "Guide for Concrete Highway Bridge Deck Construction (ACI 345R-91)", ACI Manual of Concrete Practice, Part 4, 1995, pp. 345R-1-36.
- 7.2 ACI Committee 345: "Routine maintenance of Concrete Bridges (ACI 345.1R-92)", ACI Manual of Concrete Practice, Part 2, 1995, pp. 345R-1R-1-13.
- 7.3 ACI Committee 201: "Guide to Durable concrete (ACI 201.2R-92)", ACI Manual of Concrete Practice, Part 1, 1995, pp. 201.2R-1-41.
- 7.4 ACI Committee 546: "Guide for Repair of Concrete Bridge Superstructure (ACI 546-1R-80)", ACI Manual of Concrete Practice, Part 2, 1995, pp. 546.1R-1-20.
- 7.5 ACI Committee 362: "Guide for the Design of Durable Parking Structures" (ACI 362.1R-94)", ACI Manual of Concrete Practice, Part 4, 1995, pp. 362-1R-1-42.
- 7.6 Bjegovic D., Ukrainczyk V.: "Repair of R.C. Structures Damaged by Reinforced Corrosion", Proceedings of 6th International Conference Structural Faults + Repair 95, Extending the Life of Bridges, Civil+Building Structures, July 3-5. 1995, London, pp. 243–247.
- 7.7 ASTM Standardization News: "How Corrosion Impacts Our Daily Lives, Our Safety and Our Economy", October 1998, pp. 28-31.
- 7.8 EN 1542:1999: Products and systems for the protection and repair of concrete structures – Test methods – Measurement of bond strength by pull-off.

## 8 Summary and Conclusions

Concrete delamination (spalling) in bridge decks is a serious problem. Damage in reinforced concrete structures is caused by many factors, such as alkali-aggregate reaction, faulty design and construction, reinforcement corrosion, corrosive action of chlorides, geotechnical problems, or in general by mechanical, chemical, physical, biological and environmental loads, as explained in Chapter 2. To make proper assessment of a structure's durability, it is important to define for it active environmental loads, reactions, material performance and corresponding testing methods. Parameters obtained from the results of performed test can be used in the design to determine the size of the cross section, quality and the size of reinforcement, as well as material properties and parameters for the quality assurance and quality control.

A new way in the durability design of reinforced concrete structures is described in Chapter 3. A mathematical model for the durability design of reinforced concrete structures that insures bearing conditions during the entire service life of a structure, is presented. Unlike the existing methods, the time framework of the calculation is extended to the period of propagation of the reinforcement corrosion process in concrete. This contributes considerably to an extension of the service life. The model is based on the proposal of supplementing the existing design procedure, based on the criteria of serviceability limit states, by a new criterion that insures the bearing capacity of the cross-section under conditions of a simultaneous action of traditional and environmental loads. Environmental loads incorporate durability factors, which can be expressed either qualitatively or quantitatively, and thus are included into the design procedures. Environmental loads influence mostly changes in the active bearing capacity of the reinforcement cross-section. The result of the proposed design procedure, according to the corrosion limit state criterion, are the initiation and propagation periods of corrosion process, i.e. the structure's service life and its maintenance time schedule within the entire designed service life. Finally, dimensioning according to the corrosion limit

state criterion often reduces the amount of reinforcement, in comparison to the amount necessary according to the conditions of the cracking limit state.

A new approach in mix design of concrete was presented in the Chapter 4: Concrete production by means of a knowledge supported mix design. Mix design of concrete is based on the assurance of the conditions of the structure's design, assurance of the required workability and concrete strength, and the durability assurance. The research conducted using this approach resulted in a development of the information system CAMDOC for the computer aided mix design. CAMDOC provides ability of simple and quick reaction to all variable changes in the concrete mix, equipment and concrete production, for the purpose of the concrete quality control.

To better understand how to make high quality concrete, it is important to consider the concrete structure, external influences and transport properties. Those are in details described in Chapter 5. Since all reinforced concrete, many of high importance, are susceptible to corrosion, corrosion monitoring and evaluation of service life is necessary. For this purpose, different electrochemical and nonelectrochemical methods are shown in Chapter 6. Finally, many reinforced concrete structures end up with a corrosion induced damage. Repair of reinforced concrete structures damaged by corrosion is described in Chapter 7. To minimize future damage, many protective systems have been proposed, some of which have been shown to be effective while others have failed. It was beyond the scope of this report to discuss all the possible systems. However, the most successful systems are listed below, from which the first two are mostly recommended:

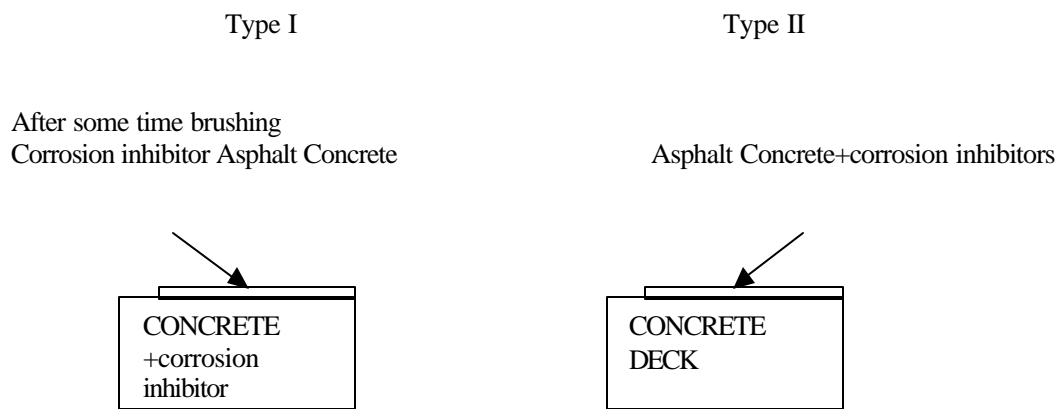
- Replacement of the existing concrete with concrete containing a corrosion inhibitor (Appendices C and D).
- Overlays of Asphalt Concrete with corrosion inhibitors (Appendices C and D).
- Overlays and patches of very low water-cement ratio (0.32) using conventional low-slump concrete, latex-modified concrete overlays, concrete containing silica fume, and concrete containing high-range water reducing admixtures.

- “Waterproof” membranes.
- Surface protective-barrier systems produced from selected silanes, siloxanes, epoxies, polyurethanes, and methacrylates.
- Cathodic protection.
- Polymer impregnation.

After an investigation (Appendix D: Study of the Effectiveness of MCI/VCI Incorporated in Asphalt Primer Coating), suggested overlays are presented in Figure 8.1:

- Replacement of the existing concrete with concrete containing a corrosion inhibitor (Appendix C).
- Overlays of Asphalt Concrete with corrosion inhibitors (Appendix C).

#### Types of Bridge Deck Overlays



**Fig. 8.1.** Suggested blacktop resurfacing procedures.

Laboratory and site testing of overlays with corrosion inhibitors in concrete and asphalt concrete are recommended for further research work. Also, research and progress should be made by employing new concepts and models, and latest advances in concrete and steel production and protection. Recommended literature is given in Appendix E.

## Appendix A Basic Principles of the Fuzzy Sets Theory

The fuzzy sets theory was introduced by Zadeh [A.1] in 1965, as a result of a lack of classical mathematical apparatus for algorithm definition of certain systems' responses to input stimulus. Application of this theory allows for regulation of the system's control by a series of rules that simulate human (experienced) behavior. The fuzzy sets theory reached its peak in practice in eighties and its quick development to date has resulted in amazing achievements concerning the supervision of even the most complex processes. To explain the procedures applied in calculation of the reinforcement corrosion limit state safety factor, the basic principles of the fuzzy sets theory are briefly presented.

### Membership Function

We can take a set  $X$  to be a certain universal set in a usual sense, the set of all the bridges in New Jersey for example. We can also observe set  $S$ , which includes all the fatigued bridges in New Jersey. Can all the elements associated with this set be undoubtedly determined? A certain set  $X$  element cannot be associated with set  $S$  with no doubt, but an expert can determine whether the element is "more" or "less" associated with the set (if it has a greater or lesser degree of association). A real function  $m_S(x)$  is attributed to set  $S$  that fulfills the condition:

$$0 \leq m_S(x) \leq 1, \text{ for each } x \in X.$$

The value  $m_S(x)$  represents the element  $x$  grade of membership to the set  $S$ . The set  $S$  is called a fuzzy set, and the  $m_S(x)$  function is called the fuzzy set  $S$  membership function.

### Fuzzy Sets' Union and Intersection

By fuzzy sets, just like by the classical ones, basic calculation operations (union  $\cup$  and intersection  $\cap$ ) are defined.

If  $A$  and  $B$  are two fuzzy sets with the respective the membership functions  $m_A(x)$  and  $m_B(x)$ , then  $A \cup B$  and  $A \cap B$  are the fuzzy sets with the following respective functions:

$$m_{A \cup B}(x) = \max_{x \in X} \{m_A(x), m_B(x)\},$$

$$m_{A \cap B}(x) = \min \{m_A(x), m_B(x)\}.$$

$$x \in X$$

The fuzzy set A is said to be a subset of the fuzzy set B ( $A \subseteq B$ ) if:

$$m_A(x) \leq m_B(x).$$

### **Relation and Composition**

The fuzzy relation Z on the set X is defined as the fuzzy set Z for which the Cartesian product of the set X with itself ( $X \times X$ ) is a universal set. Accordingly, the fuzzy relation membership function Z is defined by the respective membership function:  $m_Z(x_1, x_2) \leq 1, (x_1, x_2) \in X \times X$ .

If X and Y are two universal sets where the fuzzy sets A and B are defined on, then the fuzzy relation R between the sets A and B is defined as the fuzzy set R with the respective membership function  $m_R(x, y), (x, y) \in X \times Y$ .

If the relation Z on the set X, and the relation R between the sets A and B are defined, then their composition  $Z \circ R$  is defined as a fuzzy set with a respective membership function:

$$m_{Z \circ R}(x, y) = \max \{ \min \{ m_R(x, v), m_Z(v, y) \} \},$$

$$v \in X \quad (x, y) \in X \times Y.$$

Although application of the fuzzy theory appears simple at the first sight, it is in fact very complex and the appropriate computer support is absolutely necessary.

### **References**

- A.1 Zadeh L.A.: "Fuzzy Sets", Information and Control, 1965, Vol. 8, pp. 338-357.

## Appendix B Review – Research in Progress in USA

From <http://www.eng.auburn.edu/center/ncat/rescprog.htm>

The following research projects pertaining to hot mix asphalt (HMA) pavements are in progress.

STATE	Florida	NCHRP
PROJECT	Implementation of SHRP Indirect Tension Tester to Mitigate Cracking in Asphalt Pavements & Overlays	Testing and Inspection Levels for Hot-Mix Asphalt Concrete Overlay (NCHRP 10-30A)
RESEARCHER(S)	Roque, Univ. of Florida	Russell, University of Wisconsin
COST	330,000	260,000
COMPLETION DATE	1999	1998
OBJECTIVES	Provide FDOT with a practical & effective system to obtain & specify relevant asphalt mixture properties needed to design crack resistant pavements & overla ys.	Develop a rational method for determining the minimum level of both agency and contractor testing and inspection activities



## **FHWA 97 – CD**

From C:\FHWA- 97\research\hpc2\contnt.htm

### **6.1.2 Bridge Deck Overlays**

#### **6.1.2.1 Washington Overlays**

Twelve concrete bridge decks were rehabilitated and/or protected with latex-modified concrete (LMC) and low-slump dense concrete overlays in the State of Washington. These decks were evaluated by Babaei and Hawkins [1990] to identify the factors that have affected the serviceability of the overlaid bridge decks. The evaluation included overlay freeze-thaw scaling, surface wear and skid resistance, surface cracking, bond with the underlying deck, chloride and water intrusion, and the overlay's ability to retard continued reinforcing steel corrosion. The results of the evaluation indicate that, regardless of concrete deterioration caused by reinforcing steel corrosion, concrete overlaid bridge decks will require resurfacing after about 25 years of service, as a result of traffic action and weathering. Typical forms of distress are freeze-thaw scaling, extensive wear in wheel lines, lack of skid resistance, and the loss of overlay bond. Concrete overlays are resistant but not impermeable to chloride infiltration. If the overlay surface is without cracking, it is an indication that corrosion of steel reinforcement in the salt-contaminated underlying deck is less extensive.

#### **6.1.2.2 Virginia Overlays**

An alternate to LMC often used by the departments of transportation is a dense concrete containing silica fume. In Virginia, a two-lane, four-span bridge deck was overlaid by such a concrete with addition of silica fume at 7% or 10% by weight of cement. Test results [Ozyildirim 1993] indicate that the concrete bonds well to the base concrete and has a very low permeability, high strength, and satisfactory freeze-thaw resistance. Over a 5-year evaluation period in the field, there was evidence of cracking and increase in half-cell potentials and chloride content, indicating a tendency to corrosion. However, the same evidence was observed with LMC overlays. Thus silica fume concrete can be used effectively as an alternate to LMC. Just like LMC, plastic shrinkage is recognized as a potential problem

with silica fume concrete. Therefore, immediate and proper curing after placement is essential.

#### **6.1.2.3 Oregon Overlays**

A similar study was conducted in Oregon using a microsilica-modified concrete [Miller 1991]. Seven concrete bridge decks were covered with microsilica concrete in 1989. After one year in service, cracking and delamination were observed in the overlays. However, the cracks and delaminations were not extensive (the worst deck had only 2.5% of its surface delaminated) and comparable to what had also been observed in LMC overlays. More serious crackings and delaminations were observed near construction and expansion joints. The only maintenance performed was the sealing of cracks on one deck with methacrylate and sand at a cost of \$4,000. The sealant was effective. The overlay met two of their three design objectives after one year in service. They were adding strength to the deck and providing a smooth and durable wearing surface. However, because of crackings, they could no longer seal the underlying deck from the intrusion of chlorides.

#### **6.1.2.4 Polymer Concrete Overlays**

Sprinkel [1993] reviewed the status of polymer concrete overlays for concrete bridge decks, and provided information on the properties of the concretes used, proper application methods, and the performance record of the overlays. He pointed out that polymer overlays constructed with epoxy, methacrylate, and polyester styrene binders and graded silica and basalt aggregates can provide skid resistance and protection against chloride intrusion for 1 to 20 years. They are an economical technique for extending the service life of reinforced concrete decks, especially when the overlays must be constructed during off-peak traffic periods to minimize inconvenience to the travelling public.

## **FHWA April 1, 1996**

From <http://www.hend.com/6005/designated/tbo/status/tboframe.htm>

### **Thin Bonded Overlay and surface Laminations**

The purpose of the Thin Bonded Overlay (TBO) and Surface Lamination project is to demonstrate technologies for surface rehabilitation using improved materials. The ultimate goal is to reduce life-cycle costs of pavements and bridge decks by extending their useful life.

Section 6005 (a) (e) (7) of the Intermodal Surface Transportation Efficiency Act of 1991 designated \$2.5 million for TBO for each fiscal year through 1997. Thirty-three projects have been approved in seventeen states through FY96. Seven TBO pavement projects and fifteen bridge deck projects have been constructed through 1995.

For fiscal year 1996, three pavement projects and eight-bridge deck projects are under construction. For fiscal year 1997, three pavement projects and five-bridge deck projects were planned for construction.

TBO projects were first solicited in February 1993 and again in July 1994. Project selection priorities were graded by:

- Promising technologies,
- Compatibility with National Technology priorities and the Strategic Highway Research Program
- Specific Pavement Studies experiment on bonded concrete pavement overlays (SPS-7), and Willingness and enthusiasm of State DOT's to participate.

Technologies: A large number of technologies for overlays and surface laminates are being investigated both individually and in combinations, under this program. The technologies include: Silica fume, fly ash, corrosion inhibitors, both steel and polymer reinforcing fibers, and polymer additives.

## **Project Solicitations**

*First Solicitation.* The first round of thin bonded overlay and surface lamination (TBO) projects was solicited in February 1993. Nine highway pavements and 14 bridge deck projects were originally selected. The status of these projects will be given in this report.

*Second Solicitation.* The solicitation for the second round of projects was distributed in July 1994. States were invited to submit proposals for fiscal year (FY) 1995-97 funding. The following technologies/systems were given priority for funding:

*Bridge Deck Overlays.* Ceramic concrete, also known as Z concrete; concrete mixes (concrete and corrosion inhibitors; concrete and silica fume; concrete, silica fume, and corrosion inhibitors; concrete with high-volume fly ash; and concrete with slag); and the salviacim process, also known as resin-modified concrete or APCCC.

**Bridge Deck Projects by Type of Overlay (All Current Projects) FY 1996 – 1997**

<b>Type</b>	<b>State</b>
13 mm (0.5 in) fiber concrete	IL
Concrete w/silica fume	WY
Concrete w/silica fume	WY
Dense Concrete	TX
Dense Concrete	TX
Latex-modified Concrete	ID
Flexolith	OR
GranCem Slag Cement	KS
Microlite	WV
Microsilica concrete w/MCI	OR
Microsilica w/steel fibers, Latex, Slag, HSC	DE
Silica Fume	OK
Silica Fume	LA
Silica Fume	LA
Latex-modified concrete	NC
Silica fume concrete w/polypropylene	ID
Silica fume concrete w/polypropylene	ID
Silica Fume w/polypropylene fibers	KS
Silica Fume w/steel fibers	KS
15% Silica Fume 20 mm (3/4in), slag, fly ash, corrosion inhibitor (CI)-calcium nitrite, CI--Rheocrete, CI--Armatec, silica fume (control)	VA
Slag/Silica Fume	AR
Slag/Silica Fume	AR
Thoropatch HCR	WV
Specrete (Microlite)	OH
Specrete (Microlite)	OH
Thorotop HCR, Flexolith, Methyl Methacrylate (surface laminates) Silica fume( 50 mm overlay)	MT
Type 1P Cement	KS

From <http://www.fhwa.dot.gov/engineering/hng40/spproj.htm#ISTEA> of 1991, Section 6005(e)7,

**Thin Bonded Overlay and Surface Lamination (Pavements and Bridges)**

Thin Bonded Overlays or Surface Laminates of Bridge Decks - There was major interest in constructing bridge deck overlays as a part of this program. This reflects the corrosion problem on many existing bridge decks and the importance of using thin overlays to reduce dead loads on the bridges. Under this program, about 28 projects have been constructed or have been approved in the States of DE, ID(3), IL, KS(4), LA(2), MT, NC, NY, OH(2), OK, OR(3), TX(2), VA(2), WV(2), and WY(3). A wide variety of products including slag, silica fume, fibers, and corrosion inhibitors were tested. Very few construction and evaluation reports have been received for these projects at this time.

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## **Bridge and Structures Related Research – Summary**

### **CONCRETE BRIDGE RESEARCH**

The focus areas in concrete research are High Performance Concrete, and Corrosion Protection of Concrete Structures. High Performance Concrete (HPC) is a major FHWA Structures Technology Program initiative. The Program is led by the HPC Bridge-Technology Delivery Team (TDT), which was formally established in May 1997. A goal of the HPC Bridge TDT is to implement this proven technology through having at least one HPC Bridge project in every State by the Year 2002. The HPC Bridge TDT consists of FHWA staff from Research and Development, Office of Technology Applications, Office of Engineering, FHWA field offices, plus academia and the private sector. The HPC Bridge TDT works with the AASHTO HPC Lead States Team and the concrete industry's National Concrete Bridge Council to implement HPC around the Nation. The second focus area 'Corrosion Protection of Concrete Structures' is aimed at developing improved corrosion protection systems for bridge steels used in reinforced, pretensioned, and posttensioned concrete structures.

#### ***Area I: High Performance Concrete - Bridges***

1. The FHWA has initiated of number of high performance concrete (HPC) bridge projects. A number of participating states have selected one or more bridges as demonstration HPC sites where improvements in concrete durability and/or strength can be made. Each project includes some degrees of research, design, instrumentation, construction, and technology transfer. The FHWA shares funding for the HPC bridge with the individual State where the project is located. Some projects have also been partially funded by a group of 10 states which combined their research funds into a 'pooled-fund'

effort. Current projects are listed below; others are under negotiation. More information on the HPC Program can be found on the web at <http://hpc.fhwa.dot.gov/>.

- **Alabama-** The Alabama Highway 199 Bridge over Uphapee Creek in Macon County, GA is currently under construction. It will have HPC in the deck, Bulb-Tee girders and substructure.
- **Colorado-** The Interstate 25 Bridge over the Yale River in Denver, Co has been constructed. It contains HPC box beams, as well as an HPC deck and substructure. A Showcase for FHWA Region 8 states was held in February, 1998.
- **Georgia-** The Georgia HPC bridge on State Route 920, Jonesboro Road, in Henry County, GA has been designed. Construction of the bridge is scheduled to begin in 1999.
- **Nebraska-** The 120th Street and Giles Road Bridge was completed in 1996 in Sarpy County, NE. This bridge incorporates HPC in the deck and girders. A Showcase was held for FHWA Region 7 states in November, 1996.
- **New Hampshire-** The Route 104 Bridge crossing the Newfound River at Bristol, NH was completed in 1996. This bridge also incorporates HPC into the deck and girders. A Showcase for FHWA Region 1 states was held in September 22-23, 1997. A second HPC bridge, the Route 3A Bridge over the Newfound River at Bristol, NH is under construction.
- **New Mexico-** Up to two bridges on Interstate 10 will be designed and constructed with HPC. The bridges are currently in the design phase.
- **North Carolina-** The U.S. 401 Bridge over the Neuse River in Wake County, NC is currently under design.
- **Ohio-** The State Route 22 Bridge at Mile Post 6.57 in Guernsey County, Ohio is currently under construction. Structural experimentation on HPC box beams was



conducted during 1997 and 1998. A showcase for surrounding states will be held February 23-24 1999.

- **South Dakota**- HPC is being used in new bridge decks on two bridges in Sioux Falls, one bridge with steel girders and one bridge with prestressed concrete girders. HPC girders and decks will also be used in a pair of bridges on I29 over a railroad near Sioux Falls.
- **Tennessee**- A new bridge is under construction which utilizes HPC in the deck and girders.
- **Texas**- The Louetta Road Overpass near Houston, TX and the San Angelo Bridge in San Angelo, TX are both HPC bridges. Both incorporate HPC in the deck, girders and substructure. A Showcase was held in Houston, TX during March 1996 to transfer this technology to a regional and national audience. Construction was completed on the San Angelo Bridge in 1997 and on the Louetta Road Overpass in 1998. Live load tests were conducted for both bridges to document performance and create a baseline for future measurements. Publications are being prepared by TX DOT and the University of Texas researchers on the bridge projects. The Louetta Road Overpass won a design award in 1998 from the Precast/Prestressed Concrete Institute.
- **Virginia**- The Virginia DOT plans to build 14 bridges utilizing HPC in the deck and/or the superstructure. A number of these bridges have been completed. The Richlands Bridge near Bristol, VA will be the first one in Virginia to utilize the 0.6-in diameter prestressing strands in the HPC girders. A Showcase to transfer this technology for FHWA Region 3 states was held in June 24-26, 1997.
- **Washington**- The Eastbound State Route 18 Bridge over State Route 516 in King County, WA has been constructed. A Showcase for FHWA Region 10 states was held in August 18-20, 1997.

## ***Area II: Corrosion Protection of Concrete Structures***

### **1. Corrosion Protection Systems for Bridges in Corrosive Environment**

- - Columbus Technologies. The objectives are to define the concentration of species and environmental conditions corrosive to reinforcing and prestressing steel. This research focuses on two main issues: (1) to define the desired physical and chemical properties that must be possessed by concrete for patching and overlays (rehabilitation), and (2) to define the desired physical and chemical properties required by concrete in new construction if it is to avoid the corrosion-induced concrete deterioration in the presence of well defined corrosive conditions. The research is divided into three major tasks: Task A - Corrosive environments studies; Task B - Concrete chemical and physical properties and Task C - Long-term corrosion performance. An interim report containing results of Tasks A and B is available.

### **2. Corrosion Resistant Reinforcement for Concrete Components**

-Wiss, Janey, Elstner Associates, Inc. The objective is to develop a cost-effective new breed of bridge coatings (e.g., organic, inorganic, ceramics, metallic) and metallic alloys (as reinforcement) which are significantly more corrosion-resistant than the fusion-bonded, epoxy-coated reinforcement presently used for reinforced concrete (R/C) bridge application in adverse environments. Three interim reports listed below were published after completion of the screening tests. Based on the screening tests on 57 different organic, inorganic, ceramic, metallic-clad and alloyed rebars; 11 rebar types were tested in-concrete for a period of 96 weeks. Significant results after 96 weeks of in-concrete testing are as follow:

- Previous research, field data and research under progress all indicate better performance when both reinforcing mats are of the same material over black rebar in the bottom mat.

- The use of steel surface chemical pretreatments did not appear to increase the corrosion performance for epoxy-coated rebars (ECR) when bottom mat was black steel.
- Stainless steel 316 showed excellent performance in concrete after 96 weeks of ponding (Cl<sup>-</sup> concentrate at steel level of about 20 lbs/yd<sup>3</sup>). Stainless 304 showed some corrosion in the rebar and a measurable macro cell current. Hence, 316 stainless steel appears to perform much better than 304 stainless steel.
- In general, copper-clad rebars have shown good performance and are comparable or better than ECR.
- Not surprisingly, defects and holidays diminish the ECR performance.
- In general, bent bars (both ECR and metallic-coated) showed inferior performance over straight bars.
- Short term cathodic disbondment, hot water immersion, and salt spray screening tests for ECR did not correlate well when tested in fabricated reinforced concrete specimens under longer duration. In summary, this five-year research study supports the continued use of epoxy-coated reinforced bars as a cost-effective corrosion protection systems for bridge decks.
- **Verification of Effectiveness of Epoxy-Coated Rebars**
  - ConCorr Inc. The objective of this regional pooled fund study is to investigate the field performance of epoxy-coated reinforcing steel in bridge decks in the States of Pennsylvania and New York. To accomplish the stated objective, 240 cores were extracted from 80 selected bridge decks. The 80 bridges were visually surveyed. The extracted cores were tested in the laboratory for permeability of concrete, chloride content at rebar level, and detailed evaluation of extracted ECR segments with regard to holidays, coating thickness, adhesion loss, and observed corrosion on bare areas and under coatings. Based on the statistical analysis of laboratory and field visual survey data; ECR appears to be an effective corrosion protection system for bridge decks.

- **Improvement in Graphite Reference Cells for Reinforced Concrete-SBIR Phase II**

- ConCorr Inc. The primary objectives of this study was to further develop the modified graphite electrode, test its long term performance, and identify the best graphite and the modification process that can provide the reproducibility desired for low-cost man production. Final report on this study has been completed.

- **Rehabilitation of Prestressed Concrete Bridge Components by Non-electrical (conventional) Methods**

- Construction Technology Laboratories. This study focused on conventional technology and developed methods for the rehabilitating salt contaminated prestressed concrete (PS/C) bridge members without using cathodic protection. The study identified suitable materials and evaluated them to reduce corrosion of existing prestressing steel in PS/C bridge members. Based on 3-years of laboratory testing, indications are that conventional repairs are temporary measures, and do not appear to ensure long-term protection of the prestressing steel and associated conventional reinforcement. The final report will be available in January 1999.

- **Magnetic-Based System for NDE of Prestressing Steel in Pre-Tensioned and Post-Tensioned Concrete Bridges**

- The objective of this study is to design, fabricate and demonstrate a nondestructive evaluation (NDE) system based upon principles of magnetic field variation for detecting and locating imperfections such as corrosion and fracture of steel elements in reinforced, pre-tensioned, and post-tensioned concrete bridge members. Interim design report includes a brief summary of the technical developments in the area of magnetic flux leakage and the proposed refinements for fabricating an automated NDE systems consisting of (a) basic sensing unit; (b) Beam-Rider unit; (c) Push Cast unit for horizontal surfaces; and (d) Pier Inspection system. A fabricated system was recently tested in the

Milwaukee area on selected pre-tensioned, prestressed AASHTO beams. This demonstration was very successful as the system performed as designed.

- **Development of an Embeddable Micro instruments for Reinforced Concrete**

- Virginia Transportation Research Council and University of Virginia. The objective of this study is to develop an inexpensive micro-instrument that will serve as an integrated, intelligent system for performing electrochemical measurements such as rate of corrosion of embedded steel, chloride at steel level, and pH of concrete at steel interface. This micro-instrument circuit board has been fabricated and has been successfully tested for rate of corrosion of black steel in simulated concrete pore water solution contaminated with chloride ions. The results were presented at the National Association of Corrosion Engineers (NACE) annual meeting, Corrosion/97. The paper presented at the meeting is available. Another paper has been accepted for presentation in the ICCRRCS, 1998 at Orlando, Florida.

- **Investigation of Some Issues Related to Electrochemical Chloride Extraction (ECE) from Reinforced Concrete**

- Virginia Transportation Research Council and University of Virginia. The objectives of this investigation are: (1) to develop a set of definite procedures for estimating, before an ECE treatment is applied to a concrete structure, the optimum treatment time for the structure; and (2) to formulate a model with which the expected beneficial life of the treatment for a concrete structure can be estimated.

- **Field Evaluation of Corrosion Inhibitors for Concrete**

- Virginia Transportation Research Council. The objective of this national pooled fund study is to evaluate the effectiveness of available corrosion inhibitors for rehabilitation and repair of salt-contaminated reinforced concrete bridge members. Specifically, the study is now evaluating various corrosion inhibitors for:

- Ability to mitigate corrosion in short-term and long-term repairs;

- Effect on behavior of anodic regions around repairs; and
- Compatibility with portland cement-based repair mortar and concrete mixtures.

Both laboratory and field studies will be performed to evaluate the above parameters.

- **Corrosion Inhibitors in Concrete**

- Florida Department of Transportation and University of South Florida. The objective of this study is to evaluate the use of corrosion inhibitors mainly for new construction with limited effort on rehabilitation. The study has the following objectives:

- Establish the use of corrosion inhibitors for new construction corrosion prevention and evaluate the ability of the corrosion inhibitor to be physically present at the rebar surface after long periods (30-50 years) following construction;
- Estimate the effectiveness of the corrosion inhibitor after 30-50 years of aging in concrete;
- Determine the extent of possible negative side effects of the corrosion inhibitor presence;
- Quantitatively assess the extension of corrosion initiation and propagation periods as a function of amount of corrosion inhibitor introduced; and
- Establish the suitability of the corrosion inhibitor for rehabilitation of corroding structures.

- **Prediction of Chloride Penetration in Concrete**

- University of Toronto (Dr. S. Hooton and Dr. Thomas). The broad objective of this research is to develop a new method or methods for predicting chloride ion penetration into portland cement concrete contained in bridges and pavements, and to correlate the results from this new test or tests with a long-term ponding test. The new method(s) will be utilized for predicting resistance of concrete to chloride ion penetration for use

in evaluating new mixes, accepting or rejecting new concrete according to specifications, and evaluating in-place concrete. This study began in September, 1997. (FHWA Contacts: Susan N. Lane & Marcia Simon).

From <http://hpc.fhwa.dot.gov/presentations/Nebraska/aitcinHPC.htm>

## **The Use of High-performance Concrete in Bridge Construction**

### WHAT IS HPC?

HPC can be broadly defined as low water/cement (W/C) or low water/binder (W/B) ratio concrete. The term binder being used to qualify any mixture of portland cement and fly ash, slag, silica fume, limestone or silica filler, rice-husk ash, ground pumice, diatomaceous earth, or any two or three of these finely divided materials. The water/cement or water/binder ratio of the most widely used HPC falls between 0.30 and 0.35 (1, 2, 3, 4, 5), but some HPCs with W/C or W/B ratios lower than 0.30 have been used (6, 7). Usual concretes, also called normal-strength concretes (NSC), usually have water/cement ratios in excess of 0.45.

### ADVANTAGES OF USING HPC

HPC provides a number of technical advantages over usual concrete in many applications; HPC is stronger, stiffer and more durable. It offers higher resistance to abrasion, deicing salts, scaling, and chemicals. Despite its higher unit cost, on the order of 50% when compared to 30 to 40 MPa (4500 to 6000 psi) concrete, the initial cost of a structure can be decreased when HPC is used, as it was shown during the construction of Highway 50 bridge near Mirabel Airport (2). In some cases, the social benefits make HPC much more attractive than usual concrete by shortening significantly the construction or repair time (5). While it may be rather difficult to give any general rule on the economical advantages of HPC, it must be remembered that the unit cost of any concrete represents only a fraction of the price that has to be paid to get a certain level of strength in a particular structural element.

### DURABILITY OF HPC

The durability of any concrete is closely linked to its permeability. Permeability and ease of ingress of aggressive external agents are directly proportional. Therefore, making concrete durable means making it impermeable, that is, with a low W/C or W/B ratio. Impermeability must be followed by proper curing to prevent shrinkage cracking. Appropriate water curing can help prevent producing an impervious concrete in between two cracks.



## USE OF HPC IN BRIDGE CONSTRUCTION IN CANADA

Use of High Performance Concrete (HPC) for bridge construction is quite recent in Canada. It started in 1991 and is developing rapidly. The experience gained from the construction of these first HPC bridges showed that there is no problem to make and place consistently a 60 MPa (9000 psi) air-entrained concrete complying to very tight specifications if the ready mix producer is well prepared, no matter what is the ambient temperature.

All these bridges were built using:

- air-entrained HPC having almost all the time a design strength of 60 MPa (9000 psi), but in fact the average strength measured from field specimens was comprised all the time between 70 and 85 MPa (10 000 and 12 500 psi);
- different brands of a blended silica fume cement containing 7.5 to 8.5 % silica fume;
- an average site slump of 180 mm (7 in.);
- a HPC having a maximum delivery temperature of 200°C (700°F) in summer conditions and 250°C (800°F) in fall conditions necessitating the use of crushed ice in summer and hot water in late fall;
- most of HPCs were placed using buckets in order to keep the spacing factor (-L) below 230 Jim (0.08 in.) value except for the P.E.I. and Willow Creek bridges where pumps were used. At the beginning, the use of HPC was selected to improve the durability of the bridges but, the construction of the Montée St-Rémi viaduct proved that the use of HPC resulted in a 5 percent reduction of the initial cost of the structure as shown in Figure 1. In fact, this viaduct was one of three viaducts having respectively 31, 41 and 52 m (100, 135 and 170 ft) spans that, were built by the same contractor on the same project. The shorter and longer viaducts were designed and built with a 35 MPa (5000 psi) concrete while the 41 m (170 ft) one was built with HPC. When these 3 viaducts were completed the final unit cost of 1 m<sup>2</sup> (1 sq. yd) of bridge deck was compared in each case. Assuming a factor equal to 1.00 to the shorter span, it was found that the cost of 1 m<sup>2</sup> (≈1 sq. yd) of HPC bridge deck was only

0.98 instead of the 1.03 it should have cost if it had been built with a 35 MPa (5000 psi) concrete.

The savings resulted from a significant reduction of the columns of concrete used to build the footings, the abutments and the bridge deck as well as a significant reduction of the amount of steel reinforcement. The 35 MPa concrete was priced at \$100 CDN/m<sup>3</sup> (\$40 US/cu. yd) while the 60 MPa (9000 psi) was priced at \$130 CDN/m<sup>3</sup> (\$55 US/cu. yd) including the cost of the crushed ice used to cool the HPC (\$ 9.50/m<sup>3</sup>).

#### DURABILITY OF HPC

Nobody contest that HPCs are more durable than NSCs for bridge construction due to their very compact microstructure as clearly shown by ASTM C1202 test, also known as a rapid chloride ion permeability test. But freeze-thaw durability is still a matter of controversy. There is not a general consensus on the three following questions:

- 1° Is entrained-air necessary to ensure freeze -thaw durability of HPC?
- 2° If entrained-air is necessary, what is the critical spacing factor (C) that makes a HPC freeze-thaw durable?
- 3° Is ASTM C666 Procedure A a fair testing method to decide, if a HPC is freeze-thaw durable ?

From <http://www.aashtoware.org/products/darwin/overview.htm>

## **The Evaluation of Pavement Design**

DARWin 3.0

### **Overlay Design Module**

Seven different overlay types can be designed in the Overlay Design Module: AC overlay of AC pavement, AC overlay of fractured PCC pavement, AC overlay of PCC pavement, AC overlay of AC/PCC pavement, bonded PCC overlay, unbonded PCC overlay, and PCC overlay of AC pavement. Each overlay can be designed using up to three different methods—component analysis, nondestructive testing, and remaining life. The Overlay Design Module also contains analytical routines to perform backcalculation and calculate deflection load transfer using falling weight deflectometer (FWD) data.

### **Flexible Structural Design Module**

### **Rigid Structural Design Module**

### **Life Cycle Cost Module**

## **Appendix C      Corrosion Inhibitors**

- European Committee for Standardization (CEN) offers the final draft (pr ENV 1504-9) as general principles for the use of products and systems in repair works [C.1]. The same strategy could be used for the protection of new structures.
- The Italian Standard UNI 9747, “Corrosion of reinforcing steel in concrete in aggressive conditions – Intervention and prevention methods”, among others takes also the use of corrosion inhibitors into account
- ACI 222 R-89 has also mentioned the application of corrosion inhibitor.

### **Definition of corrosion inhibitor**

- A corrosion inhibitor for metal in concrete is a chemical substance that reduces the corrosion of the metal without reducing the concentration of the corrosive agent [C.3].
- A corrosion-inhibiting admixture is a chemical compounds which, when added in small concentrations to concrete or mortar, effectively checks or retards corrosion [C.4].

### **Types of Corrosion Inhibitors**

According to Fischer [C.5]:

1. Interface inhibitors, which decrease the velocity of physical, electrochemical and /or chemical processes of electrode reactions taking place immediately at the metal/ electrolyte interface.
2. Electrolyte – Layer inhibitors, which may decrease the velocity of physical and chemical processes of the electrode reactions, is caused by substances dispersed or dissolved in the electrolyte layer.

According to Fischer, but depending upon the origin of the protective species [C.5]:

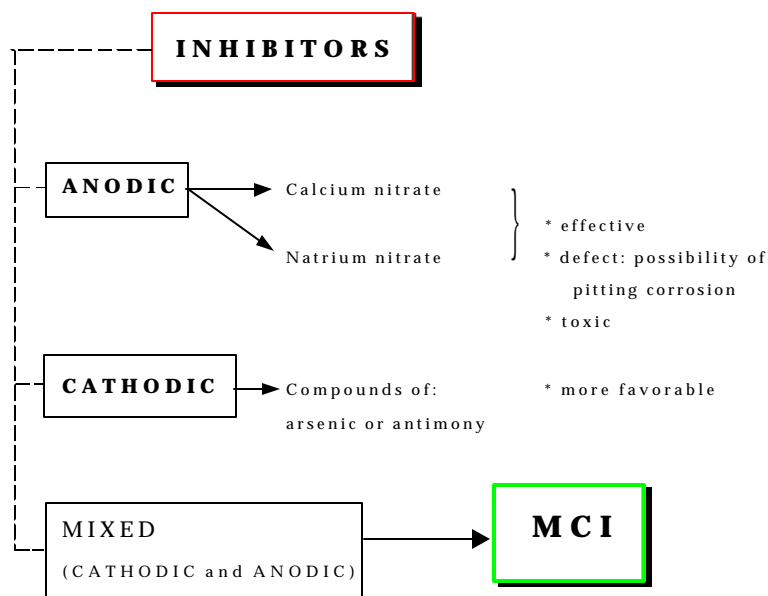
1. Primary inhibitors, which are substances that are present a priori in the bulk of the electrolyte layer without chemical change in their composition.
2. Secondary inhibitors, which are the substances that are not a priori present in the bulk of the electrolyte. They must be generated at the interface or in the electrolyte layer by a chemical or electrochemical process.

According to ACI 222 R-89, corrosion inhibiting admixtures can be either organic or inorganic. With respect to the effect of the corrosion inhibitor on the corrosion process, the inhibitors can be divided in the following three groups, and this is the most widely used classification:

1. Anodic
2. Cathodic
3. Mixed.

**Anodic inhibitors** prevent the metal corrosion by anodic passivation, by taking an active role in the anodic process, that is, they oxidize instead of the base metal. The most widely used anodic inhibitors are calcium/sodium nitrite and sodium chromate. A number of reports have dealt with the use of nitrite based inhibitors, started with Andrade and others [C.6-C.23]. Mortars prepared with ZnO are also used to inhibit corrosion during exposure to sea water [C.4]. If the anodic inhibitor is not present in sufficient quantity on the entire metal surface (which can be considered as possible at a defective and/or insufficient concrete homogenization), so called "pitting corrosion" can occur that can be even more dangerous than not applying the inhibitor at all. Pitting corrosion is localized corrosion and the attack could be significant, causing severe pitting.

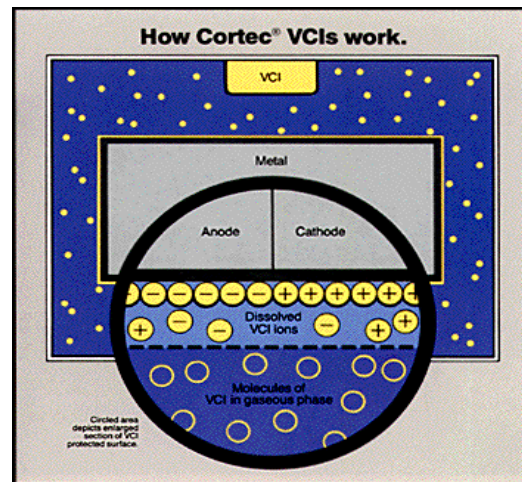
According to Ramachandran [C.4], sodium nitrite when used in cracked concrete exposed to seawater attack has shown that corrosion actually increases. Sodium nitrite is also reported to be deleterious to strength development, to cause efflorescence and also the expansion due to alkali-aggregate reaction. Berk [C.3] and other literature indicates that calcium nitrite raises the chloride threshold value at which corrosion initiates.



**Fig. C.1.** Inhibitor types.

**Cathodic inhibitors** either slows the cathodic reaction itself, or they selectively precipitate on cathodic areas to increase circuit resistance and restrict diffusion of reducible species to cathodes. Some cathodic inhibitors make the discharge of hydrogen gas more difficult and they increase the hydrogen overvoltage. Compounds of arsenic and antimony are examples of this type of inhibitor, which are often used in acids or in systems where oxygen is excluded. Other cathodic inhibitors utilize the increase in alkalinity at cathodic sites to precipitate insoluble compounds on the metal surface. The cathodic reaction, hydrogen ion and/or oxygen reduction causes the environment immediately adjacent to the cathodes to become alkaline; therefore, ions such as calcium zinc, or magnesium, may be precipitated as oxides to form a protective layer on the metal. The adsorption of cathodic inhibitor on the metal surface depends on the type of the inhibitor, that is, on the electrostatic forces of attraction between metal and the inhibitor. The insufficient quantity of the cathodic inhibitors is not a cause of pitting corrosion and this is their basic advantage over the anodic inhibitors.

**Mixed (cathodic and anodic) inhibitors** are also adsorbed on the metal surface, creating a barrier of molecular dimensions. They influence the electrode's reaction velocity at both the cathode and anode. A mixed inhibitor is usually more desirable because its effect is all encompassing, covering corrosion resulting from chloride attack as well as that due to microcell on the metal surface. Mixed inhibitors contain molecules in which electron density distribution causes the inhibitor to be attracted to both anodic and cathodic sites. There are various compounds accepted as mixed inhibitor tetramethyl phosphonium nitrite, tetraethyl phosphonium nitrite, sodium benzoate etc. These mixed corrosion inhibitors widely used in machinery industry and in technology processes are basically known as VCI inhibitors. In the structure engineering these inhibitors are also known as, Migrating Corrosion Inhibitors (MCIs).



**Fig. C.2.** Operation of VCI inhibitors.

VCI inhibitors are secondary electrolyte layer inhibitors. These layer may be a part of the double layer, the Nernst diffusion layer. Substances dispersed or dissolved in the electrolyte-layer cause electrolyte-layer inhibition. VCI also possess appreciable saturated vapor pressures under atmospheric conditions, thus allowing significant vapor phase transport of the inhibitive substance [C.5]. *MCIs are chemically similar to VCI. The protection mechanism*

*is identical [C.24].* The inhibition of cathodic process is achieved by incorporation of one or more oxidizing anions in an organic (amine) molecule of MCI.

Inhibitor molecules are hydrolyzed when adsorbed on the metal surface. The nitrogen of the amine group is capable of entering into a coordinate bond with metals, thus enhancing the adsorption process. Adsorption of cations increases the overpotential of metal ionization and slows down the corrosion. The mixed monomolecular film serves as a buffer to hold the pH at the interface in the optimum range for corrosion resistance. The inhibitor adsorption isn't a momentary process and requires much time for completion, resulting in chemisorbed layer on the metal surface. The MCI - migrating corrosion inhibitors of reinforcements' corrosion are the agents, which can be added to the concrete as the concrete admixtures during concrete mixing, or topically applied to the concrete surface. The compound will have an appreciable vapor pressure at ambient temperatures as well as the capability of forming a stable bond with the metal surface. MCI migrating inhibitors create a protective layer on the entire reinforcement surface on the basis of mixed corrosion inhibitor effect. It means that the corrosion cannot appear on the potential anodic places of the reinforcement, and they will not enable oxygen from the concrete to have access to the reinforcement on the potential cathodic places. The migration of migrating corrosion inhibitors through concrete can be calculated as stated in the reference [C.25]. All other testing results, according to migrating corrosion inhibitor effectiveness, could be find in the references [C.26-C.40].

### **Corrosion inhibitor testing**

ASTM Committee C09.0384, Committee on Corrosion Inhibitors is developing a specification for admixtures that are proposed to be use as a corrosion inhibitor. The draft of this specification requires the admixture to meet the following three requirements when tested in accordance with ASTM G-109:

- the measured average macrocell current must be less than 10 microampere at any time during the period of the test,



- the average current must be less than 2 microampere at the test completion,
- average visible corrosion measured as percentage of corroded area must be less than 15% of average corroded area in the control specimen.

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## Appendix D CORTEC Laboratories' Review [26]

### **CORTEC LABORATORIES**

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Internet <http://www.cortecvci.com>

### **Study of the Effectiveness of MCI/VCI Incorporated in Asphalt Primer Coating**

#### **Background:**

Asphalt primer coatings do not contain migrating corrosion inhibitors to protect the reinforcing steel in concrete.

**Purpose:** Investigate the ability of MCI/VCI added to an asphalt primer coating to protect the reinforcing steel in concrete.

**Methods:** Compatibility Test  
ASTM B-117 (Salt spray chamber)  
F-12 Cyclic Corrosion Test (modified)

**Materials:** CSS-1H, provided by Koch

MCI-2020

VCI-644

Panels (carbon steel SAE 1010)

Methanol

#### **Procedure:**

1. The following solutions were prepared:
  - a. CSS-1H+4% MCI-2020
  - b. CSS-1H+2% VCI-644

2. The compatibility test was performed for the above solutions at 50°C for 24 hr.
3. Panels were washed with methanol, air-dried and coated with the above solutions by dipping. The panels were then placed in the cabinet after 96 hr. of drying. The samples were inspected for signs of corrosion on a regular basis.
4. Two panels (one bare, the other coated as in step 3) were attached to the lid of one-gallon jars containing 100cm<sup>3</sup> tap water at bottom. The samples were subjected to testing of 8 hr. at room temperature and 16 hr. at 50°C for 4 cycles. The bare panels were inspected for any presence of corrosion.

**Results:**

**Compatibility Test**

<b>Material</b>	<b>Results</b>
Solution A	Compatible
Solution B	Compatible

**Protection Ability**

	<b>Time Before Corrosion (Days)</b>	<b>Presence of Corrosion</b>
<b>Material</b>	<b>ASTM B-117</b>	<b>F-12 Test</b>
Solution A	2 (Corrosion started)	No visible sign of corrosion
Solution B	>2	No visible sign of corrosion
CSS-1H	<1	Corrosion

**Conclusions:**

1. CSS-1H is compatible with MCI-2020 and VCI-644.
2. Adding MCI-2020 at a concentration level of 4% by weight of VCI-644 at a concentration level by 2% weight to the asphalt primer CSS-1H raised its ability to protect steel and also to provide vapor phase protection in the presence of chlorides.

## Appendix E Recommended Literature

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