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Investigation of the causes of deterioration of old reinforced concrete constructions and possibilities of their restoration

HEIKI ONTON

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TALLINN UNIVERSITY OF TECHNOLOGY
Faculty of Civil Engineering
Department of Structural Design

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Supervisior: Extraordinary Professor Karl Õiger, Department of Structural Design,
Tallinn University of Technology.

Opponents: Professor Ralf Lindberg, Institute of Structural Engineering,
Tampere University of Technology;
Professor Lembi-Merike Raado, Department of Building Production,
Tallinn University of Technology.

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Declaration:

Hereby I declare that this doctoral thesis, my original investigation and achievement, submitted for the doctoral degree at Tallinn University of Technology has not been submitted for any academic degree.

/Heiki Onton /

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1 INTRODUCTION

1.1 General

Concrete is the most widely used construction material in the world, and its rise to this position has played a major part in the shaping of civilisation from as long ago as 7000 BC.

Concrete is a building material composed of cement, crushed rock or gravel, sand and water, often with chemical admixtures and other materials. It was known to the Romans, the Egyptians and to even earlier Neolithic civilisations. After the collapse of the Roman Empire its secrets were almost lost, only to be rediscovered in more recent times. Indeed its modern development spans no more than 184 years - 1824 is the date on the patent for the manufacture of the first Portland cement, one of the most important milestones in concrete's history [30].

In recent years the deterioration of 20th century reinforced concrete buildings has become a significant problem. Deterioration may change the appearance of a structure, and affect its behaviour under normal working conditions or its structural safety. Before diagnosing the causes of deterioration or failure of a concrete structure, a sound understanding of the physical, chemical and mechanical actions that lead to defects is necessary. Over the years, the type and quality of concrete materials and methods of construction have varied considerably. Deterioration can result from a range of factors, including design, construction practice, materials, lack of maintenance, the environment and loadings applied to the structure. Evidence of deterioration may be visible, such as cracking or excessive deflections caused by reinforcement corrosion, fire or overloading, or may be identified during inspection.

It would be simplistic to suggest that it will be possible to identify a specific, single cause of deterioration for every symptom detected during an evaluation of a structure. In most cases, the damage detected will be the result of more than one mechanism. For example, corrosion of reinforcing steel may open cracks that allow moisture greater access to the interior of the concrete. This moisture could lead to additional damage by freezing and thawing. In spite of the complexity of several causes working simultaneously, given a basic understanding of the various damage causing mechanisms, it should be possible, in most cases, to determine the primary cause or causes of the damage seen on a particular structure and to make intelligent choices concerning selection of repair materials and methods.

Generally, only after the cause or causes are known can rational decisions be made concerning the selection of a proper method of repair and in determining how to avoid a repetition of the circumstances that led to the problem.

This doctoral thesis focuses on the causes of inadequate duration of old damaged reinforced concrete structures. Also, an estimation and analysis of residual carrying capacity is presented. As a result of the studies, possible restoration solutions on the example of the Tallinn hydroplane hangar are provided.

1.2 Aims of the research work

- To study and analyze reasons of inadequate durability of reinforced concrete in the area of old (historic) damaged reinforced concrete structures and to present an analysis of the estimation and studies.
- To investigate the technical condition of materials and structures of reinforced concrete along with the analysis of residual carrying capacity on the selected historic Tallinn and Papisaare hydroplane hangars:
  - materials studies of reinforced concrete,
  - studies and assessment of residual carriyng capacity of reinforced concrete constructions.
- To conduct an analysis of possible restoration solutions for supporting structures of Tallinn hydroplane hangars that are objects of heritage value and to describe applicability of the selected restoration solutions on those objects, taking into account the results from the studies described above.
- To study methods that are needed for assessment of structural condition of old reinforced concrete constructions and their conservation and prolong their lifetime.

Acknowledgements

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2 LITERATURE REVIEW

2.1 Durability of concrete

Durability is one of the two most important properties of concrete, the other one being compressive strength. A.M. Neville [1].

It is essential that every concrete structure should continue to perform its intended functions, that is maintain its required strength and serviceability, during the specified or traditionally expected service life.

Traditionally, reinforced concrete has been considered as a highly durable structural material requiring little or no maintenance over many decades. In fact, it is this, together with its versatility, that makes concrete the most popular structural material in many parts of the world. But, like any other material, concrete is not completely inert to chemical action or immune from physical deterioration arising from climatic changes, abrasion, damage from high velocity water, fire, impact, explosion, foundation failure or overloading.

There was an assumption that ´´strong concrete is durable concrete´´, the only special considerations being the effects of alternating freezing and thawing and some forms of chemical attack. It is now know that, for many conditions of exposure of concrete structures, both strength and durability have to be considered explicitly at the design state [1].

Concrete durability has been defined by the American Concrete Institute as its resistance to weathering action, chemical attack, abrasion and other degradation processes [12].

- **Causes of inadequate durability of concrete**

All materials deteriorate: that is not a defect but rather inherent characteristics. The total deterioration of material can expressed as a function of the environment, design, workmanship, material properties, and its use [3].

Concrete has the potential of an almost unlimited life, unless it is subjected to chemical attack by an aggressive environment or suffers some physical damage. Serious carbonation, chemical attack on the concrete, cracking and spalling due to poor quality materials or workmanship, and/or corrosion of the reinforcement are all signs of distress that indicate to a concrete of low durability.

Inadequate durability manifests itself by deterioration which can be due either to external factors or to internal causes within concrete itself. The various actions can be physical, chemical, or mechanical.

Mechanical damage is caused by impact, overload, movement, explosion or vibration.

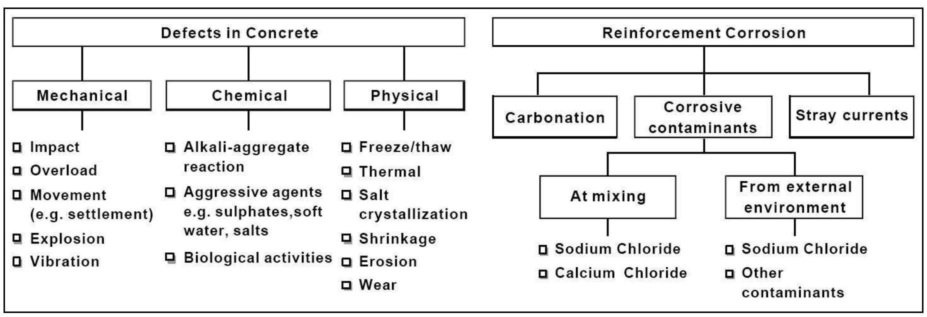
Chemical causes of deterioration include the alkali-silica and alkali carbonate reactions. External chemical attack occurs mainly through the action of aggressive ions, such as chlorides, sulfates, or of carbon dioxide, as well as many natural or industrial liquids or gases.

Physical causes of deterioration include the effects of shrinkage, erosion, wear, high temperature or of differences in the thermal expansion of the aggregate and the hardened cement paste. An important cause of damage is alternating freezing and thawing of concrete and the associated action of de-icing salts (see Table 2.1).

With the exception of mechanical damage, all the adverse influences on durability involve the transport of fluids through the concrete. There are three fluids principally relevant to durability which can enter concrete: water, pure or carrying aggressive ions, carbon dioxide, and oxygen. They can move through the concrete in different ways, but all the transport depends primarily on the structure of the hydrated cement paste.

Durability of concrete largely depends on the ease with which fluids, both liquids and gases, can enter into, and move through, the concrete; this is commonly referred to as permeability of concrete.

Table 2.1 Common causes of defects [22]



- **Permeability of concrete**

A concrete of low permeability is generally more durable than the one which is highly permeable [3].

The aspect of the structure of hardened cement paste relevant to permeability is the nature of the pore system within the bulk of the hardened cement paste and also in the zone near the interface between cement paste and the aggregate. The interface zone occupies as much as one-third to one-half of the total volume of hardened cement paste in concrete and is known to have a different microstructure from the bulk of the hardened cement paste. For these reasons, the interface zone can be expected to contribute significantly to the permeability of concrete. However, Larbi [14] found that, despite the higher porosity of the interface zone, the permeability of concrete is controlled by the hardened cement paste, which is the only continuous phase in concrete.

Larbi's views are supported by the fact that the permeability of hardened cement is not lower than that of concrete made with a similar cement paste.

The permeability of concrete is a function of its porosity as well as of the size, distribution, shape, tortuosity and continuity of the pores [3]. If the porosity is high and the pores are interconnected, they contribute to the transport of fluids through concrete so that its permeability is also high. If the pores are discontinuous or otherwise ineffective with respect to transport, then the permeability of the concrete is low, even if its porosity is high.

The pores relevant to permeability are those with a diameter of at least 120 or 160 nm. These pores have to be continuous. Pores which are ineffective with respect to flow, that is to permeability, include, in addition to discontinuous pores, those which contain adsorbed water and those which have a narrow entrance, even if the pores themselves are large (see Figure 2.1).

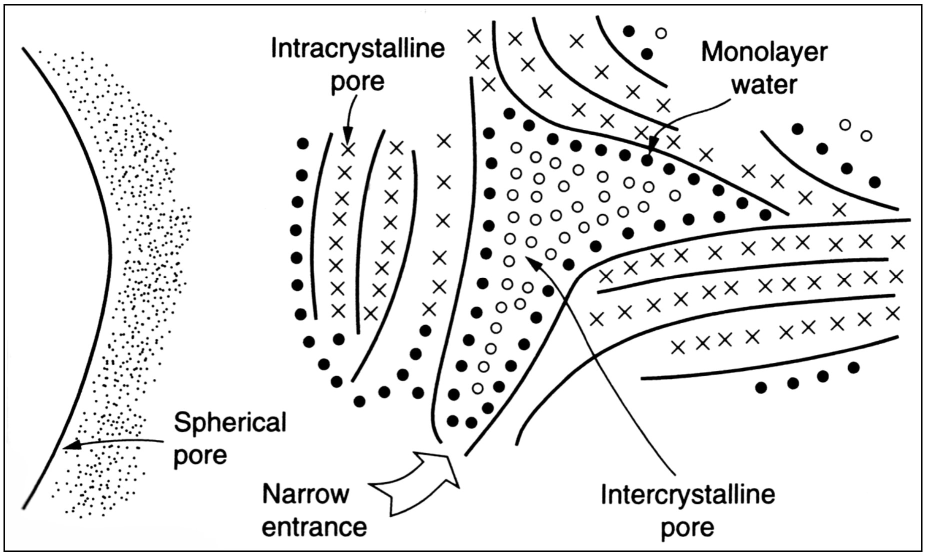


Figure 2.1 Diagrammic representation of the pore system in hydrated cement paste based on Rahman's model [13]

Aggregate can also contain pores, but these are usually discontinuous. Moreover, aggregate particles are enveloped by the cement paste so that the pores in the aggregate do not contribute to the permeability of concrete. The same applies to discrete air voids, such as entrained-air bubbles [1].

Structure of hydrated cement

Many of the mechanical properties of hardened cement and concrete appear to depend not as much on the chemical composition of the hydrated cement as on the physical structure of the products of hydration, viewed at the level of colloidal dimensions.

Four compounds listed in Table 2.2 are usually regarded as the major constituents of cement.

Tabel 2.2 Main compounds of Portland cement

Name of compound	Oxide composition	Abbreviation
Tricalcium silicate	$3CaO.SiO_2$	C_3S
Dicalcium silicate	$2CaO.SiO_2$	C_2S
Tricalcium aluminate	$3CaO.Al_2O_3$	C_3A
Tetracalcium aluminoferrite	$4CaO.Al_2O_3.Fe_2O_3$	C_4AF

There exist minor compounds, such as MgO, TiO_2, Mn_2O_3, K_2O and Na_2O; that usually amount to no more than a few per cent of the mass of cement.

A general idea of the composition of cement presented in Tabel 2.3, provides oxide composition limits of Portland cements.

Tabel 2.3 Usual composition limits of Portland cement

Oxide	Content, per cent
CaO	60 - 67
SiO_2	17 - 25
Al_2O_3	3 - 8
Fe_2O_3	0.5 - 6.0
MgO	0.5 - 4.0
Alkalis (as Na_2O)	0.3 - 1.2
SO_3	2.0 - 3.5

The pattern of formation and hydration of cement is shown schematically in Figure 2.2.

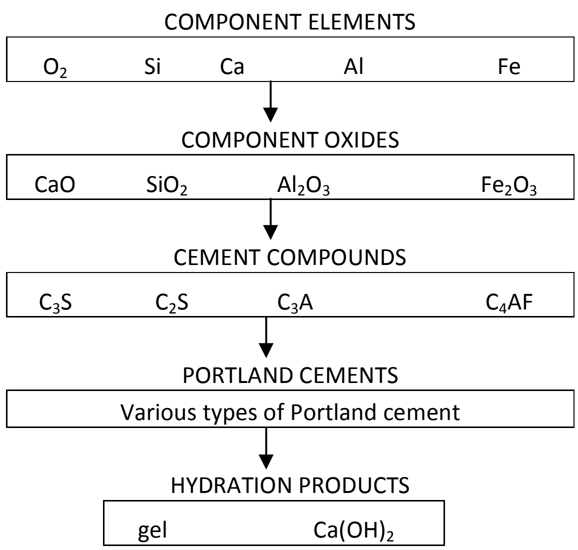


Figure 2.2 Schematic representation of the formation and hydration of Portland cement [1]

Fresh cement paste is a plastic network of cement in water, but once the paste has set, its apparent or cross volume remains approximately constant. At any stage of hydration, the hardened paste consists of very poorly crystallized hydrates of various compounds referred to collectively as gel, of crystals of $Ca(OH)_2$, some minor components, unhydrated cement, and the residue of the water-filled spaces in the fresh paste. These voids are called capillary pores but within the gel itself, there exist interstitial voids called gel pores. The nominal diameter of gel pores is about 3 nm while capillary pores are one or two orders of magnitude larger. Thus, in the hydrated paste there are two distinct classes of pores represented diagrammatically in Figure 2.3.

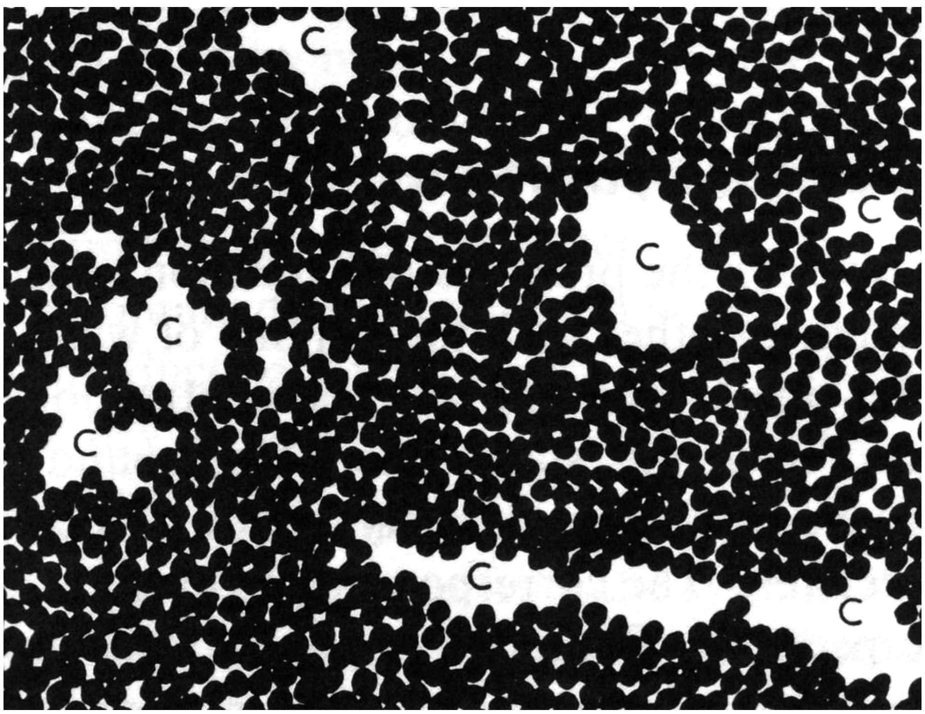


Figure 2.3 Simplified model of paste structure. Solid dots represent gel particles; interstitial spaces are gel pores; spaces such as those marked C are capillary pores. Size of a gel pore is exaggerated [1]

Structure of the cement stone is shown in Figure 2.4.

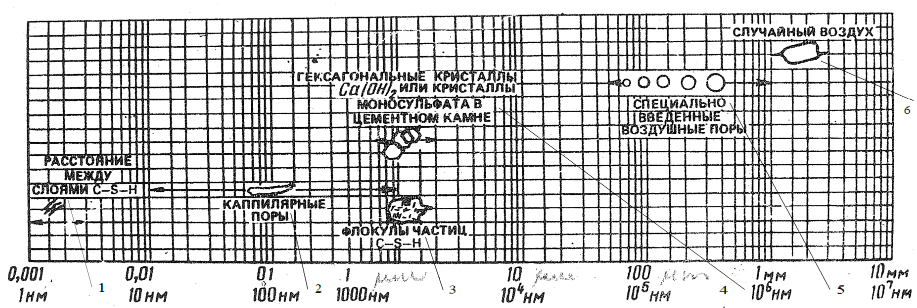


Figure 2.4 Structure of the cement stone: 1 – distance between the layers of C-S-H crystals; 2 –capillary pores; 3 – C-S-H crystals; 4 – $Ca(OH)_2$ crystals and crystals of mono sulfate; 5 – entrained air; 6 – air bubbles [10]

Hydrated calcium silicates (C-S-H)

Water permeability in concrete

The hardened cement paste is composed of particles connected over only a small fraction of their total surface. For this reason, a part of the water is within the field of force of the solid phase; it is adsorbed. This water has high viscosity but is, nevertheless, mobile and takes part in the flow. Although the cement gel has a porosity of 28 per cent, its permeability is only about $7x10^{-16}$ m/s [1]. This is due to the extremely fine texture of hardened cement paste: the pores and the solid particles are very small and numerous, whereas in rocks, the pores, though fewer in number, are much larger and lead to a higher permeability. For the same reason, water can flow more easily through the capillary pores than through the much smaller gel pores: the cement paste as a whole is 20 to 100 times more permeable than the gel itself [1]. It follows that the permeability of hardened cement paste is controlled by its capillary porosity (see Figure 2.5).

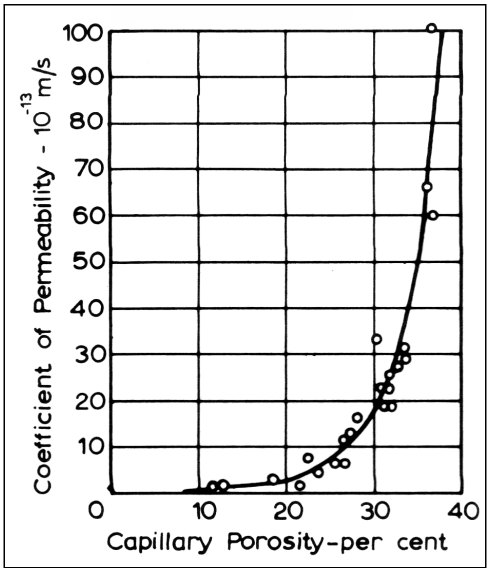


Figure 2.5 Relation between permeability and capillary porosity of cement paste [1]

For the cement pastes hydrated to the same degree, the permeability is lower the higher the cement content of the paste, the lower the water/cement ratio.

Figure 2.6 shows values obtained for pastes in which 93 percent of the cement has hydrated [1].

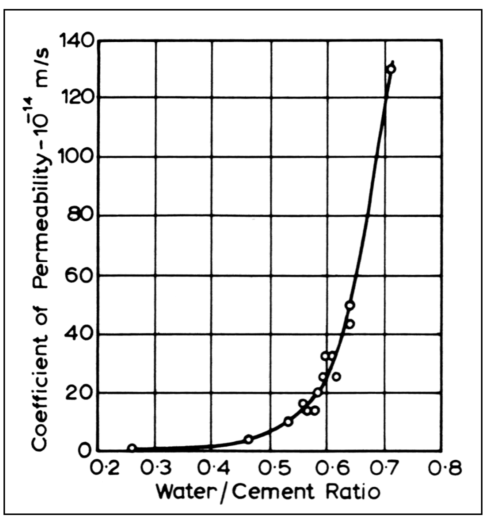


Figure 2.6 Relation between permeability and water/cement ratio for mature cement pastes (93% of cement hydrated)

The slope of the line is considerably lower for the pastes with water/cement ratios below about 0.6. Figure 2.6 shows also that a reduction of water/cement ratio, for example, from 0.7 to 0.3 lowers the coefficient of permeability by three orders of magnitude.

Permeability of concrete is also of interest in relation to water-tightness of liquid-retaining and some other structures, and also with reference to the problem of hydrostatic pressure in the interior of dams. Furthermore, ingress of moisture into concrete affects its thermal insulation properties [1].

In general terms, it is possible to say that the higher the strength of hardened cement paste the lower its permeability (because strength is a function of the relative volume of gel in the space available to it).

Air and vapour permeability

The ease in which air, some gases and water vapour can penetrate into concrete is relevant to the durability of concrete under various conditions of exposure.

When transport of a gas or a vapour through concrete is the result of a concentration gradient and not of a pressure differential, diffusion takes place. As far as the diffusion of gases is concerned, carbon dioxide and oxygen are of primary interest: the former leads to carbonation of hydrated cement paste, and the latter makes the progress of corrosion of embedded steel possible.

Papadikis [15] has presented expressions for the effective diffusion coefficient of carbon dioxide as a function of the relative humidity of the air and of the porosity of hardened cement paste or of the compressive strength of concrete. The diffusion through water is four orders of magnitude slower than through air. It should be noted that the diffusion coefficient changes with age because the pore system in concrete changes with time, especially when hydration of concrete continues.

Oxygen diffusion through concrete is strongly affected by moist curing, prolonged curing reducing the diffusion coefficient by a factor of about 6. The moisture condition of the concrete also has a large influence because water in the pores significantly reduces the diffusion; therefore air permeability is strongly affected by its moisture content [1].

Movement of water vapour through concrete can occur as a result of a humidity differential on its two opposite sides. Water vapour transmission of concrete is generally affected in a similar manner to air permeability.

According to L.H. Son [3], factors contributing to the degree of permeability of concrete include the following:

- The quality of cement: for the same water: cement ratio, coarse cement tends to produce a paste with a higher porosity than a finer cement.
- The permeability of the aggregate itself affects the behaviour of concrete: if the aggregate has a low permeability, its presence reduces the effective area over which flow can take place.
- The quality and quantity of the cement paste: the quality of cement paste depends on the amount of cement in the mix, the water: cement ratio and the degree of hydration of cement. Permeability decreases rapidly with the progress of hydration and it is also lower if the cement content of the paste is increased.
- The degree of compaction of concrete: a well-compacted concrete reduces the porosity and hence the permeability of concrete.
- The standard of curing: permeability of steam-cured concrete is generally higher than that of wet-cured concrete.
- The characteristics of any admixtures used in the mix: air-entraining is expected to increase the permeability of concrete although it reduces segregation and bleeding, and improves workability.
- The presence of cracks allows direct entry of moisture.

- **Corrosion of reinforcement**

Corrosion of reinforcement is one of the major causes of deterioration of reinforced concrete structures in many locations.

Mechanism of attack

The protective passivity layer on the surface of embedded steel, which is selfgenerated soon after the hydration of cement has started, consists of y-F_2O_3 tightly adhering to the steel. As long as that oxide film is present, the steel remains intact.

Corrosion of steel is a electro-chemical process that requires an oxidizing, moisture and electron flow within the metal. When there exists a difference in electrical potential along the steel in concrete, an electrochemical cell is set up: there from anodic and cathodic regions, connected by the electrolyte in the form of the pore water in the hardened cement paste. The positively charged ferrous ions Fe^{++} at the anode pass into the solution while the negatively charged free electrons e^- pass through the steel into the cathode where they are absorbed by the constituents of the electrolyte and combine with water and oxygen to form hydroxyl ions $(OH)^-$.

These travel through the electrolyte and combine with the ferrous ions to form ferric hydroxide which is converted by further oxidation to rust (see Figure 2.7).

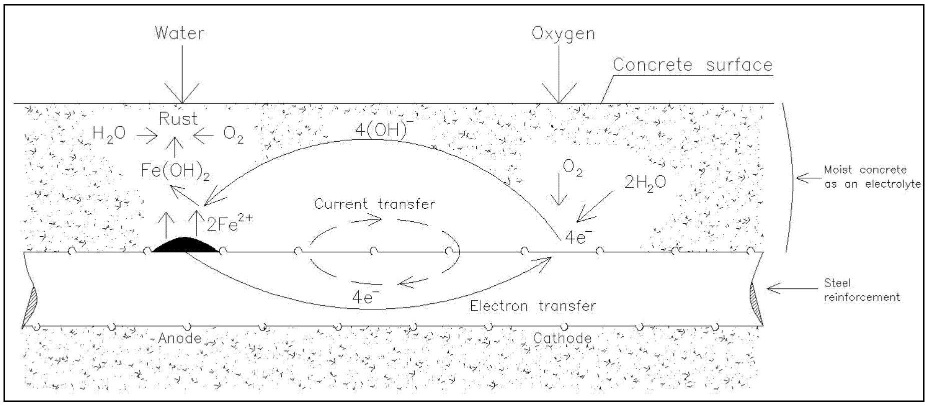


Figure 2.7 Simplified model of the corrosion process of reinforcement steel in concrete

The reactions involved are as follows:

Anodic reactions:

$$Fe \rightarrow Fe^{++} + 2e^-$$

$$Fe^{++} + 2(OH)^- \rightarrow Fe(OH)_2 \text{ (ferrous hydroxide)}$$

$$4Fe(OH)_2 + 2H_20 + O_2 \rightarrow 4Fe(OH)_3 \text{ (ferric hydroxide)}$$

Cathodic reaction:

$$4e^- + O_2 + H_2O \rightarrow 4(OH)^-$$

It can be seen that oxygen is consumed and water is regenerated but it is needed for the process to continue. Thus, there is no corrosion in, dry concrete, probably below a relative humidity of 60 per cent; nor is there corrosion in concrete fully immersed in water, except when water can entrain air, for example by wave action. The optimum relative humidity for corrosion is 70 to 80 per cent. At higher relative humidities, the diffusion of oxygen through concrete is considerably reduced.

Because the electrochemical cell requires a connection between the anode and the cathode by the pore water, as well as by the reinforcing steel itself, the pore system in hardened cement paste is a major factor influencing corrosion. In electrical terms, it is the resistance of the ´´connection´´ through concrete that controls the flow of the current. The electrical resistivity of concrete is greatly influenced by its moisture content, by the ionic composition of the pore water, and by the continuity of the pore system in the hardened cement paste.

There are two consequences of corrosion of steel. First, the products of corrosion occupy a volume several times larger than the original steel so that their formation results in cracking (characteristically parallel to the reinforcement), spalling or in delamination of concrete (see Figure 2.8). This makes it easier for aggressive agents to ingress toward the steel, with a consequent increase in the rate of corrosion.

Second, the progress of corrosion at the anode reduces the cross-sectional area of the steel, thus reducing its load-carrying capacity.

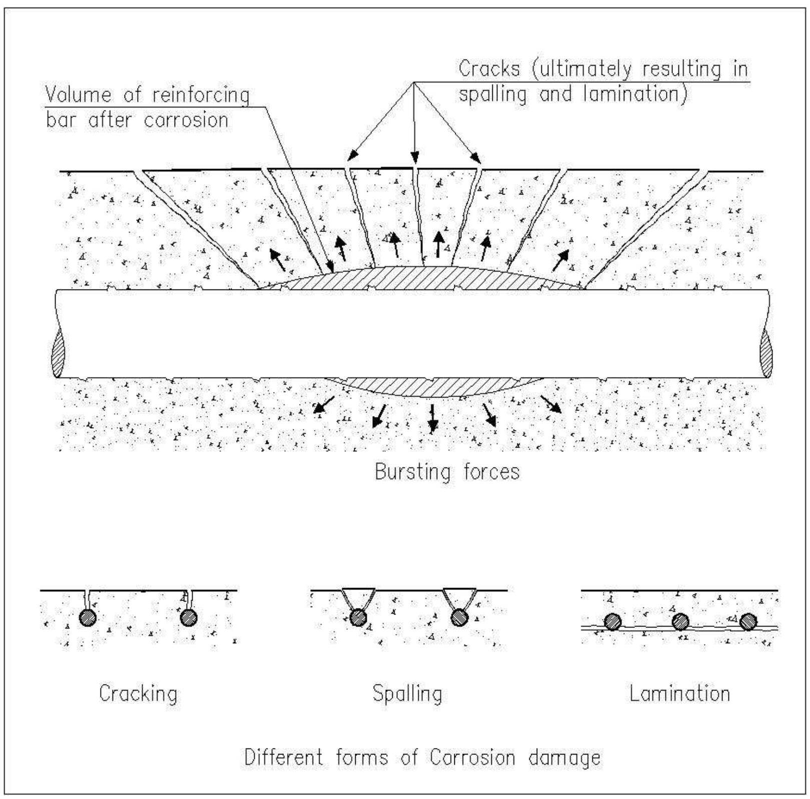


Figure 2.8 Cracking and spalling of concrete caused by corrosion of the reinforcement [3]

Protection of steel

When steel is embedded in concrete, the concrete cover provides a mechanical barrier to the movement of water and oxygen to the steel. This barrier is more or less effective depending on the quality of concrete, that is, its permeability and the thickness of the cover.

When concrete sets, the calcium hydroxide or lime produced by hydration goes into the pores of the set cement gel as a strong alkaline solution with high pH value between 12.5 and 13.5. This, in turn, forms a very thin film of oxide on the surface of the reinforcing steel and protects the latter against corrosion.

Reinforcement will not corrode as long as the high alkalinity in the concrete is maintained. This phenomenon is generally known as passivation of the steel by the alkaline concrete environment.

The corrosion reaction, however, can still proceed (see Figure 2.9) when [3]:

- Chloride or carbon dioxide penetrates through the concrete cover to the reinforcement and destroys the natural passivity provided by free lime in the hydrated cement.
- Low concrete resistivity allows electrolytic cells to be established at the steel surface.
- Oxygen penetrates through the cover to fuel the corrosion process.

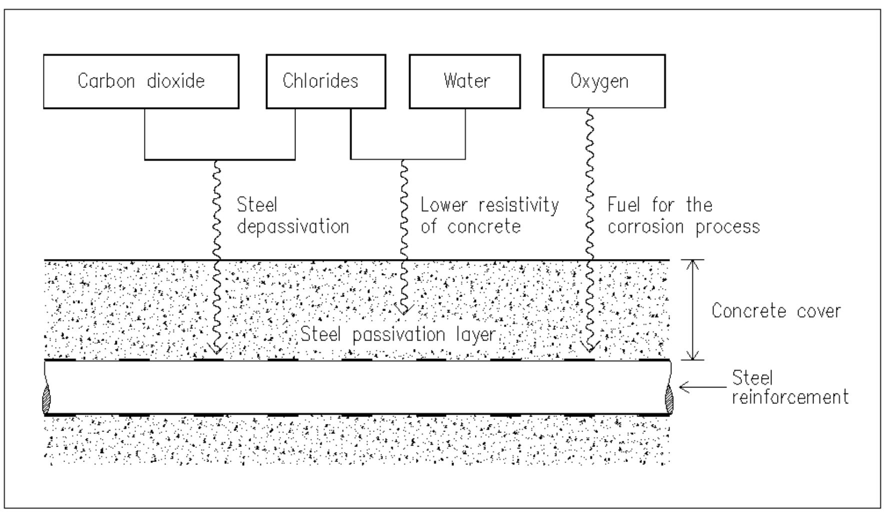


Figure 2.9 Critical environmental factors on corrosion of steel reinforcement

Carbonation of concrete

Discussion of the behaviour of concrete is generally based on the assumption that the ambient medium is air which does not react with the hydrated cement paste. In reality air contains CO_2 which, in presence of moisture, reacts with hydrated cement.

Of the hydrates in the cement paste, the one which reacts with CO_2 most readily is $Ca(OH)_2$, the product of the reaction being $CaCO_3$, but the other hydrates are also decomposed, hydrated silica, alumina, and ferric oxide being produced. In the concrete containing Portland cement only, it is solely the carbonation of $Ca(OH)_2$ that is of interest.

Effects of carbonation

Carbonation does not cause deterioration of concrete but it has important effects. With respect to durability, the importance of carbonation lies in the fact that it reduces the pH of the pore water in the hardened Portland cement paste from 12.6 to 13.5, to a value of about 9. When all of $Ca(OH)_2$ has become carbonated (see Figure 2.10), the value of pH reduces to 8.3 [1].

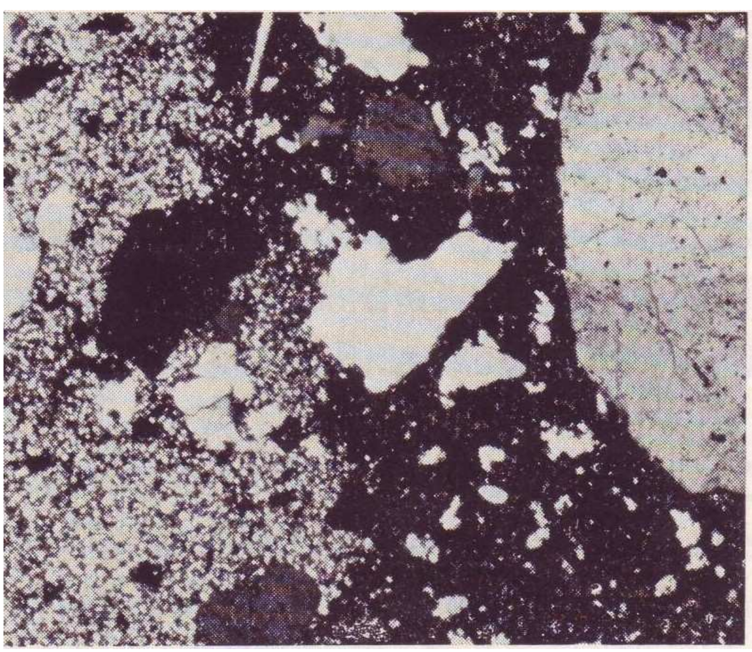


Figure 2.10 Transition over carbonated (left- white) to non carbonated zone (right–dark) of the cement stone under the microscope [8]

The significance of lowering of the pH is as follows. Steel embedded in the hydrating cement paste rapidly forms a thin passivity layer of oxide which strongly adheres to the under laying steel and gives it complete protection from reaction with oxygen and water, that is from the formation of rust or corrosion. This state of steel is known as passivation. Maintenance of passivation is conditional on an adequately high pH of the pore water in contact with the passivating layer. Thus, when the low pH reaches the vicinity of the surface of the reinforced steel, the protective oxide film is removed and corrosion can take place, provided oxygen and moisture necessary for the reactions of corrosion are present.

For this reason, it is important to know the depth of carbonation and specifically whether the carbonation front has reached the surface of the embedded steel.

It should also be noted that if cracks are present, CO_2 can ingress through them so that the front advances locally from the penetrated cracks.

Rate of carbonation

Carbonation occurs progressively from the outside of concrete exposed to CO_2, but does so at decreasing rate because CO_2 has to diffuse the pore system, including the surface zone of concrete already carbonated. Such diffusion is a slow process if the pores in the hydrated cement paste are filled with water because the diffusion of CO_2 in water is four orders of magnitude slower than in air. If there is insufficient water in the pores, CO_2 remains in gaseous form and does not react with the hydrated cement.

It follows that the rate of carbonation depends on the moisture content of the concrete, which varies with the distance from its surface. If the surface of concrete is exposed to a variable humidity, with periodic wetting, the rate of carbonation is reduced because of a slowing down of the diffusion of CO_2 through saturated pores in the hardened cement paste. Conversely, sheltered parts of a structure undergo carbonation at a faster rate than those exposed to rain, which significantly slows down the progress of carbonation. In the interior of buildings, the rates of carbonation can be high, but there are no ill consequences of this in so far as carbonation of embedded steel is concerned unless the carbonated concrete is subsequently wetted.

An example [19] of the progress of carbonation over a period of 16 years is shown in Figure 2.11.

The highest rate of carbonation occurs at a relative humidity from 50 to 70 per cent [1]

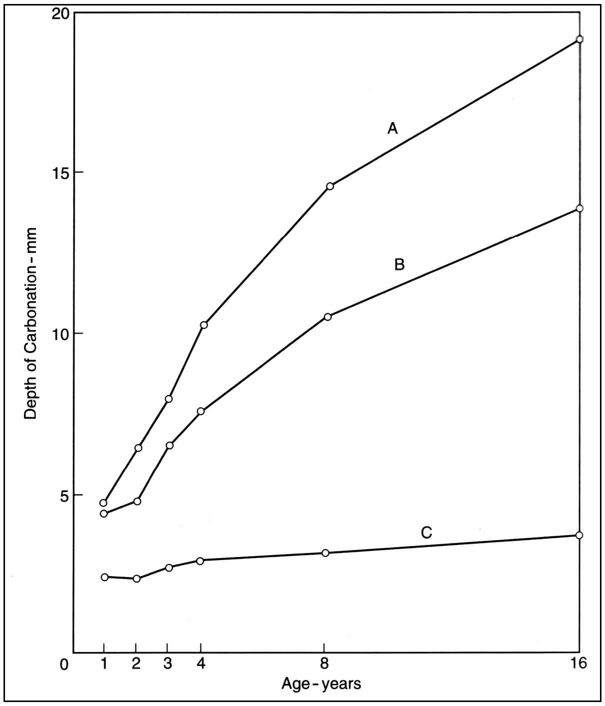


Figure 2.11 Progress of carbonation with time of exposure under different conditions: (A) 20 ^{0}C and 65% relative humidity; (B) outdoors, protected by a roof; (C) horizontal surface outdoors in Germany. The values are averages for concretes with water/cement ratios of 0.45, 0.60 and 0.80, wet-cured for 7 days (based on [19])

The rate of carbonation is also dependent on the following factors [3]:

- Concentration of carbon dioxide in the atmosphere: rate increases with increasing carbon dioxide concentration in the air.
- Condition of concrete cover: any imperfections in the cover such as segregation, poor compaction or cracking enable carbonation to progress more rapidly.

Factors influencing carbonation

The fundamental factor controlling carbonation is the diffusivity of the hardened cement paste. The diffusivity is a function of the pore system of the hardened cement paste during the period when the diffusion of CO_2 takes place. It follows that the type of cement, the water/cement ratio, and the degree of hydration are relevant. All these influence also the strength of the concrete containing any given hardened cement paste. For this reason, it is often said that the rate of carbonation is simply the function of the strength of concrete.

Alternatives to use of strength as a parameter include expressing carbonation as a function of the water/cement ratio or of the cement content. Neither strength nor water/cement ratio is informative about the microstructure of the hardened cement paste in the surface zone of concrete while the diffusion CO_2 takes place. A factor which has a great influence on the outer zone is the curing history of concrete.

Despite the considerable variability in the rate of carbonation in different locations, typical values reported by Parrott [17] and shown in tables 2.4 and 2.5 are of interest.

Table 2.4 Depth of carbonation as a function of strength [17]

Exposure	Depth of carbonation after 50 years, mm	
	25 MPa Concrete	50 MPa Concrete
Sheltered outdoors	60 to 70	20 to 30
Exposed to rain	10 to 20	1 to 2

The values of Table 2.4 must not be treated as the norm. For sheltered concrete outdoors in the United Kingdom or a similar climate, in 90% of cases, the depth of carbonation will not exceed the values shown in Table 2.5.

Table 2.5 Maximum depth of carbonation in sheltered concrete outdoors in the United Kingdom [17]

28 day strength MPa	Depth of carbonation after 30 years mm
20	45
40	17
60	5
80	2

Further aspects of carbonation

Carbonation has some positive consequences. Because $CaCO_3$ occupies a greater volume than $Ca(OH)_2$, carbonation may help the hydration of hitherto unhydrated cement. These changes are beneficial and they result in increased surface hardness, increased strength at the surface, reduced surface permeability, reduced moisture movement, and increased resistance to those forms of attack which are controlled by permeability [1].

Carbonation accelerates chloride-induced corrosion of reinfocement.

Unlike Portland cement concrete, with supersulfated cement there is a loss of strength on carbonation, but, because this applies to the surface zone of the concrete only, the loss is not structurally significant.

Because carbonation affects the porosity and also the pore size distribution (causing a decrease in the volume of pores, especially of the smaller ones) of the outer zone of concrete, the penetration of paint into concrete will vary [1]

Sakuta [16] has proposed the additives to be used that absorb carbon dioxide which has entered the concrete, thereby preventing carbonation.

Chloride attack

When the chloride concrentration exceeds of 0.4% by weight of cement if chlorides are cast into concrete and 0.2% if they diffuse in, corrosion of reinforcement is observed [18].

This concentration of chloride ions in concrete can penetrate into and break down the protective film which forms on the reinforcement in an alkaline solution.

The consequent effects are to reduce the alkalinity of the concrete to pH=6, increase the flow of corrosion currents and penetrate the passivating iron oxide film on the steel surface. Chloride ions activate the surface of the steel to form an anode, the passivated surface being the cathode. The reactions involved are as follows:

$$Fe^{++} + 2Cl^{-} \rightarrow FeCl_2$$

$$FeCl_2 + 2H_20 \rightarrow Fe(OH)_2 + 2HCl$$

Thus, Cl^- is regenerated so that the rust contains no chloride, although ferrous chloride is formed at the intermediate stage.

The anode is much smaller in area than the cathode and, as a result, the formation of the ferrous ions in the anode extends deeper and deeper, and pit is formed (see Figure 2.12).

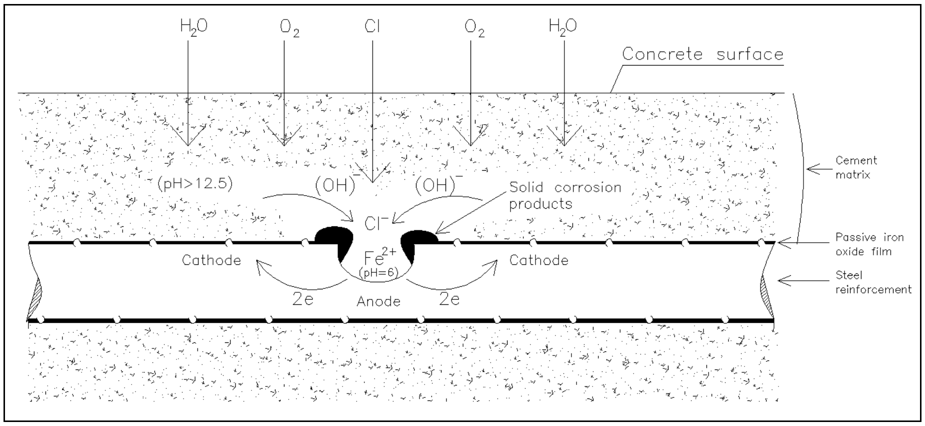


Figure 2.12 Pitting corrosion caused by chlorides

The resultant expanded products cause cracking and spalling of the surrounding concrete which, in turn, promotes chloride intrusion and further corrosion.

Only soluble chlorides are involved in the corrosion process. They are, therefore, compounded where the concrete is porous and wet.

From the fresh concrete, the mix materials may be subjected to various degrees of contamination associated with chlorides derived from the aggregates, the water, de-icing salts or the use of calcium chloride as an accelerating agent in concrete. From external environments, the problem is aggravated in highly polluted areas. Also, the presence of cracks in the concrete structure will obviously increase the penetration of chlorides and hasten the corrosion of the reinforcement.

It should be noted that, in practice, it is possible to have both carbonation and chloride attack taking place simultaneously. The risk of corrosion in this case is very much dependent on the chloride ion content and the rate of carbonation (Figure 2.13).

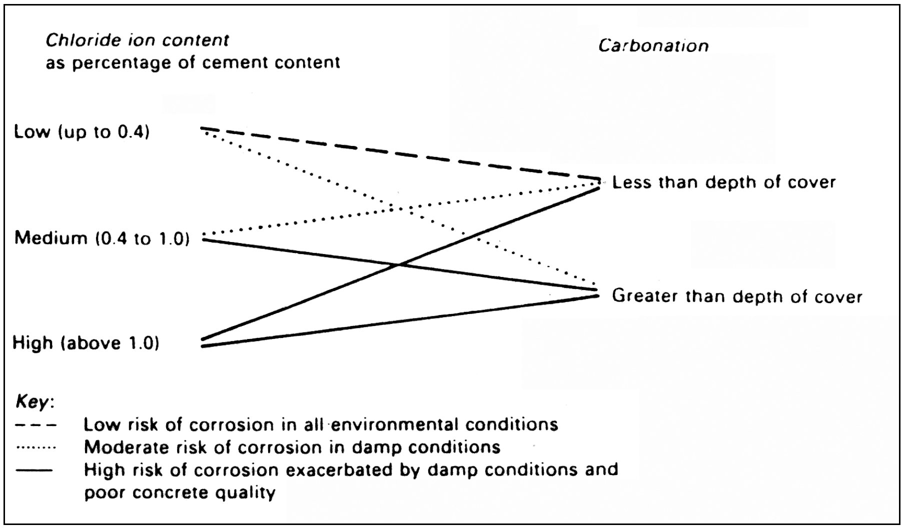


Figure 2.13 Risk of corrosion in relation to concrete analysis [3]

- **Chemical attack on concrete**

Acid attack

Concrete is generally well resistant to chemical attack, provided an appropriate mix is used and the concrete is properly compacted.

Generally speaking, chemical attack of concrete occurs by way of decomposition of the products of hydration and formation of new compounds which, if soluble, may be leached out and, if not soluble, may be disruptive in situ. The attacking compunds must be in solution. The most vulnerable cement hydrate is $Ca(OH)_2$.

A limited list of substances which attack concrete is given in Table 2.6 .

Table 2.6 A list of some substances that cause severe chemical attack of concrete [1]

Acids	
Inorganic	Organic
Carbonic	Acetic
Hydrochloric	Citric
Hydrofluoric	Formic
Nitric	Humic
Sulfuric	Lactic
Phosphoric	Tannic

Other substances

Aluminium chloride	Vegetable and animal fats
Ammonium salts	Vegetable oils
Hydrogen sulfide	Sulfates

Inorganic, or mineral, acids are manufactured products and are mainly solutions of gases in water. Hydrochloric, sulfuric, nitric and phosphoric acids produced and used in large amounts, present a more serious risk of contamination of soils and water. Only sulfuric acid may occur naturally in soils and ground water.

Organic acids are those which have their origins in nature. They are far more numereous than the inorganic acids and their effects on concrete are much more difficult to predict. Organic acids are widely distributed in nature, occuring in living plants and as decomposition products of vegetable and animal matter [2].

Concrete can by attacked by acid liquids with a pH value below 6.5 but the attack is severe only at a pH below 5.5; below 4.5, the attack is very severe. A concentration of CO_2 of 30 to 60 ppm results in a severe attack, and above 60 ppm a very severe attack occurs[3].

Sulfate attack

Solid salts do not attack concrete but, when present in solution, they can react with the hydrated cement paste. Particularly common are sulfates of sodium, potassium, magnesium, and calcium which occur in soil, groundwater, and industrial waste, sea-water, in clays or in acid rains.

The attack occurs only when the concentration of the sulfates exceeds a certain threeshold. Above that, the rate of sulfate attack increases with an increase in the strength of the solution, but beyond a concentration of about 0.5% of $MgSO_4$ or 1% of $NaSO_4$ the rate of increase in the intensity of the attack becomes smaller [1].

The reaction of the various sulfates with the hardened cement paste are follows:

1) Sodium sulfate attacks $Ca(OH)_2$:

   $Ca(OH)_2 + Na_2SO_4.10H_2O \rightarrow CaSO_4.2H_2O + 2NaOH + 8H_20.$

   In flowing water, $Ca(OH)_2$ can be completely leached out but if NaOH accumulates, equilibrium is reached, only a part of the SO_3 being deposited as gypsum.

2) Calcium sulfate attacks only calcium aluminate hydrate, forming calcium sulfoaluminate ($3CaO.Al_2O_3.3CaSO_4.32H_2O$), known as ettringite.

   The reaction with calcium aluminate hydrate can be formulated as follows:

   $$2(3CaO.Al_2O_3.12H_2O) + 3(Na_2SO_4.10H_2O) \rightarrow$$
   $$3CaO.Al_2O_3.3CaSO_4.32H_2O + 2Al(OH)_3 + 6NaOH + 17H_20.$$

3) Magnesium sulfate attacks calcium silicate hydrates as well as $Ca(OH)_2$ and calcium aluminate hydrate. The pattern of reaction is:

   $$3CaO.2SiO_2.aq + 3MgSO_4.7H_2O \rightarrow$$
   $$3CaSO_4.2H_2O + 3Mg(OH)_2 + 2SiO_2.aq + xH_2O$$

The resulting crystallized products from the reaction (ettringite) cause an increase in the volume of hardened concrete and contribute to internal disruption or spalling of the surface. The change in solid volume for reactions with sulfates can be calculated from the densities and molecular volumes given in Table 2.7 [2].

Table 2.7 Molecular volumes of concrete compounds [2; 4]

Compound	Molecular weight	Density	Molecular volume (mL)
$Ca(OH)_2$	74.1	2.23	33.2
$Mg(OH)_2$	58.3	2.38	24.5
$CaSO_4.2H_2O$	272.2	2.32	74.2
$3CaO.Al_2O_3.6H_2O$	378.2	2.52	150.1
$4CaO.Al_2O_3.19H_2O$	668.3	1.81	369.2
$3CaO.Al_2O_3.CaSO_4.12H_2O$	622.3	1.99	312.7
$3CaO.Al_2O_3.3CaSO_4.31H_2O$	1236.6	1.73	714.9

The consequences of sulfate attack include also loss of strength of concrete due to the loss of cohesion in the hydrated cement paste and of adhesion between it and the aggregate particles. Concrete attacked by sulfates has a characteristic whitish appearance.

The intensity and rate of sulfate attack depend on a number of factors, the principal ones being [3]:

- The percentage of tricalcium aluminate in the cement: rate of attack is faster when there is a higher amount of tricalcium aluminate.
- The permeability of concrete.
- The solubility of the sulfates.
- Concentration of the solution.
- The rate at which the sulfate is removed by the reaction with the cement can be replenished: concrete which is exposed to the pressure of sufate-bearing water on one side experiences a higher rate of attack.

The damage usually starts at edges and corners and is followed by progressive cracking and spalling which reduce the concrete to a friable or even soft state.

The vulnerability of concrete to sulfate attack can be reduced by the use of cement low in tricalcium aluminate, such as sulfate-resisting cement etc.

In addition, every effort must be made to produce an impermeable concrete. For concrete structures in sulfate bearing soils, protective coatings such as bitumens, tars and epoxy resins could be applied on exterior surfaces.

Leaching of lime

Leaching of lime compounds, mentioned earlier, may under some circumstances lead to the formation of salt deposits on the surface of the concrete, known as efflorescence. This is found, for instance, when water percolates through poorly compacted concrete or through cracks along badly made joints, and when evaporation takes place at the surface of the concrete. In the process of cement hydration, soluble calcium hydroxide is formed. This material is easily dissolved by water that is lime-free and contains dissolved carbon dioxide. As a result of this action, water will slowly leach out the lime from the concrete and in doing so will weaken the hydraulic bond and etch the surface. At the surface, reaction between $Ca(OH)_2$ with CO_2 will cause precipitation of the white deposit of calcium carbonate [1].

Generally, this type of leaching does not result in any serious problem; the loss of strength is not substantial, but the porosity of concrete is increased and its capacity for holding water is thus greater. The consequence is a possibility of corrosion of the reinforcement.

A homogeneous and dense concrete with a low permeability significantly reduces the effectiveness of the leaching action.

Early efflorescence can be removed with a brush and clean water. But heavy deposits of salt may require acid treatment with dilute hydrochloric acid. Since lime is removed by the acid, the surface of the concrete becomes darker.

Alkali-aggregate reaction

Alkali-aggregate reactivity is a particular mechanism of deterioration in concrete which may occur when alkali solutions present in cement react with certain forms of silica in the aggregate to produce an alkali-silicate gel. The gel, being hygroscopic, absorbs water, and this results in an expansion of the volume that creates tensile stresses within the concrete and causes cracking.

The main external evidence for deterioration of concrete due to alkali-aggregate reactivity is the development of cracking. In unrestrained members this appears as characteristic randomly distributed cracks, but in concrete where the expansive forces are restrained by, for example, reinforcement, the pattern of cracking will be modified and cracks tend to run parallel to reinforcing bars. Other signs of the problem are weeping of the gel from cracks.

Another form of alkali-aggregate reaction is that between some dolomitic limestone aggregates and the alkalis in the cement. The main effect is expansion of concrete similar to that occurring as a result of alkali-silica reaction. One distinction between the two types of alkali-aggregate reactions is that in the alkali-carbonate reaction, the alkali is regenerated.

The main factors influencing the progress of the alkali-aggregate reaction includes the presence of non-evaporable water in the paste, and the permeability of the paste. Moisture is necessary for initiating and supporting the reaction which is accelerated under conditions of alternating wetting and drying.

To obviate the problem it is necessary to reduce water ingress to the concrete so that although gelling will still be present, it cannot imbibe water and exert and expansive force.

- **Effects of freezing and thawing**

Frost damage is a major cause of lack of durability of concrete unless proper precaution is taken.

Action of frost

A brief description of frost action is as follows. As the temperature of saturated concrete in service is lowered, the water held in the capillary pores in the hardened cement paste freezes in a manner similar to the freezing in the pores in rock, and expansion of the concrete takes place. If subsequent thawing is followed by re-freezing, further expansion takes place, so that repeated cycles of freezing and thawing have a cumulative effect. The action takes place mainly in the hardened cement paste; the larger voids in concrete arising from incomplete compaction, are usually air-filled and, therefore, not appreciably subject to the action of frost.

Each cycle of freezing causes a migration of water to locations where it can freeze. These locations include fine cracks which become enlarged by the pressure of the ice and remain enlarged during thawing when they become filled with water. Subsequent freezing repeats the development of pressure and its consequences.

Freezing is a gradual process, partly because of the rate of heat transfer through concrete, partly because of a progressive increase in the concentration of dissolved salts in the still unfrozen pore water (which depresses the freezing point), and partly because the freezing point varies with the size of the pore. Because the surface tension of the bodies of ice in the capillary pores puts them under pressure that is higher the smaller the body, freezing starts in the largest pores and gradually extends to smaller ones. Gel pores are too small to permit the formation of nuclei of ice at temperatures higher than -78^{0}C, so that in practice no ice is formed in them [1]. However, with a fall in temperature, because of the difference in entropy of gel water and ice, the gel water acquires an energy potential enabling it to move into the capillary pores containing ice. The diffusion of gel water which takes place leads to a growth of the ice body and to expansion.

There are two possible sources of dilating pressure. First, freezing of water results in an increase in volume of approximately 9%, so that the excess water in the cavity is expelled. The second dilating force in concrete is caused by diffusion of water leading to the growth of a relatively small number of bodies of ice. This diffusion is caused by osmotic pressure brought about by local increases in solute concentration due to the separation of frozen (pure) water from the pore water.

When dilating pressure in the concrete exceeds its tensile strength, damage occurs. The extent of damage varies from surface scaling to complete disintegration as ice is formed, starting at the exposed surface of concrete and progressing through its depth.

The expansion of the hardened cement paste or of the aggregate, can be determined by cooling the specimen through the freezing range and measuring the change in volume: frost resistant concrete will contract when water is transferred by osmosis from the hardened cement paste to the air bubbles, but vulnerable concrete will dilate, as shown in Figure 2.14.

While the resistance of freezing and thawing depends on its various properties (e.g. strength of the hardened cement paste, extensibility, and creep), the main factors are the degree of saturation and the pore system of the hardened cement paste. Figure 2.15 shows the general effect of the absorption of concrete on its resistance to freezing and thawing.

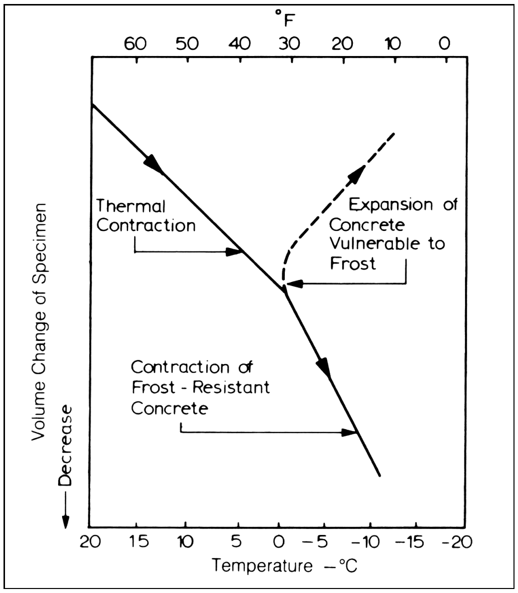


Figure 2.14 Change in volume of frost-resistant and vulnerable concretes in cooling [1]

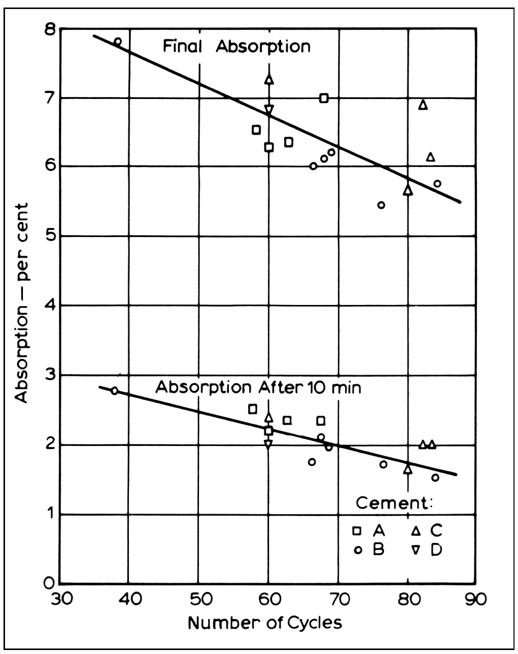


Figure 2.15 Relation between the absorption of concrete and the number of cycles of freezing and thawing required to cause a 2% reduction in the mass of the specimen [1]

- **Physical damages**

Below a brief overview of some of the physical damages is introduced.

Wear

Abrasion

Under many circumstances, concrete surfaces are subjected to wear. This may be due to attrition by sliding, scraping or percussion.

Resistance of concrete to abrasion is difficult to assess because the damaging effects depend on the exact cause of wear. But it has been established that the compression strength of concrete is the paramount factor in determining the abrasion resistance of the concrete surface. Therefore, it is obvious that lightweight concrete is unsuitable when surface wear is expected.

The aggregate properties also have some influence on abrasion resistance, but this is only significant if the aggregate quality is poor or especially good. For improved resistance, however, it is possible to treat the concrete surface with a suitable finish.

Erosion

Erosion of concrete is another type of wear which occurs in concrete in contact with flowing water containing solid particles or grit.

The rate of erosion depends on a number of factors, of which the following are more important [3]:

- The quality of the concrete in terms of its compressive strength and cement content.
- The quality of the aggregate: concrete with large aggregates erodes less than the mortar of equal strength, and hard aggregates improve the erosion resistance.
- The velocity of the flowing water.
- The quantity, shape, size and hardness of the solid particles carried by the flowing water.
- The flow characteristics, that is, whether the flow is continuous or intermittent.

In all cases, it is only the quality of the concrete in the surface zone that is relevant, but even the best concrete will rarely withstand severe erosion over prolonged periods [1].

Cavitation

A high-velocity jet of water striking a concrete surface leads to erosion of the cement paste, resulting in the loosening of the fine and coarse aggregate. Repeated erosion of the concrete creates holes and pits.

Generally, damage by cavitation is found on the surface of aprons and tunnels carrying high-velocity water. Cavitation damage is easily distinguishable from normal erosion by its jagged appearance, in contrast to the smooth worn surface eroded by water-borne particles.

Best resistance to cavitation damage is obtained by the use of high strength concrete, possibly formed by an absorptive lining (which reduces the local water/cement ratio). The maximum size of aggregate near the surface should not exceed 20 mm because cavitation tends to remove large particles. Use of polymers, steel fibres or resilient coatings may improve the cavitation resistance [1; 9].

Good bonding between the aggregate and mortar is essential. In addition, it is necessary to provide smooth and well-aligned surfaces free from irregularities, such as depressions, projection, joints and abrupt changes of profile, to ensure uninterrupted hydraulic flow.

- **Cracking**

Because cracking may impair the durability of concrete allowing ingress of aggressive agents, it is relevant to give a brief overview of the types and causes of cracking.

In addition, cracking may adversely affect the water tightness or sound transmission of structures or their appearance. With respect to appearance, the acceptable crack width depends on the distance from which it is viewed and on the function of the structure.

Cracking occurring in fresh concrete is plastic shrinkage cracking and plastic settlement cracking. Another type of early plastic settlement cracking is known as crazing, which can occur on slabs or walls when the surface zone of concrete has higher water content than that deeper in the interior. The pattern of crazing looks like an irregular network with spacing of up to about 100 mm.

In addition, a somewhat different kind of surface damage, known as blisters, can occur if some bleed water or large air bubbles are trapped just below the surface of the concrete by a thin layer of laitance induced by finishing. Blisters are 10 to 100 mm in diameter and 2 to 10 mm thick [20].

In hardened concrete, cracking may be caused by drying shrinkage or by restrained early-age thermal movement. The various type non-structural cracks are listed in Table 2.8 and schematically in Figure 2.16 [20]. It is useful to note that, whereas one particular cause may initiate a crack, its development can be due to another cause.

We should note that from energy considerations, it is easier to extend an existing crack than to form a new one. This explains why under an applied load, each subsequent crack occurs under a higher load than the preceding one.

The importance of cracking, and the minimum width at which a crack is considered significant, depend on the function of the structural members and on the conditions of exposure of the concrete. Reis [21] suggested the following permissible crack widths, which still offer good guidance:

- Interior members: 0.35mm
- Exterior members under normal exposure conditions: 0.25mm
- Exterior members exposed to particularly aggressive environment: 0.15mm

Various specialized techniques, such as electro-conductive paint and light-dependent resistors-, make it possible to determine the development of cracking. However, very fine cracks are very common but not harmful, so that intensive searching for cracks serves no purpose.

Table 2.8 Classification of intrinsic cracks [20]

Type of cracking	Symbol in fig.2.16	Subdivision	Most common location	Primary cause (excluding restraint)	Secondary causes/ factors	Remedy (assuming basic redesign is impossible); in all cases reduce restraint	Time of appearance
Plastic settlement	A	Over reinforcement	Deep sections	Excess bleeding	Rapid early drying conditions	Reduce bleeding or revibrate	10 min to 3 h
Plastic settlement	B	Arching	Top of columns	Excess bleeding	Rapid early drying conditions	Reduce bleeding or revibrate	10 min to 3 h
Plastic settlement	C	Change of depth	Trough and waffle slabs	Excess bleeding	Rapid early drying conditions	Reduce bleeding or revibrate	10 min to 3 h
Plastic shrinkage	D	Diagonal	Pavements and slabs	Rapid early drying	Low rate of bleeding	Improve early curing	30 min to 6 h
Plastic shrinkage	E	Random	Reinforced concrete slabs	Rapid early drying	Low rate of bleeding	Improve early curing	30 min to 6 h
Plastic shrinkage	F	Over reinforcement	Reinforced concrete slabs	Rapid early drying or steel near surface	Low rate of bleeding	Improve early curing	30 min to 6 h
Early thermal contraction	G	External restraint	Thick walls	Excess heat generation	Rapid cooling	Reduce heat and/or insulate	1 day to 2 or 3 weeks
Early thermal contraction	H	Internal restraint	Thick slabs	Excess temperature gradients	Rapid cooling	Reduce heat and/or insulate	1 day to 2 or 3 weeks
Long-term drying shrinkage	I		Thin slabs and walls	Inefficient joints	Excess shrinkage Inefficient curing	Reduce water content Improve curing	Several weeks or months
Crazing	J	Against formwok	Walls	Impermeable formwork	Rich mixes Poor curing	Improve curing and finishing	1 to 7 days, sometimes much later
Crazing	K	Floated concrete	Slabs	Over-trowelling	Rich mixes Poor curing	Improve curing and finishing	1 to 7 days, sometimes much later
Corrosion of reinforcement	L	Carbonation / Chloride	Columns and beams	Inadequate cover	Poor quality concrete	Eliminate causes listed	More than 2 years
Alkali-aggregate reaction	M		Damp locations	Reactive aggregate plus high-alkali cement		Eliminate causes listed	More than 5 years
Blister	N		Slabs	Trapped bleed water	Use of metal float	Eliminate causes listed	Upon touching
D-cracking	P		Free edges of slabs	Frost-damaged aggregate		Reduce aggregate size	More than 10 years

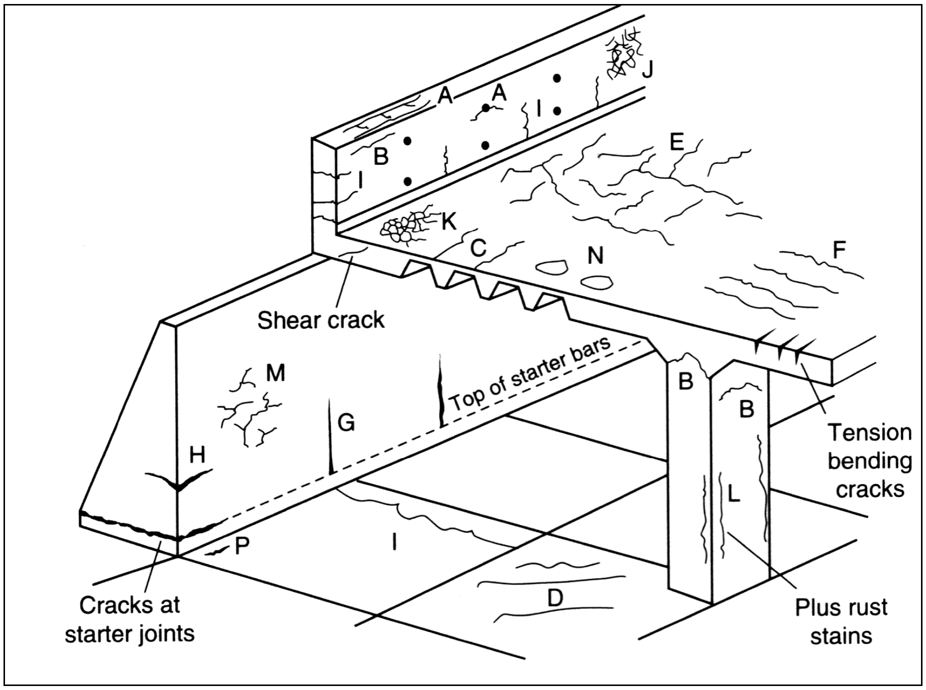


Figure 2.16 Schematic representation of the various types of cracking which can occur in concrete (see Table 2.8) [20]

- **Human factors causing concrete failures**

Faulty design

The effects of unsatisfactory design and/or detailing include poor appearance, inability to continue its function and catastrophic failure. These problems can be minimized only by a thorough understanding of structural behaviour in its broadest sense.

Errors in design and detailing that may result in unacceptable cracking include improper selection and/or detailing of reinforcement, restraint of members subjected to volume changes caused by variations in temperature and moisture, lack of adequate contraction/expansion joints, and improper design of foundations, resulting in differential movement within the structure.

Poor construction practices

A wide variety of poor construction practices can result in cracking in concrete structures.

Malpractices on the site include the following [3]:

- faulty formwork construction leading to grout leakage;
- inadequate vibration, causing honeycombs;
- misplacement of steel which reduces the specified concrete cover;
- improper placing of concrete, such as dropping concrete from a great height to cause segregation;
- lack of curing, causing incomplete hydration of cement;
- poor construction joints.

All these malpractises can lead to significant departure from the specification and hence considerable reduction in the durability of an otherwise well-designed structure.

Poor-quality materials

Even with good constituent materials, concrete mixed too long before use can cause difficulties in placing and compacting. Wrong proportioning of constituent materials, as for example, insufficient cement, high aggregate content or high water/cement ratio, could lead to inferior concrete.

Reinforcement bars with high salt or silt content, lumpy cement and polluted water used for mixing all result in poor quality concrete.

Use of the structure

Changes in the use of structures, such as buildings, warehouses can lead to overloading. With the thrust on improved productivity and mechanization involving the use of heavier equipment and cargo handling machinery, live and wheel loads have increased considerably over the years. These, in turn, contribute to greater impact and dynamic loading which are detrimental to the structures that were not designed and constructed for such purposes.

In particular, excessive loads beyond the elastic limit of concrete may result in creep. For example, the development of micro-cracking and shortening of columns may create a chain-reaction effect that can spread to other structural members causing ultimate collapse.

Other common defects in concrete as a result of overloading include spalling, cracking, flexural and shear fractures, and local disintegration.

2.2 Structural condition assessment

2.2.1 Assessment of historic buildings

Structural condition assessment of historic buildings requires a different level of expertise and experience than that for conventional modern buildings. There are several factors that distinguish historic structures from their modern counterparts. Historic structures were built by design standards and construction methods that were vastly different than those of the modern era. The engineer conducting an evaluation of a historic structure must be fully aware of the parameters under which the structure was designed and constructed originally. The original structural design theory, the intended building use, and construction practices are all important conditions that need to be studied and understood in order to address the current condition of a historic structure appropriately. Unlike in modern buildings, structural material properties often are unknown in historic structures, and drawings defining the structural system may be limited or may not exist at all. The fact that a building is designated as ´´historic´´ also means that it is statistically more prone to have developed long-term structural problems than younger buildings, and greater care must be taken to investigate possible deterioration [24]. Lastly, engineer must be more sensitive to the architectural fabric of historic structures as compared with modern buildings when evaluating them because preservation of the original materials and finishes is paramount.

Some countries generally take 50 years as the minimum age for designating a structure as historic [24].

2.2.2 Process of assessment

Table 2.1 shows common causes of defects according to the European Standard of EN 1504-9 [22]. With respect to later planning of repairs, generally defects in concrete and defects caused by reinforcement corrosion should be distinguished. The purpose of main assessments is [11]:

- to identify the cause or causes of defects;
- to establish the extent of defects;
- to establish where the defects can be expected to spread to parts of the structure, that are at present unaffected;
- to assess the effect of defects on structural safety;
- to identify all locations, where protection or repair may be needed.

More details on the requirements for assessment are given in EN 1504-9 [22].

2.2.3 Inspection and diagnosis of concrete defects

There is a need for a regular system of inspection (see Figure 2.17) of all reinfoced concrete structures so that any deterioration can be detected and recorded at its early stages, and a decision then taken on what remedial works, if any, should be carried out. However, it must be noted that the object of periodic inspections is to arrest deterioration at its early stages and it in no way implies that such structures are especially vulnerable to structural failure [3].

- **The survey**

The objectives of a structural survey are obviously dictated by the requirements. Whether the survey is to satisfy a mandatory requirement or to assess the strength of the structural members after a fire, the underlying requirement for the following is to be identified.

What is the present state of deterioration and condition of the member?

What will be the future rate of deterioration?

Are repairs required urgently?

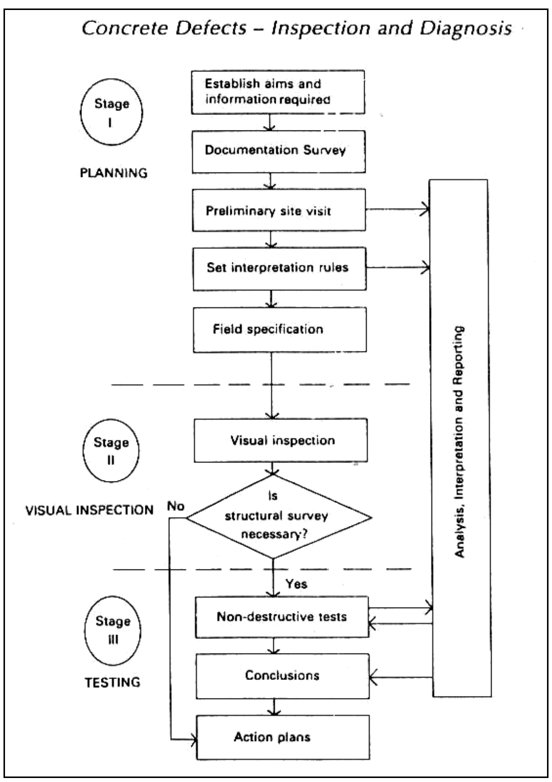


Figure 2.17 Flowchart to illustrate the inspection process [3]

- **Selection of test methods**

Test selection is based on a combination of factors such as:

- The availability and reliability of calibrations;
- The effects and acceptability of surface damage;
- Practical limitations such as member size and type, surface condition, depth of test zone required, location of reinforcement and accessibility;
- The degree of accuracy required;
- Economic consideration of the value of work under investigation and the cost of delays in relation to the cost of the test program.

Generally, the complexity of calibration tends to be greatest for those tests which cause the least damage. For example, while surface hardness and pulse velocity tests cause no damage, are cheap and quick, and are ideal for comparative and uniformity assessments, their calibration for absolute strength estimates poses many problems. Core tests, on the other hand, provide the most reliable information, but also cause the most damage and are slow and expensive.

When a comparison with concretes of similar quality is all that is necessary, the choice of the test method is governed primarily by practical limitations. For example, the surface hardness test may be used for new concrete, while ultrasonic is selected where two opposite faces of the member are accessible.

The test program is also influenced by the costs of the tests in relation to the value of the project involved, the costs of delays to construction and of possible remedial works.

An overview of the tests related to reinforced concrete aging studies is available in literature [3; 9; 26].

- **Interpretation of results**

It is essential that there is agreement on the way that results should be interpreted in order to avoid disputes. Interpretation should take into account the capabilities and accuracies possible from the tests, as well as environmental effects and practical difficulties.

Examples of environmental factors include differential rates of weathering and chemical attack between parts of existing structures, as well as the influence of moisture conditions. The last factor is particularly important for permeability, integrity and strength tests, because the calibrations prepared under laboratory conditions may not be the same as those for site conditions.

Assessment of moisture conditions internally within concrete is often beset with difficulties, such as variations of in-place concrete strength and mix proportions, and the influence of reinforcing steel on test results. These factors will affect the locations and the number of individual tests to be carried out, which is very often a compromise between accuracy and cost.

Having established and agreed on the procedures to be adopted, interpretation should be on-going throughout testing by an experienced engineer. In this way, the program may be modified as necessary.

- **Recommendations for action**

If a visual inspection has not shown any signs of distress or deterioration, the structure may be assumed to be safe, and non-destructive testing is unnecessary. At best, it is only necessary to check all the structural calculations and details, and construction records. If the check reveals deficiencies in design or construction, doubts arise as to why there are no symptoms. A thorough structural check may be required in this case.

If there are signs of distress, deterioration or structural malfunction, the engineer has to make the crucial recommendation as to whether the building should be demolished, or its structure repaired and strengthened.

If there are signs of severe distress in a structure, it may be better to evacuate the building, particularly if monitoring indicates that cracks are widening and the deformations are increasing.

2.3 Concrete repair

- **Phases of repair projects**

The phases of repair projects follow a logical sequence, which is dominated by engineering aspects. Table 2.3.1 gives a general scheme according to the European Standard EN 1504-9 [22]. This figure shows the wellknown elements, such as assessment, planning, design and quality control.

Table 2.3.1 The phases of repair projects [22]

Management of the Structure	Process of Assessment	General Planning	Design of Repair Work	Repair Work	Acceptance of Repair Work
Basic considerations and actions					
Conditions and history of structure Documentation Commissions for maintenance	Defects and their classification and causes	Options Principles Methods	Definitions of the intended use of products Requirements - substrate - products - work - specifications - drawings	Choice and use of products and equipment Tests of quality control Health and safety	Acceptance testing Remedial works Documentation
Parts of the EN 1504 series and clauses of EN 1504-9					
Clause 8 of EN 1504-9	4.3 of EN 1504-9	Clauses 5 and 6 of EN 1504-9	EN 1504-2 to EN 1504-7 and clause 7 and annex A of EN 1504-9	Clause 9 of EN 1504-9 and ENV 1504-10	

- **Options, principles, and methods for protection and repair**

The rules for the use of products and systems for protection and repair of concrete structures are based on a hierarchy of different levels, namely options, principles and methods.

According to EN 1504-9 [22], the following options shall be taken into account in deciding the appropriate action to meet the future requirements for the life of the structures:

a) do nothing for a certain time;
b) re-analysis of structural capacity, possibly leading to downgrading of the function of the concrete structure;
c) prevention or reduction of further deterioration,without improvement of the concrete structure;
d) improvement, strengthening or refurbishment of all or parts of the concrete structure;
e) reconstruction of part or all of the concrete structure;
f) demolition of all or part of the concrete structure.

For protection and repair, different principles have been defined, separately for repair and protection of damages to the concrete and damages induced by reinforcement corrosion.

Tables 2.3.2 and 2.3.3 show the six principles for protection and repair of concrete and the five principles to prevent damages due to reinforcement corrosion, respectively. To protect or repair a concrete structure, according to the principles, different methods are available. These principles and methods are based on the European Standard EN 1504-9 [22]. The system of options, principles and methods is the basis for product selection by the designer.

Table 2.3.2 Principles for repair and protection for damages of concrete

Principle no.	**Principle and its definition**
Principle 1 [PI]	Protection against ingress
Principle 2 [MC]	Moisture control
Principle 3 [CR]	Concrete restoration
Principle 4 [SS]	Structural strengthening
Principle 5 [PR]	Physical resistance
Principle 6 [RC]	Resistance to chemicals

Table 2.3.3 Principles for protection against reinforcement corrosion

Principle No.	**Principle and its definition**
Principle 7 [RP]	Preserving or restoring passivity
Principle 8 [IR]	Increasing resistivity
Principle 9 [CC]	Cathodic control
Principle 10 [CP]	Cathodic protection
Principle 11 [CA]	Controlling of anodic areas

REHABCON provides a manual for users to choose a strategy for the rehabilitation of concrete structures (Figure 2.3.1) [23].

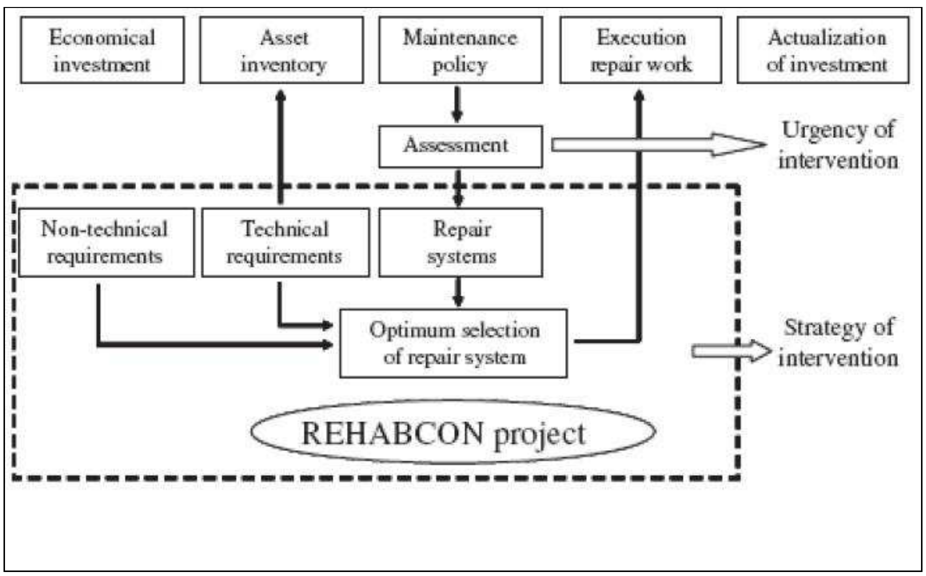


Figure 2.3.1 Scheme of general procedure described in the manual

There are various methods and materials available for repair or rehabilitation of concrete structures; they may not be used directly in project specifications because each repair project may require unique remedial action.
Concrete material and methods for repair and rehabilitation are described more precisely in literature [5; 6; 7; 9; 22; 23; 40-49].

- **Planning and selection of products**

Table 2.3.4 shows the systematics of planning according to EN 1504-9 [22]. As already shown in Table 2.3.1, planning starts with the assessment of the status of the structure. The following is to be selected: options (repair strategy), repair principles and repair method, as defined in the previous section. Based on this selection scheme, the repair materials can be chosen. EN 1504-9 defines performance characteristics for every repair method, separately for all intended uses and for certain intended uses [11].

Table 2.3.4 Systematics of planning according to EN 1504-9 [22]

Assessment of the status of the structure (defects ...)
Elaboration of the repair strategy (do nothing, repair, ..., demolition)
Definition of the repair principles (e.g. protection against ingress ...)
Selection of the repair method (e. g. impregnation ...)
Definition of the repair materials (strength, modulus, adhesion, etc.)
Definition of inspection and maintance requirements

The conditions under which the repair material will be placed and the anticipated service or exposure conditions can have a major impact on the design of a repair and selection of the repair material. The following factors should be considered in planning a repair strategy [11]: 1) application conditions (geometry, temperature, moisture, location), 2) service conditions (temperature, chemical attack, chemical attack, appearance, service life).

Most repair projects will have unique conditions and special requirements that must be thoroughly examined before the final repair material criteria can be established. Once the criteria for a dimensionally compatible repair have been established, materials with the properties necessary to meet these criteria should be identified. A variety of repair materials have been formulated to provide a wide range of properties. Since these properties will affect the performance of a repair, selecting the correct material for a specific application requires careful study.

Properties of the materials under consideration for a given repair may be obtained from manufacturer's data sheets, evaluation reports, contact with suppliers, or by conducting tests [11].

- **Factors affecting the durability of concrete repair systems**

To achieve durable repairs it is necessary to consider the factors affecting the design and selection of repair systems as parts of a composite system. Selection of a repair material is one of the many interrelated steps; equally important are surface preparation, the method of application, construction practices, and inspection. The critical factors that largely govern the durability of concrete repairs in practice are shown in Figure 2.3.2. These factors must be considered in the design process so that a repair material compatible with the existing concrete substrate can be selected. Compatibility is defined as the balance of physical, chemical, and electrochemical properties and dimensions between the repair material and the concrete substrate. This balance is necessary if the repair system is to withstand all anticipated stresses induced by volume changes and chemical and electrochemical effects without distress or deterioration in a specified environment over a designated period of time. For detailed discussions of compatibility issues and the need for a rational approach to durable concrete repairs, see Emmons, Vaysburd, and McDonald (1993 and 1994) [9]. Dimensional compatibility is one of the most critical components of concrete repair. Restrained contraction of repair materials, the restraint being provided through bond to the existing concrete substrate, significantly increases the complexity of repair projects as compared to new construction. Cracking and debonding of the repair material are often the result of restrained contractions caused by volume changes. Therefore, the specified repair material must be dimensionally compatible with the existing concrete substrate to minimize the potential for failure. Those material properties that influence dimensional compatibility include drying shrinkage, thermal expansion, modulus of elasticity, and creep.

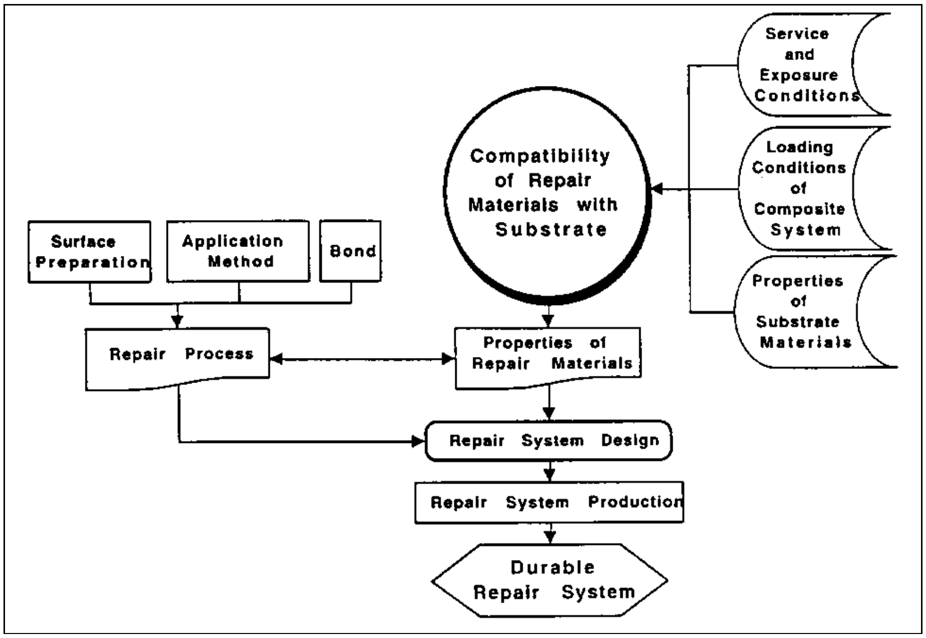


Figure 2.3.2 Factors affecting the durability of concrete repair systems according to Emmons and Vaysburd 1995 [9]

- **Properties of repair materials**

In addition to conventional Portland-cement concrete and mortar, there are hundreds of proprietary repair materials on the market, and new materials are continually being introduced. This wide variety of both specialty and conventional repair materials provides a greater opportunity to match material properties with specific project requirements; however, it can also increase the chances of selecting an inappropriate material. No matter how carefully a repair is made, use of the wrong material will likely lead to early repair failure. These properties should be considered before any material is selected for use on a repair or rehabilitation project as follows: compressive strength, modulus of elasticity, coefficient of thermal expansion, adhesion/bond, drying shrinkage, creep, and permeability.

- **Repair of cracking**

The wide variety of types of cracking described in Section 2.1 suggests that there is no single repair method that will work in all instances. A repair method that is appropriate in one instance may be ineffective or even detrimental in another. For example, if a cracked section requires tensile reinforcement or posttensioning to be able to carry imposed loads, routing and sealing the cracks with a sealer would be ineffective. On the other hand, if a concrete section has cracked because of incorrect spacing of contraction joints, filling the cracks with a high-strength material, such as epoxy will only cause new cracking to occur as the concrete goes through its next contraction cycle. Potential repair materials and methods may be selected with the procedures shown in Figures 2.3.3 and 2.3.4.

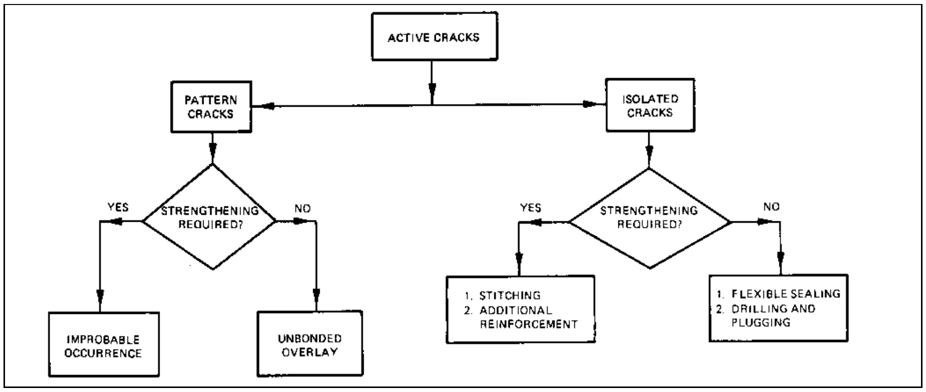


Figure 2.3.3 Selection of repair method for active cracks after Johnson [9]

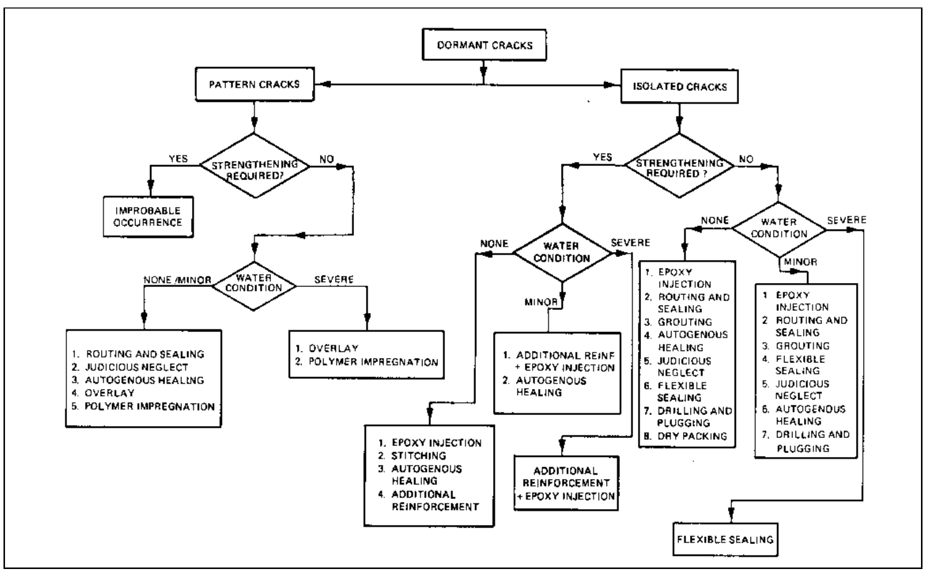


Figure 2.3.4 Selection of repair method for dormant cracks after Johnson [9]

- **Repair of spalling and disintegration**

Spalling and disintegration are only symptoms of many types of concrete distress. There is no single repair method that will always apply. For example, placing an air-entrained concrete over the entire surface of concrete that is deteriorating because of freezing and thawing may be a sound repair method. Use of the same technique on concrete deteriorating from strong acid attack may not be effective.

General repair approach can be selected from Table 2.3.6, which presents a comparison of the possible causes of spalling and disintegration symptoms and the general repair approaches that may be appropriate for each case. Table 2.3.7 relates the repair approaches shown in Table 2.3.6.

Table 2.3.6 Causes and repair approaches for spalling and disintegration [9]

	Cause	Deterioration likely to continue		Repair approach
		Yes	No	
1	Erosion (abrasion, cavitation)	X		Partial replacement Surface coatings
2	Accidental loading (impact, earthquake)		X	Partial replacement
3	Chemical reactions			
	Internal	X		No action Total replacement
	External	X	X	Partial replacement Surface coatings
4	Construction errors (compaction, curing, finishing)	X		Partial replacement Surface coatings No action
5	Corrosion	X		Partial replacement
6	Design errors	X	X	Partial or total replacement based on future activity
7	Temperature changes (excessive expansion caused by elevated temperature and inadequate expansion joints)	X		Redesign to include adequate joints and partial replacement
8	Freezing and thawing	X		Partial replacement No action
	Note: This table is intended to serve as a general guide only. It should be recognized that there are probably exceptions to all of the items listed.			

Table 2.3.7 Repair methods for spalling and disintegration [9]

	Repair approach	Repair method
1	No action	Judicious neglect
2	Partial replacement (replacement of only damaged concrete)	Conventional concrete placement Drypacking Jacketing Preplaced-aggregate concrete Polymer impregnation Overlay Shotcrete High-strength concrete
3	Surface coating	Coatings Overlays
4	Total replacement of structure	Remove and replace

- **Concrete removal and preparation for repair.**

Most repair projects involve removal of distressed or deteriorated concrete. Regardless of the cost or complexity of the repair method or of the material selected, the care with which deteriorated concrete is removed and with which a concrete surface is prepared will often determine whether a repair project will be successful.

Concrete removal and preparation procedures as well material and methods for repair and rehabilitation are described more precisely in literature [5; 6; 7; 9; 22; 23; 40 - 49].

2.4 Conclusion

To summarize coverage in literature, it should be underlined that in spite of quite deep analysis of differente types of damages, no adequate investigations reporting on old, 90-years of age or more reinforced concrete structures were found. One of the aims of this thesis is to study the latter.

3 METHODS USED IN THE THESIS

The methods presented below are based on the methods used in the investigation, analyses and restoration in the case studies of Tallinn and Papisaare hydroplane hangars.

3.1 Methods for structural condition assessments

The following methods were used in the structural assessments of hydroplane hangars:

- The history, location, the composition of the materials and building technology used were introduced.
- Elements and geometry of carrying structures were described.
- Visual investigation of damaged structures was done.
- Materials studies were made according to the methods chosen and described in Section 5.4.
- Summary and conclusions concerning structural studies were made.

3.2 Methods for assessment of residual carrying capacity analysis

The assessment and analysis of residual carrying capacity of hydroplane hangars were made according to the following methods:

- The performance of the structures was analyzed by the finite element method using contemporary calculation programs.
- Strength calculations were done according to the norms and standards in force.
- Summary and conclusions of assessment of residual carrying capacity studies were made.

3.3 Methods for restoration

Restoration of reinforced concrete structures of hydoplane hangars was accomplished by the following methods:

- On the basis of structural assessments and studies and analyses of residual carrying capacity of structures and elements requiring fast restoration were selected.
- Proper structural solutions for the restoration of the selected objects were chosen.
- Structural solutions and their applicability on the objects were proposed.
- Summary and conclusions of selection of restoration solutions and their applicability on the objects were made.

4 HISTORY OF STUDIED OBJECTS

The object of study of this thesis is the hydroplane hangars in Tallinn and Papisaare (Saaremaa). Below a brief history of these objects is introduced.

- **Brief history of Tallinn hydroplane hangars**

Between the years 1916-17 a hangar for hydroplanes was designed and erected in Tallinn, Estonia (Figures 4.1 – 4.4). This large span structure, courageous and advanced in its time, consists of three spherical and seven cylindrical reinforced concrete roof shells and four double-decker towers. It is the first implemented early 20^{th} century concrete structure of this type in the whole world. No drawings or calculations have been preserved.

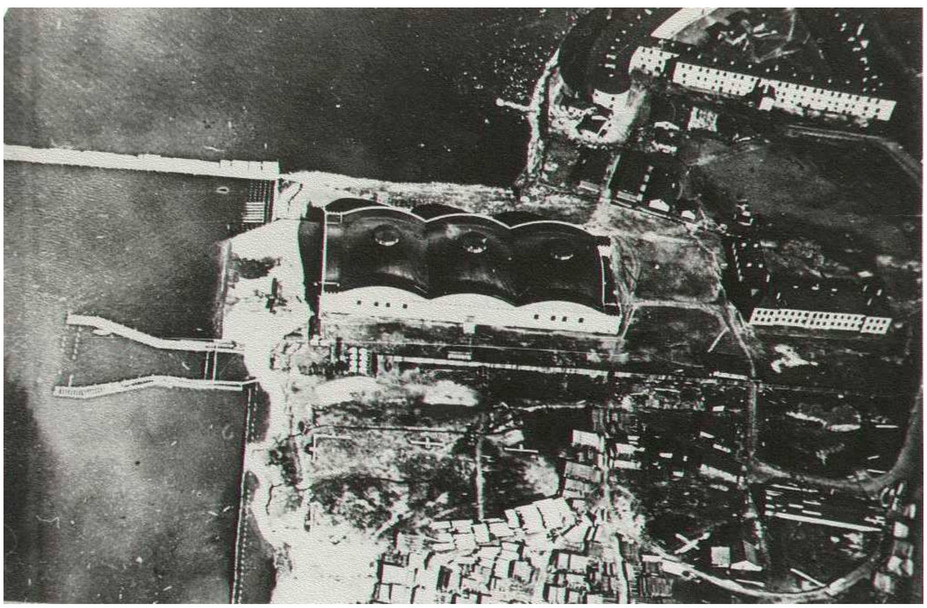


Figure 4.1 View from the west in the 1930-s

Only a few essential historic facts are described in this thesis. A detailed historic account can be found in literature [32].

In 1907 a defence plan was developed in Russia, authorized by the Czar Nicholas II in 1911 (strategic principles of the Tallinn-Porkkala line were vital for Estonia), according to which a naval operative base had to be built by 1916. Originally, the Czarist defence plan did not include air force bases, but due to the fast development of air force during World War I, the defence plan was amended. In accordance with the new plan, the new defence bases included placements for airplanes.

The hydroplane hangars in Tallinn and Papisaare are thus one element in a holistic defence /fortification system for the Baltic Sea area created by Czarist Russia in the first decades of the 1900s - just in advance of the Bolshevist coup d'etat, in the shadow of which the Baltic countries succeeded in restoring/rebringing themselves among the independent European states.

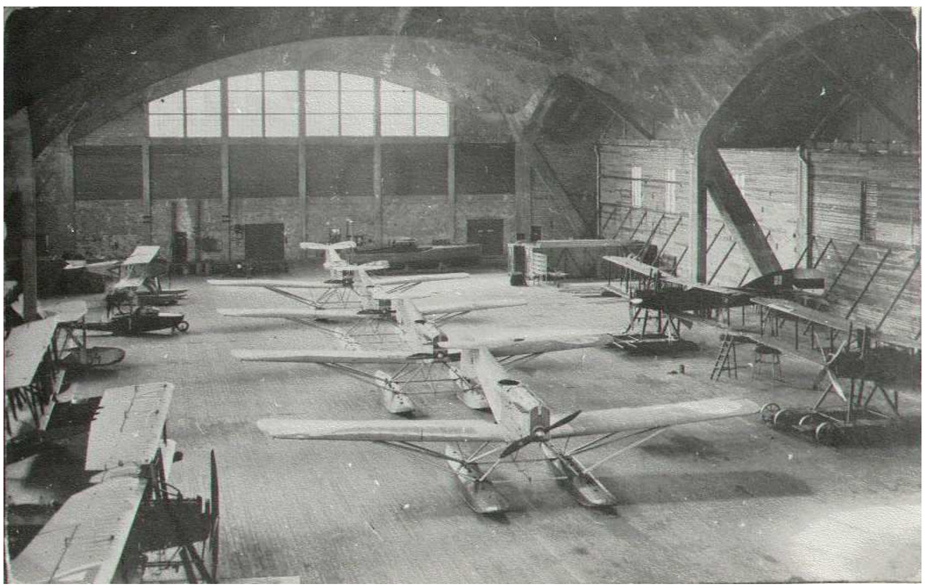


Figure 4.2 Hydroplanes inside the hangars in the 1920-s

The preparation work for this naval base construction for seaplanes started in 1913 by research and financial calculation. March 1916 can be considered a starting shot for design: then 11 construction companies were invited to participate in a design competition. For design purposes, a technical specification was provided. The winner was the Danish company Christian& Nielsen (designer Sven Schultz). The same company also acquired the building contract.

The project consisted of two groups of hangars – the first had three and the second one two hangars. Construction started in July 1916 and lasted up to 13 October 1917. Due to the October Revolution in Russia, the new Russian government ordered Christian & Nielsen to terminate the work and finished the contract. By that time the construction of the first group of hangars was practically completed. In the second group of hangars basement had been constructed.

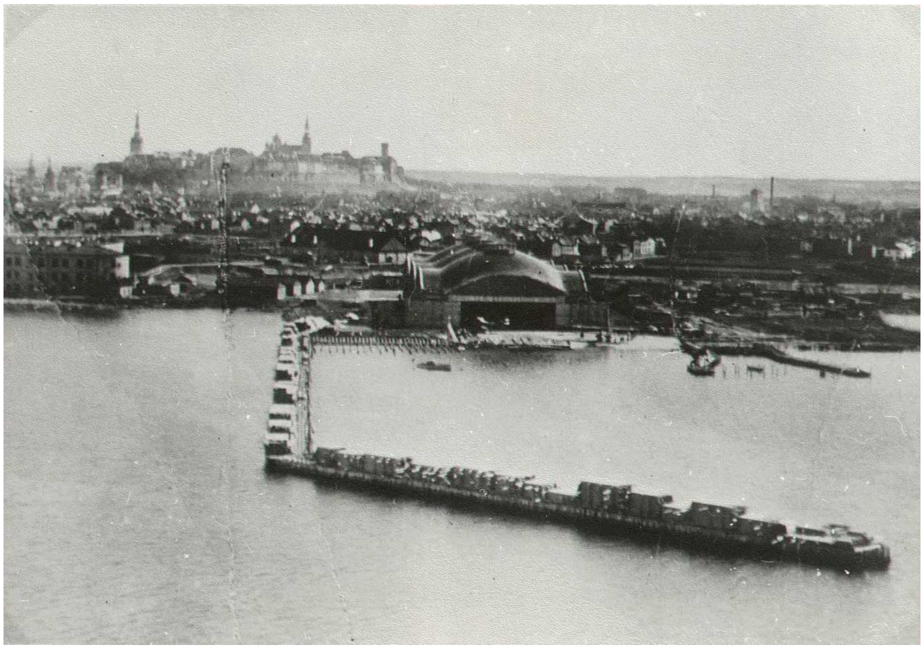


Figure 4.3 Hydroplane hangars in Tallinn, view from the north in the 1930s

During 85 years no substantial repairs were performed which would prolong the lifetime of the reinforced concrete roof shells. Originally, the upper surface of the hangar shells was covered with a kind of tar paper, it is, however, not clear which material exactly was used. Mentions can only be found from the year 1938 when the old isolation was scrabbled from the broken places and the concrete cracks were filled with tar. After that the shell was covered with bitumen previously heated to 120^0C. Before lubrication the shell was covered with yellow unbleached cloth. Sand was poured into hot bitumen. The measures concerned only one shell which faces the city. The two remaining shells facing the sea were tarred. Later, in the 1960s all the reinforced concrete roof shells were tarred.

Figure 4.4 View from the west

The building has attracted attention of several international organizations, such as the International Association for Shell and Spatial Structure (IASS), International Federation of Concrete Fib (CEP-FIP), and the International Committee for the Conservation of the Industrial Heritage (TICCIH). The building is under the protection of antiquities as Estonian National Heritage object No 452 and is well known at least in the Nordic Countries.

In 2001, an investigation was ordered by Tallinn Cultural Heritage Department from Tallinn University of Technology (TUT) to analyze the actual technical condition of the structure, its stress and strain state and to design a restoration project. On the basis of this project, inevitable repairs were performed on the middle roof that was in the worst condition. However further work no financing sources were available.

- **Brief history of the Papisaare hydroplane hangars in Saaremaa**

According to the historic studies [32] Papisaare (Saaremaa), hydroplane hangars (Figure 4.5) belonged to the Tallinn Porkala defence line described above that composed a so-called pre-station.

The researchers [32] were not able to collect detailed data concerning the history of the structure, designers and builders of the hangars, properties of the materials used, etc.

In spite of that it can be assumed that the hangars were designed and built according to the requirements existent at that time (see p. 5.4.1.1)), in view of the properties of materials and structural conditions. That was also confirmed by the materials studies conducted with reinforced steel within this research.

Figure 4.5 Papisaare hydroplane hangars, view from the north

5 CASE STUDIES OF STRUCTURAL CONDITION ASSESSMENT

5.1 Introduction

This research focused on the structural conditions of the Tallinn and Papisaare hydroplane hangars.

The geometry of structures and principles of behaviour in terms of carrying capacity were specified and visual and materials research was conducted for structural condition assessments. Materials research included the following: properties and chemical composition of reinforced steel; compressive strength, frost resistance, water absorption of concrete; X-ray analysis of cement composition; analysis of concerete structure by electron microscopy, and assessment of the extent of carbonation.

The methods used in materials research are described in the relevant subsections.

The results of the studies completed are described in the conclusions in the final part of this chapter.

5.2 Geometry and design

- **Tallinn hydroplane hangars**

The hydroplane hangar consists of three 36.4 x 36.4 m in plan reinforced concrete spherical shells and seven short cylindrical shells 36.4 x 6.8 m connected with them. In every corner there are two storey towers 6.8 x 6.8 m in plan (see Figure 5.1). At the top of each spherical shell - there is a 12-sided lightning window with a diameter of 10 m. The internal bearing distance is 116.0 x 36.4 m, the height of the doors is 10.0 m.

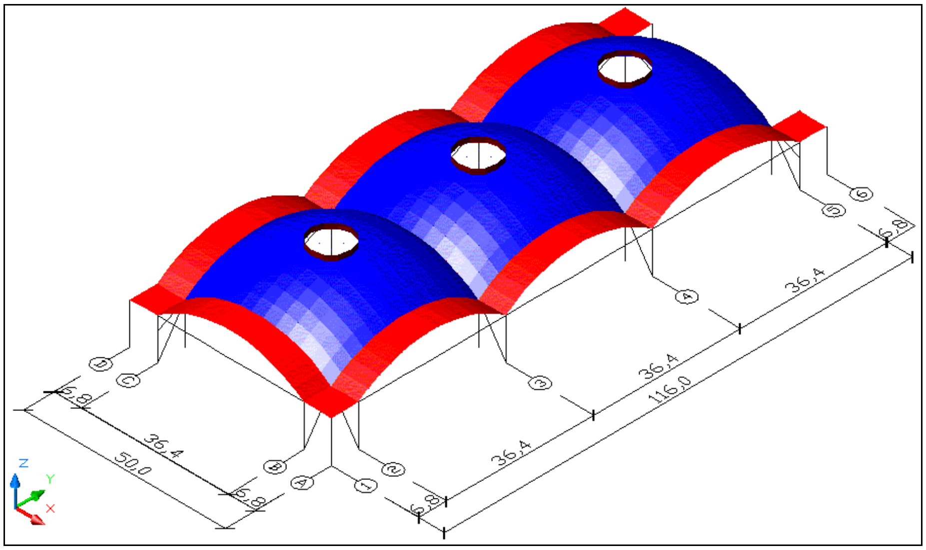


Figure 5.1 Dimensions of the hangar

The whole structure is carried by 24 columns and 12 angle braces (Figure 5.2). The columns are supported on the piled foundation. According to the original project, the piles are connected to a horizontal reinforcement concrete tie beam to receive the horizontal support reaction. More precise data about the quantity, parameters and foundation plate thickness of the reinforcement concrete pile are missing. Cross-section of the column and angle braces is between 0.60 – 1.30 m.

Lower beams of all cylindrical shells are equipped with two angle braces to receive the horizontal side wind load. Through those angle braces the wind load will be transferred to the edge of the spherical shell (Figure 5.3). Cross-section of the tension beams is 0.4 x 0.6 m. From the west and north side, the cylindrical shells are restricted with fulfilled reinforced concrete girder, east and south side the area between reinforced concrete girder braces are covered with a wood slat. The southern part of the building houses a single-storied workshop.

The shell structure has edge elements of a cross-section 0.60 x 0.75 m (Figure 5.4). The diameter of reinforcing bars of columns, angle braces and edge elements is ~ 25 mm.

Figure 5.2 Reinforced concrete columns and angle braces

Figure 5.3 Reinforced concrete angle braces used to take wind loads

Figure 5.4 Edge element of the shells and girder covered by timber slatings

The corner towers and wall columns are connected with a horizontal reinforcement concrete beam.

Hangar walls are constructed of concrete small blocks. Certain parts of load resistance are taken by those walls and columns. To increase wind load resistance, the reinforcement concrete columns and wall blocks are connected with metal braces. Thickness of external concrete walls is 0.3 m, there are columns with a spacing of 6.0 m and a cross-section of 0.30 x 0.40 m between the walls (Figure 5.5).

Figure 5.5 Concrete block wall, view from the east

From the engineering point of view, the most interesting part of the construction is the reinforced concrete roof shell of the hangars. The height of the shell edge from the floor is 10 m. The maximal height of the spherical shells under the lightening windows is 22 m (from the floor) and that of cylindrical shells to the top is 16 m. The maximum rise of spherical shells is 12 m and cylindrical ones 6 m. Spherical and cylindrical shells have the curvature radius of 30.6 m. The thickness of shells is 80…150 mm (in corner areas 400…720 mm).

The ratio between thickness t (~110 mm) and bearing span

$$\frac{t}{L} = \frac{110}{36400} = \frac{1}{331},$$

would be rather small in today's terms.

The shells described are thin as can be seen from the ratio

$$\frac{t}{Rmin} < \frac{1}{20},$$

where R_{min} is the curvature radius.

- **Papisaare hydroplane hangars**

The hydroplane hangar consists of three 22.0 x 21.8 m (see Figure 5.6) in plan reinforced concrete frames (24 columns and 18 beams). Information about the foundation the quantity, parameters and foundation plate thickness of the reinforcement concrete pile are missing.

The internal bearing distance is 22.0 x 21.8 m, the height of the doors is 5.15 m.

The whole structure is carried by 24 columns and 18 beams. The columns are supported on the piled foundation. Cross-section of the columns is 0.30 – 0.45 m and that of the beams is between 0.30 – 1.40 m. To resist deformations caused by temperature variations there are deformation joints in the middle opening of the hangar (Figure 5.7).

The second hangar consists of two 22.0 x 21.8 m concrete frames.

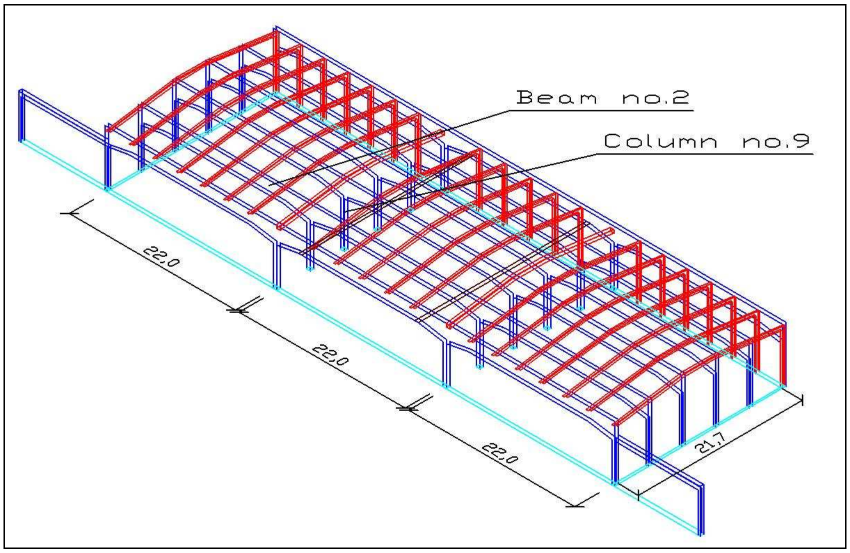


Figure 5.6 Dimensions of the hangar

Figure 5.7 Deformation joint

5.3 A visual inspection of the deteriorated reinforced concrete construction

A visual inspection of the exposed concrete is the first step in an on-site examination of a structure. The purpose of such an examination is to locate and define areas of distress or deterioration.

- **Tallinn hydroplane hangars**

Because of very poor maintenance during the last decades and due to the absence of roofing, windows and doors, water and moisture has penetrated in the concrete surface, causing moisture and frost damages of concrete and corrosion of reinforcement and washing out of binder.

In some places corrosion of reinforcement is so extensive that only prints of the corrosion are left. On the lower surface of the shells nearby 50 % of the cover layer is absent and some pieces of concrete cover are continuously falling down.

Due to the missing cover layer (Figure 5.8) and decreasing cross section of reinforcement, the reinforcement does not serve its purpose in receiving the internal forces. The condition of the upper surface of the shells is better, due to the thicker cover layer of concrete.

Extensive damages are seen everywhere (columns, edge elements, diagonals; figure 5.9). Restoration and reinforcement is still necessary.

Central spherical shell has an area of (~ 6 m^2) where part of the shell was broken, settled downward ~ 70 mm (Figure 5.10, area A).

The crack between the spherical and cylindrical shells was formed both in the north side and middle shells (Figure 5.10, area B). The present dangerous situation is reduced on the middle shells.

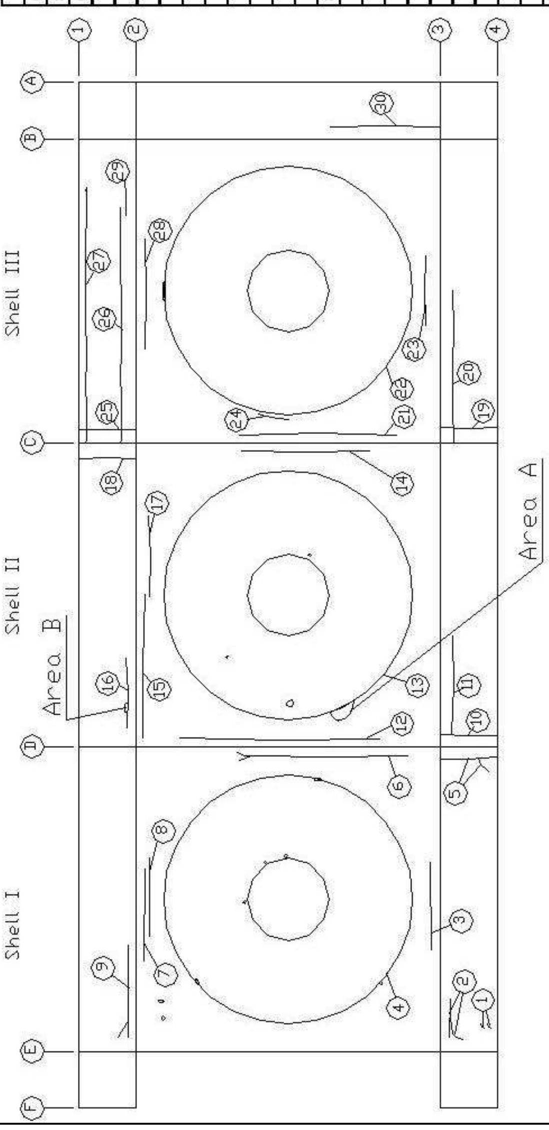
Figure 5.8 Shell coverage

Figure 5.9 Damages of the lower surface

Many cracks have emerged on the top surface of shells. The total length of the cracks is about 600 m (see Figure 5.10) and most of them pervade the shells. The cracks have been caused by temperature and humidity changes; possibly also by the shrinking concrete volume (the 100 m long construction has no temperature joints).

All three shells have rounded cracks, caused by work-joints originating from the erection.

Now the upper surface of the shell is covered only partly with oxidized bitumen. Most upper surfaces of the shell have been washed with rain and are cracked. In autumn 2001, the central part of the spherical shell and two cylindrical shells were covered with SBS type roofing.



Specification of the cracks			
Cracks width > 1,5 mm			
Shell I		Shell III	
Crack no	length, m	Crack no	length, m
1	2,6	19	6,8
2	8,1	20	14,8
3	10,5	21	18,4
4	93,5	22	93,5
5	8,3	23	8,7
6	22,1	24	3,6
7	10,9	25	6,8
8	9,4	26	29,0
9	13,2	27	31,0
Total:	178,6	28	13,2
Shell II		29	3,0
10	6,8	30	13,5
11	12,7	Total:	242,3
12	24,7		
13	93,5		
14	15,0		
15	18,5		
16	9,0		
17	9,5		
18	6,8		
Total:	196,5		

Figure 5.10 Cracks and deformed areas in the reinforced concrete roof

- **Papisaare hydroplane hangars**

Damages are seen everywhere (columns, beams; Figure 5.11). Due to the missing cover layer and decreasing cross section of reinforcement, the reinforcement does not serve its purpose in receiving the internal forces. Restoration and reinforcement is still necessary.

The reason of the damages is poor maintenance during the last decades and due to the absence of roofing, widows and doors, water and moisture has penetrated in the concrete surface, causing moisture and frost damages of concrete and corrosion of reinforcement and washing out of binder.

Figure 5.11 Damages of the hangar frame

5.4 Materials research concerning reinforced concrete

In order to analyze the condition of concrete and reinforcement, material~~s~~ research was performed in the Center for Materials Research, Laboratory of Mechanical Testing and Metrology, Testing Laboratory of Building Materials of TUT and ETUI BetonTEST OÜ. During the research the cause, extent and nature of corrosion were determined. According to the results of the analysis, a restoration solution was developed.

5.4.1 Materials used according to the initial project

1) Tallinn hydroplane hangars

The hangars had to be built according to the design norms valid in Czarist Russia. On the bases of those norms materials were chosen and calculations were made: concrete 1:2:3 with allowed 28-day concrete compressive strength at least 150 kg/cm^2; the allowed strength for walls - 30 kg/cm^2; tension strength of reinforcement steel bars - 1000 kg/cm^2 and shear strength - 800 kg/cm^2; volume weight of reinforcement concrete - 2400 kg/m^3, volume weight of concrete - 2200 kg/m^3; and design wind load - 200 kg/m^2.

2) Saaremaa Papisaare hydroplane hangars

Construction works of Saaremaa Papisaare hydroplane hangars were ordered by the naval construction division of Czarist Russia. Therefore, in spite of absence of archival data, it is assumed that the requirements set for materials of the erected reinforced concrete hangars were in compliance to the conditions set to the builder of Tallinn hydroplane hangars (see section 5.4.1 1)).

5.4.2 Compressive strength of concrete

- **Compressive strength of concrete of the Tallinn hydroplane hangar construction**

The compressive strength of concrete was determined by two different methods: the indirect ("no smashing" method) and the direct method.

a) Indirect method

The compressive strength of concrete by the indirect method was determined by the SCHMIDT´I strength impact hammer type N according to the standard EN 12398. According to the methods eight tests were carried out in each measuring point and the mean measuring result was calculated on the basis of six tests (the highest and the lowest value were subtracted). On the basis of the graph (by interpolating between vertical and horizontal readings), on the mean measuring result, the mean compressive strength of $f_{ck,cube}$ of the specimen of 200 x 200 x 200 mm was determined. The shape factor in the transfer to the sample dimensions 150 x 150 x 150 mm is α= 1.05 and the time correction factor is α_t= 0.7.

Results

Table 5.1 presents summary results of the determination of compressive strength of concrete.

Table 5.1 Determined compressive strengths of concrete

Pos.	Element/parameter	Spherical shells	Cilindrical shells	Edge elements	Area A	Area B
1	Number of measuring points	16	21	21	1	1
2	Average compressive strength $f_{ck, cube}$, N/mm^2 Sample 200 x 200 x 200 mm	12.2	14.4	21.9	35.0	33.2
3	Mean compressive strength $f_{ck, cube}$, N/mm^2 Sample 150 x 150 x 150 mm	12.8	15.1	23.0	36.7	34.9
4	Class according to EN 206-1	C 8/10	C 12/15	C 16/20	C25/30	C25/30

Area A is additionally reinforced in the middle spherical shell (see Figure 5.10).

Area B is additionally reinforced in the cylindrical shell (see Figure 5.10).

b) Direct method

The compressive strength of concrete was determined by the CORE---CASE device of the Danish German Instruments Ltd. This device is used to drill out concrete samples – centre punches of a cross-section of 50 mm and depth of 50-90 mm from a studied concerete structure. End faces were smoothed plane parallel, measured, weighed and steel cylinders of 50 mm were glued to the faces. After one to two days under the compression machine, the samples tested were subjected to compression up to breaking.

The breaking compression stress was calculated taking into account sample dimensions according to the British Standard (BS 1881, BS 6089 clause 7).

On the basis of the statistical procedure, the bulk density and compressive strength of concrete in each sample and the corresponding cubic compressive strength $f_{ck,cube}$ and the mean compressive strength of sample series, standard deviation and the variation factor were calculated. On the basis of these data, class C (according to EN norms) of concrete compressive strength of the tested structural element(s) was determined.

Results

Table 5.2 presents summary results of the determination of the class of concrete compressive strength.

Tabel 5.2

Pos.	Element/ Parameter	Shell	Edge elements, girders	Columns
1	Number of center punches	12	7	3
2	Mean bulk density, g/cm^3	1.94	2.05	2.32
3	Standard deviation -”-	0.05	0.06	0.03
4	Variation factor %	2.46	3.08	1.08
5	Mean compressive strength $f_{ck,\ cube}$, N/mm^2	12.9	20.5	28.0
6	Same, kg/cm^2	129	205	280
7	Standard deviation, N/mm^2	1.4	1.5	0.9
8	Variation factor, %	10.7	7.3	3.0
9	Class according to EN 206-1	C8/10	C16/20	C20/25

- **Concrete compressive strength of Papisaare hydroplane hangars**

Methods of determination of concrete compressive strength

The compressive strength of the samples was determined according to the requirements of the standard EVS-EN 12390:2002. Two concrete pieces were selected as samples for testing, one of which was taken from the reinfornced concrete column and the other from the main beam. In the laboratory two testing sample cubes of the dimensions 10x10x10 cm and 7x7x7 cm were sawn out from the concrete pieces.

Results

The compressive strength of the testing sample cubes sawn out is shown in Table 5.3.

Table 5.3

Testing Sample denotation		Dimensions, mm			A, cm^2	Mass, kg	Apparent-density kg/m^3	F, kN	Compressive strength, $f_{ck,\,cube}$ N/mm^2
		a	b	h					
Papisaare	1	101.0	103.0	101.0	104.0	2.3	2180.0	201.0	19.5
	2	68.5	73.0	73.5	50.0	0.8	2180.0	120.0	24.0
					Mean compressive strength,N/mm^2:				21.8
					Class according to EN 206-1:				C16/20

5.4.3 Frost resistance

Methods for the determination of concrete frost resistance

Samples were as follows:

1. Two concrete cylindrical samples of the diameter of 90 mm and L=60 mm and L=80 mm drilled out of reinforced concrete shells of the Tallinn hydroplane hangar.
2. Two concrete pieces out of which one was taken from the reinforced concrete column and the other from the main beam served as testing samples for the Papisaare hydroplane hangars. In the laboratory four cubic testing samples of the dimensions of 50x100x100 mm were sawn out.

Frost resistance of concrete testing sample no. 1 was determined according to the requirements of the standard GOST 10060-87 I.

To determine frost resistance, two cylindrical core samples with the diameter of 90 mm and depth of 60 and 80 mm were drilled out. These samples were drilled out exactly on the places, which were repaired by stitching dogs and glued cracks. According to those methods concrete cylinders were frozen in a thermoclave "Neva" with enforced air circulation at the temperature from 18+ to -2^{0}C from 2.5 to 16 hours and melted in ordinary water at the temperatures +(18+-2)^{0}C from 2.0 (+-0,5) hours. One freezing cycle consisted of freezing and melting, thus one freezing cycle lasted from 4.5 to 18.5 hours (freezing + melting).

The frost resistance of concrete was assessed by the number of cycles during which mass loss of the testing samples did not exceed 3% of the initial mass. At the beginning of the test and after each of the five freezing cycles, an external observation of concrete cylinders was done and mass changes were determined. Concrete frost resistance was assessed through the mass change at testing according to the GOST 10060-87 II method.

Frost resistance of testing samples no. 2 was determined according to the standard EVS 814:2003.

Papisaare testing samples were sawn out from concrete pieces such that the outer surface of the structure remained the outer surface of the testing sample during the freezing-melting testing. Testing sample dimensions and densities are presented in Table 5.5.

The testing samples sawn out were kept in a climate room at the temperature (20+-2)^{0}C and relative moisture (65+-5)% up to the beginning of freezing-melting. On the 3rd-5th day, a rubber cover was glued to the testing samples, whereas the edge of the cover extended 20 mm over the edge of the sample, thus enabling freezing substance to be retained on the surface of the sample. In addition, sides of the sample and the lower part were insulated by a thermal insulation material. On the 7^{th} day of maintenance, a 3 mm layer of distilled water at the temperature $(20+-2)^0$C was poured on the sample and it was kept for (72+-2) hours at the temperature $(20+-2)^0$C.

Distilled water was replaced by a 3 mm thick freezing substance – new distilled water layer at the temperature $(20+-2)^0$C before sample was placed into the froster. To avoid freezing substance vaporization, the testing sample was covered by a polyethylene film. Freezing and melting were carried out according to the conditions provided in the standard in the climate chamber with enforced air circulation. One freezing-melting cycle lasted for 24 hours. After 7, 14, 28, 42, 56 cycles the quantity of material spalling on the outer surface of the testing sample was determined. To remove the whole of the spalling, the material was poured from the surface of the testing sample to a container together with the freezing substance, followed by surface cleaning by water spray. The material spalling was separated from the liquid by filtration, then dried and weighted. For subsequent cycles, a new quantity of freezing substances was poured on the sample.

After completion of the above cycles, mass loss of each sample was determined and the summary spalling quantity Σ M (g) and the summary mass loss per unit of area - Σ S (kg/m^2) were calculated. To assess frost resistance, the mean mass loss per unit of area of four testing samples was calculated.

Results

- **Frost resistance of concrete for reinforced concrete shells of Tallinn hydroplane hangars**

Mass loss of 3% was exceeded for the first concrete cylinder after 110 and for the second after 150 freezing cycles.

Table 5.4 presents changes in concrete cylinder mass in the determined frost resistance of up to 150 freezing cycles.

Table 5.4 Changes of concrete cylinder mass in the determined frost resistance of up to 150 freezing cycles

Sample No.	Sample mass, g; sample mass change %, after frost resistance cycle											
	0		5		10		15		20		25	
	g	%	g	%	g	%	g	%	g	%	g	%
1	752.4	0.0	752.5	0.0	752.3	0.0	752.2	0.0	751.9	-0.1	752.0	-0.1
2	950.9	0.0	951.2	0.0	951.0	0.0	950.7	0.0	950.5	0.0	951.2	0.0
Sample No.	Sample mass, g; sample mass change %, after frost resistance cycle											
	30		35		40		45		50		55	
	g	%	g	%	g	%	g	%	g	%	g	%
1	751.0	-0.2	750.9	-0.2	750.3	-0.3	748.9	-0.5	746.5	-0.8	745.3	-0.9
2	950.5	0.0	951.1	0.0	950.8	0.0	950.2	-0.1	950.7	0.0	950.3	-0.1
Sample No.	Sample mass, g; sample mass change %, after frost resistance cycle											
	60		65		70		75		80		85	
	g	%	g	%	g	%	g	%	g	%	g	%
1	745.6	-0.9	743.4	-1.2	744.2	-1.1	739.5	-1.7	737.4	-2.0	736,8	-2.1
2	950.0	-0.1	950.7	0.0	951.9	0.1	950.1	-0.1	950.1	-0.1	950,4	-0.1
Sample No.	Sample mass, g; sample mass change %, after frost resistance cycle											
	90		100		105		110		115		120	
	g	%	g	%	g	%	g	%	g	%	g	%
1	736.4	-2.1	734.3	-2.4	731.4	-2.8	727.9	-3.3	716.5	-4.8	679.7	-9.7
2	950.6	0.0	950.9	0.0	951.4	0.1	951.3	0.0	950.9	0.0	950.9	0.0
Sample No.	Sample mass, g; sample mass change %, after frost resistance cycle											
	125		130		135		140		145		150	
	g	%	g	%	g	%	g	%	g	%	g	%
1	676.3	-10.1	668.3	-11.2	666.9	-11.4	661.8	-12.0	664.3	-11.7	645.9	-14.2
2	950.9	0.0	950.9	0.0	951.0	0.0	936.5	-1.5	938.2	-1.3	922.8	-3.0

- **Frost resistance of concrete for reinforced concrete beams and columns of Papisaare hydroplane hangars**
  - Frost resistance tests carried out in distilled water according to the requirements of EVS 814:2003 showed that the concrete samples sawn out had a mass loss of 0.01 kg/m² after 56 cycles.
  - The result obtained is in compliance with class KK3 of concrete frost resistance of the standard EVS 814:2003 requirements S_{56}<0.20 kg/m^2.

- Sample mass loss in the determined frost resistance of up to 56 freezing cyles is shown in Tables 5.5, 5.6.

Table 5.5 Dimensions, mass and density of concrete samples of Papisaare hydroplane hangars before frost resistance tests

Sample	Sample dimensions, mm							Mass,	Density, kg/m^3	
No.	a	b	h_1	h_2	h_3	h_4	h_{mean}	g	single	mean
1	102.0	107.0	50.7	51.1	51.0	49.4	50.6	1186	2150	2160
2	101.0	112.0	51.2	51.0	51.5	51.7	51.4	1240	2130	
3	128.0	102.5	51.0	50.6	50.2	50.2	50.5	1467	2210	
4	130.5	101.5	50.5	50.5	50.6	50.5	50.5	1441	2150	

Table 5.6 Sample mass loss of the Papisaare hydroplane hangar in the determined frost resistance according to the requirements of EVS 814:2003

Sample No.	Dimensions, mm		Area A, cm^2	Mass loss unit	Sample mass lost after frost resistance cycle				
	a	b			7	14	28	42	56
1	102.0	107.0	109.1	Σ M, g	0.0	0.0	0.1	0.1	0.2
				Σ S, kg/m^2	0.00	0.00	0.01	0.01	0.02
2	101.0	112.0	113.1	Σ M, g	0.0	0.0	0.0	0.1	0.1
				Σ S, kg/m^2	0.00	0.00	0.00	0.01	0.01
3	128.0	102.5	131.2	Σ M, g	0.0	0.0	0.1	0.1	0.2
				Σ S, kg/m^2	0.00	0.00	0.01	0.01	0.02
4	130.5	101.5	132.5	Σ M, g	0.0	0.0	0.0	0.0	0.0
				Σ S, kg/m^2	0.00	0.00	0.00	0.00	0.00
			Mean	Σ M, g	0.0	0.0	0.1	0.1	0.1
				Σ S, kg/m^2	0.00	0.00	0.01	0.01	0.01

5.4.4 Mineralogical composition of cement stone of concrete

An overview of the mineralogical composition of cement stone of concrete allows for assessments of extent and reasons for concrete corrosion occurrence.

X ray fraction analysis of cement stone

X ray fraction analysis was used to find the mineralogical composition of the cement stone of concrete. X ray analysis enables one to determine existence and quantity of substances of crystal structure in concrete.

Implementation of X ray fraction analysis methods

Samples for X ray analysis were taken as follows:

1) from reinforced concrete shells in the depth of 40-50 mm of the outer surface in Tallinn hydroplane hangars;

2) seven samples from a reinforced column in the depth of 0-150 mm from the outer surface in the Tallinn hydroplane hangar. A concrete sample of L=150 mm of a diameter of 70 mm sawn out was cut into samples of 20 mm in thickness, a total of 7 samples, out of which 5 were studied from the outer surface of the column towards the centre, samples L1 – L5;

3) seven samples from a reinforced concrete column in the depth of 0-150 mm from the outer surface in the case of the Papisaare hydroplane hangar. A cylindrical sample of L=150 mm of a diameter of 70 mm sawn out was cut into samples of 20 mm in thickness, a total of 7 samples, out of which 5 were studied from the outer surface of the column towards the centre, samples P1 – P5.

For this research concrete samples were enriched for cement stone by ca 50%, i.e. the sample was crushed in a muller and in as much as possible the amorphic part was sieved out.

A detailed account of physical aspects of X ray fraction analysis can be found in literature [27].

Results

Since the minerals of cement stone are X ray amorphic (excl. $Ca(OH)_2$), X ray fraction analysis provides information about the content of secondary crystal substances and minerals of cement clinker if no sufficient contact with water has been reached in the concrete production process.

Thus, based on the X ray analysis results, in terms of extent of (reinforced) concrete corrosion and reason of occurrence, the depth of concrete carbonation (content of $Ca(OH)_2$) and structure (porosity) are of interest..

- **Concrete sample of Tallinn hydroplane hangar shell**

As a result of X ray analysis the following crystal substances were found in the cement stone:

- Quartz – SiO_2 – aggregate (main component of sand),
- Calcite – $CaCO_3$ – product of cement stone carbonation, aggregate,
- Portlandite - $Ca(OH)_2$ – calcium hydroxide, in cement stone,
- Microcline – $KAlSi_3O_8$ – granite component is contained in quartz sand,
- Anorthite – $CaAl_2Si_2O_8$ – is contained in quartz sand.

In the sample studied, no crystal substanes containing sulphate were found.

As a result of X ray analysis, it was found that chloride content in cement stone stays below the sensitivity of 0.5% and the content of corrosion product ettringite ($3CaO*Al_2O_3*3CaSO_4*21H_2O$) is below 3% .

Powder difractogram of concrete X ray analysis of Tallinn hydroplane hangar shells is shown in Figure 5.12.

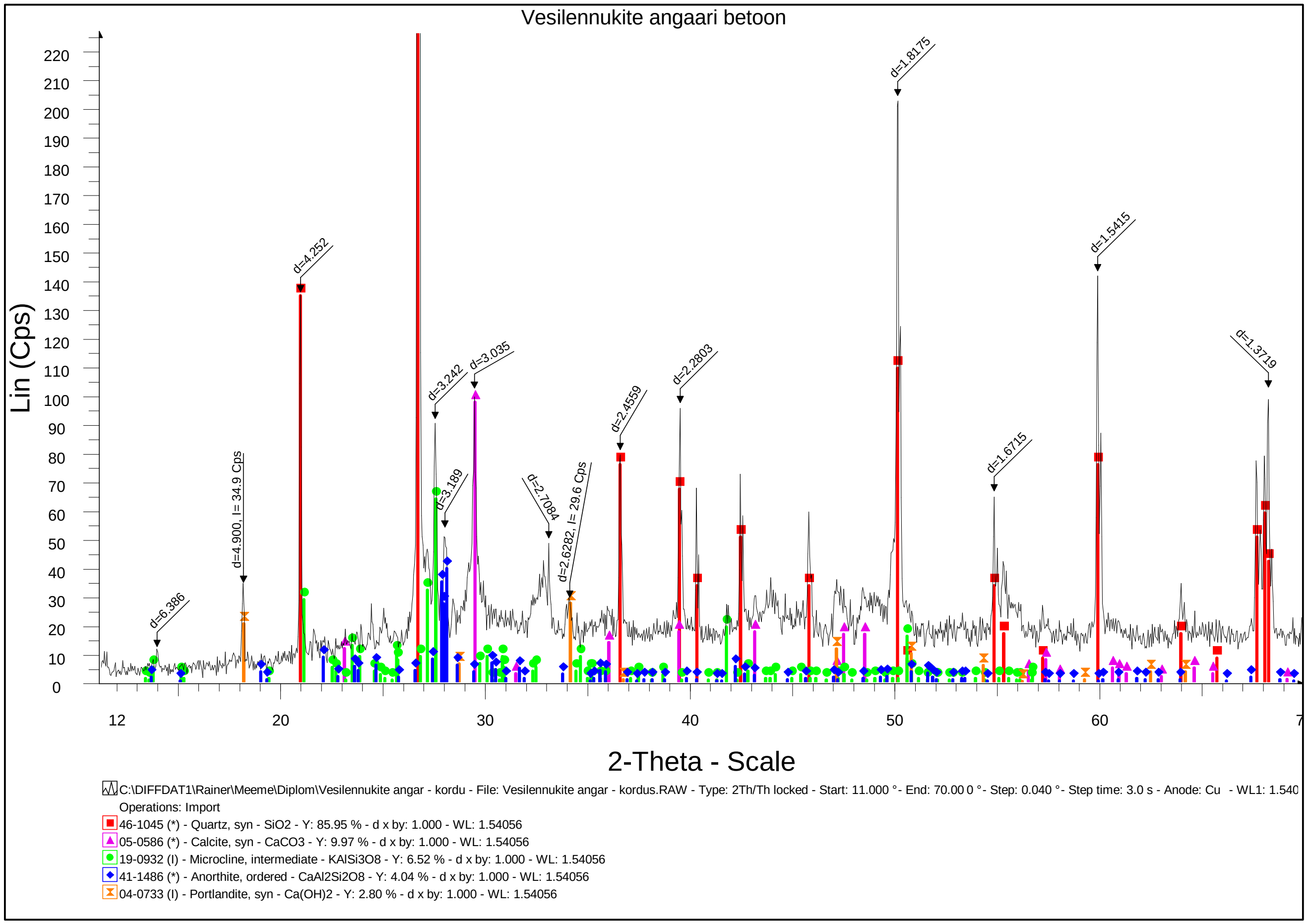


Figure 5.12 Powder difractogram of X ray analysis of concrete shells of the Tallinn hydroplane hangar

- **Concrete samples of the Tallinn hydroplane hangar column**

X ray analysis results showed the content of the following crystal substances in the concrete stone:

- Quartz – SiO_2 – aggregate (main component of sand),
- Calcite – $CaCO_3$ – carbonation product of cement stone,
- Portlandite - $Ca(OH)_2$ – calcium hydroxide, in cement stone,
- Ettringite – corrosion product of cement stone, in cement stone,
- Halite – NaCl – is contained in sand.

No crystal substances containing sulphite were found in the samples studied.
Chloride content was found on the boundary of measuring sensitivity.

$Ca(OH)_2$ concentration was found increasing in samples L1 – L5, i.e.from the outer surface of the column towards the centre.

Powder difractograms of X ray analysis of the hydroplane hangar column are presented in Figures 5.13, 5.14 (samples L1-L2 and L3-L4-L5) and in Figure 5.15 $Ca(OH)_2$ relative concentrations in samples L1- L5 are shown.

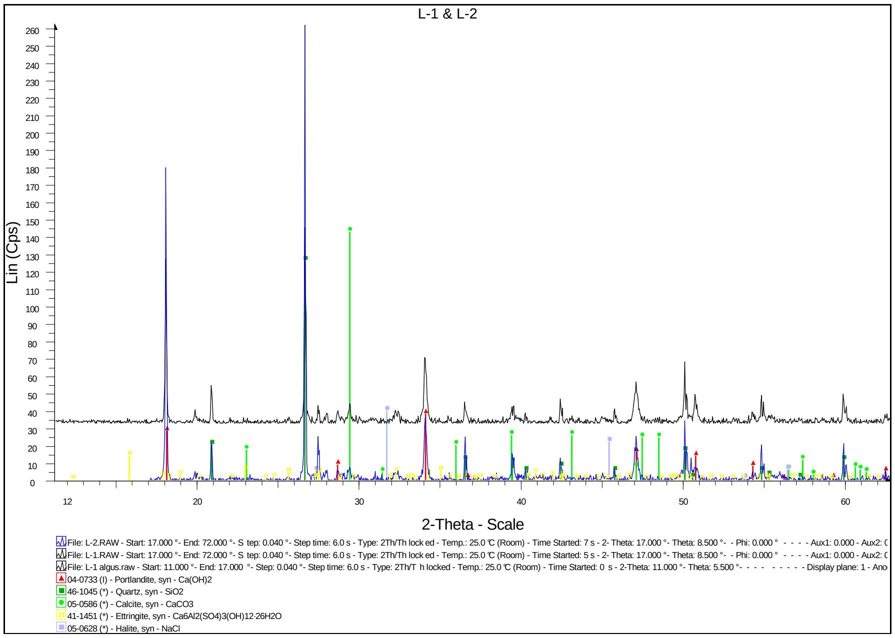


Figure 5.13 Powder difractogram of X ray analysis of the Tallinn hydroplane hangar column, samples from the depth of 0 - 20mm (sample L1) and 20 – 40 mm (sample L2) from the outer surface

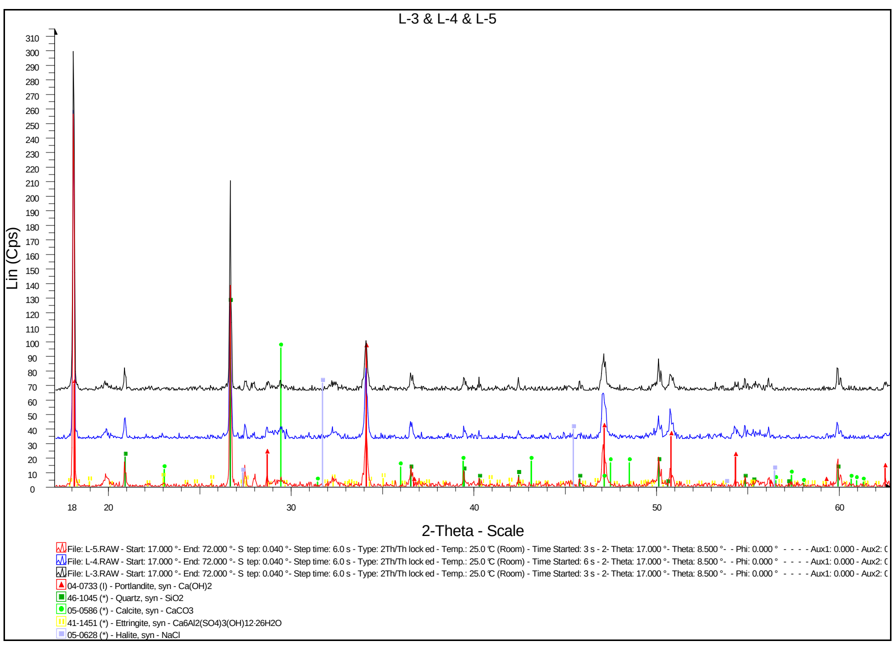


Figure 5.14 Powder difractogram of X ray analysis of the Tallinn hydroplane hangar column concrete samples from the depth of 40 mm (sample L3) up to 100 mm (sample L5 from the outer surface)

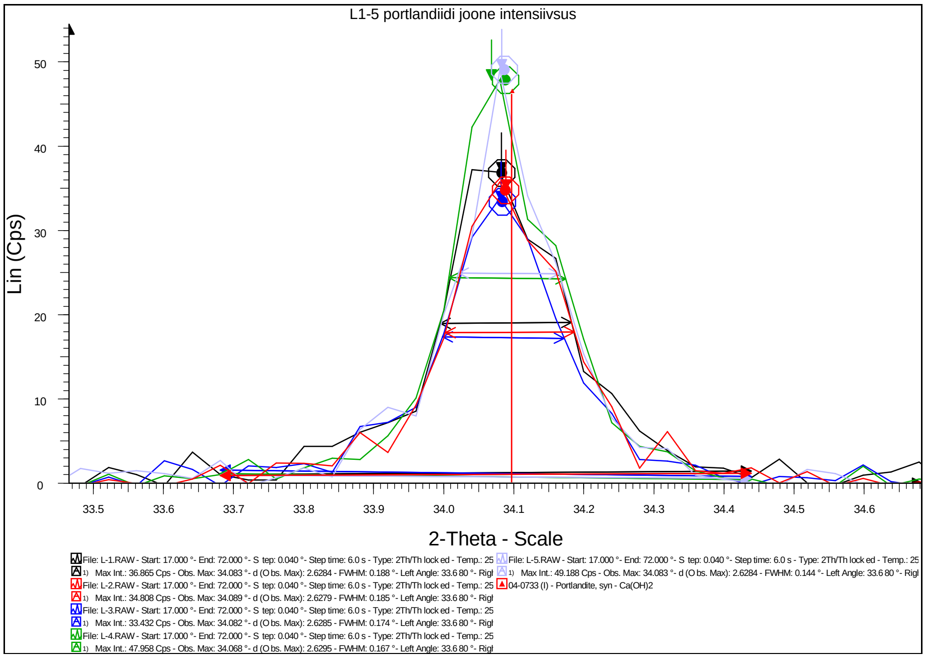


Figure5.15 Powder difractogram of X ray analysis of the Tallinn hydroplane hangar column concrete, $Ca(OH)_2$ relative concentrations (intensities), samples from the depth of outer surface (sample L1) up to 100 mm of depth on the inner surface (sample L5)

- **Concrete sample of the Papisaare hydroplane hangar column**

As a result of X ray analysis, the following crystal substances were found in the cement stone:

- Quartz – SiO_2 – aggregate (main component of sand),
- Calcite – $CaCO_3$ – carbonation product of cement stone,
- Portlandite - $Ca(OH)_2$ – calcium hydroxide, in cement stone,
- Ettringite – corrosion product of cement stone, in cement stone,
- Halite – NaCl – is contained in sand,
- Dolomite – $CaMg(CO_3)_2$ – aggregate (is contained in sand).

No crystal substances containing sulphates were found in the samples studied. Chloride content was found on the boundary of measurement accuracy.

The results showed $Ca(OH)_2$ in samples P1, P2 to be on the boundary of measuring accuracy sensitivity and in extremely small quantity in sample P3. In samples P4 and P5, $Ca(OH)_2$ concentration had increased.

Powder difractograms of X ray analysis of Papisaare hangar column concrete are shown in Figures 5.16 and 5.17.(samples P1-P2 and P3-P4-P5) and in Figure 5.18 relative concentrations of $Ca(OH)_2$ in samples P1- P5.

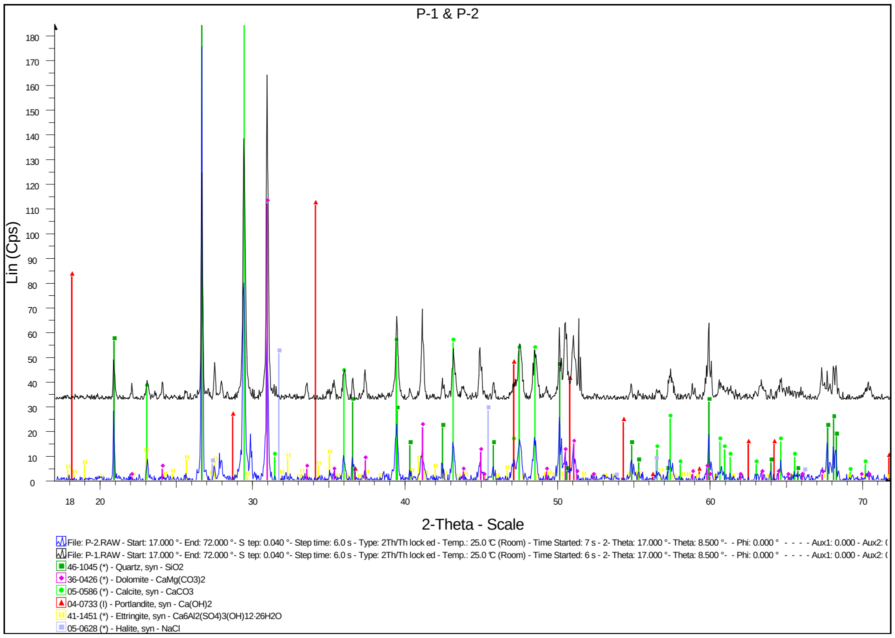


Figure 5.16 Powder difractogram of X ray analysis of Papisaare hydroplane hangar column concrete, samples from the depth of 0–20 mm (sample P1) and 20–40 mm (sample P2) from the outer surface

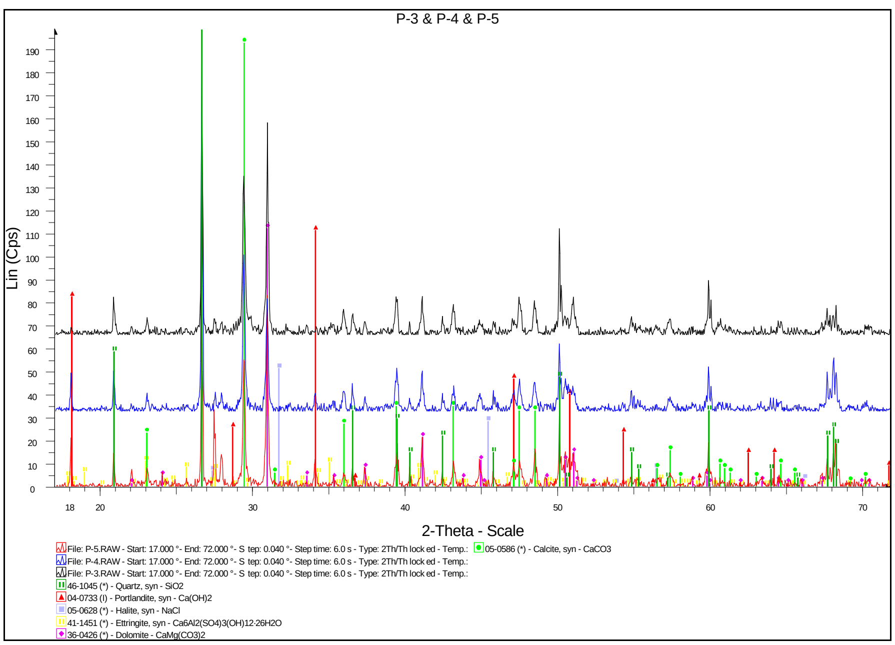


Figure 5.17 Powder difractogram of X ray analysis of Papisaare hydroplane hangar column concrete, samples from the depth of 40 mm (sample P3) up to 100 mm (sample P5) from the outer surface

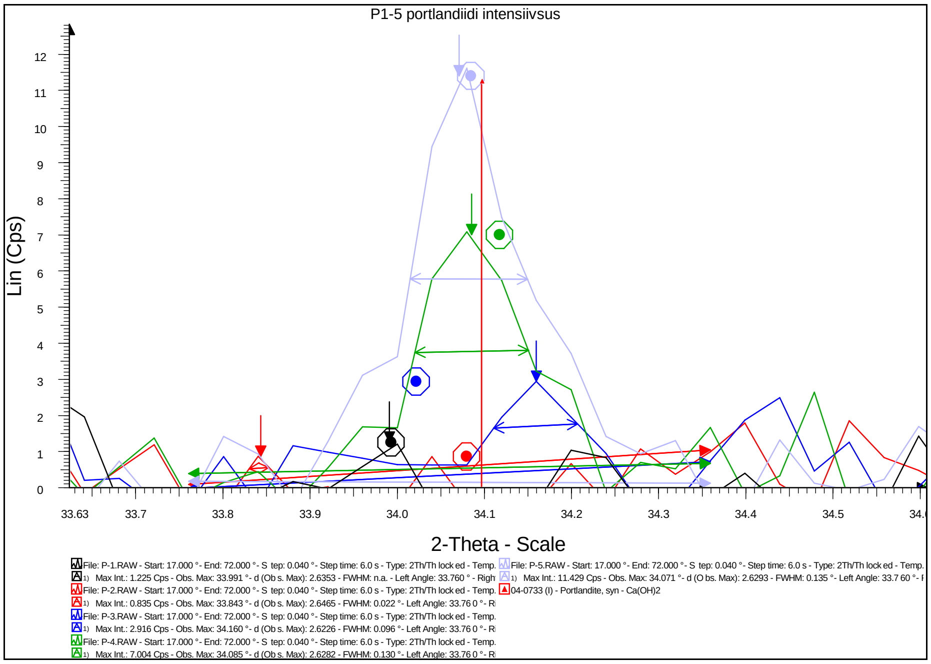


Figure 5.18 Powder difractogram of X ray analysis of Papisaare hydroplane hangar column concrete, relative concentrations (intensities) of $Ca(OH)_2$, samples from the depth, outer surface, (P1) up to 100 mm of column depth, on the inner surface (P5)

5.4.5 Carbonation depth of concrete

Methods of assessment of concrete carbonation depth

Laboratory techniques which can be used to determine the depth of carbonation include chemical analysis, X-ray diffraction, infra-red spectroscopy, and thermogravimetric analysis.

Since limestone was used as an aggregate in the concrete both for Tallinn and Papisaare hydroplane hangars, it is not possible to determine the extent of carbonation based on the $CaCO_3$ content. Carbonation extent can be assessed according to $Ca(OH)_2$ content in the cement stone of concrete by comparing results of concrete samples in different depths obtained by the quantitative X-ray phase analysis (see Section 5.4.4).

Carbonation depth can be determined approximately also using the phenolphthalein ($C_{20}H_{14}O_4$) solution. In the alkaline environment ($Ca(OH)_2$), phenolphalein solution turns rosy, in the case of $pH < 8.5$ cement stone of concrete is carbonated, neutral and no solution colouring occurs.

Results

- **Concrete sample of Tallinna hydroplane hangar reinforced concrete shells**

Powder X-ray difractogram of Tallinn hydroplane hangar shell concrete is shown in Figure 5.12.

A relatively small part of $Ca(OH)_2$ in the depth of outer surface in the cross-section of 40-50 mm showed approximately that concrete is carbonated on the whole extent of reinforced concrete cross-section.

The result obtained was confirmed by a subsequent verification with phenolphlatein.

- **Concrete samples of Tallinn hydroplane hangar reinforced column**

Powder X-ray difractogram of Tallinn hydroplane hangar column concrete is presented in Figures 5.13 and 5.14 (samples L1-L2; L3-L4-L5) and in Figure 5.15 $Ca(OH)_2$ relative concentrations in samples L1- L5.

The relatively large part of $Ca(OH)_2$ increasing from the outer surface of the column to the centre (samples L1, L2, L3, L4, L5) shows approximately that in the column studied no carbonation of concrete occurred below 20 mm from the outer surface.

The result obtained was confirmed by subsequent verification by phenolphthalein.

- **Concrete samples of Saaremaa hydroplane hangar reinforced concrete columns**

Powder X-ray difractogram of Saaremaa hydroplane hangar column concrete is presented in Figures 5.16 and 5.17 (samples P1-P2; P3-P4-P5) and in Figure 5.18 relative concentrations of $Ca(OH)_2$ in samples P1- P5 are shown.

A relatively small part of $Ca(OH)_2$ in samples P1 and P2 close to outer surface, and at the same time increasing towards the centre (samples P3, P4, P5) shows that concrete carbonation occurred from the outer surface towards the centre of the column up to 40mm.

The result obtained was confirmed by subsequent verification with phenolphlatein and loose and concrete spalling around the reinforcement steel bars.

Figure 5.19 Stalactides suspending from the lower shell surface

* Water flowing through concrete dissolves calcium in the form of $Ca(OH)_2$ that is quickly carbonated in the air, resulting in the formation of stalactides (Figure 5.19).

5.4.6 Morphology studies of concrete fracture surface

Morphology studies of concrete fracture surface by scanning electronmicroscopy enable one to determine existence of both structurally crystalline and amorfous substances in concrete [36; 37; 38], obtain information about porosity and the character and extent of corrosion.

Procedures for morphology studies methods of concrete fracture surfaces

The following concrete samples were used to determine the character of concrete fracture surface by electronmicroscopy:

1) concrete samples from the depth of 40-50 mm of shell outer surface in the case of Tallinn hydroplane hangar reinforced concrete shell;

2) two samples were taken from Tallinn hydroplane hangar reinforced concrete column. A cylindrical shell of 70 mm in diameter, L=150 mm, drilled out was sawn into 7 samples of 20 mm, out of which 2 samples were studied from the outer surface towards the centre of the column, 20-40 mm (L2) and in the depth of 80-100 mm (L5);

3) two samples were taken from Papisaare hydroplane hangar reinforced concrete column. A cylindrical sample of 70 mm in diameter, L=150 mm, drilled out was sawn into a total of 7 samples of 20 mm, out of which 2 samples were studied from the outer surface towards the centre of the column, 20-40 mm (P2) and 80-100 mm (P5) in depth.

Results

- **Samples of Tallinn hydroplane hangar reinforced concrete shells**

On the basis of Figures 5.20 – 5.24 made by electronmicroscopy it is clear that the following substances have been identified by experience in the concrete of hydroplane hangar shells:

- calcite ($CaCO_3$) crystals, Figures 5.20, 5.24;
- portlandite ($Ca(OH)_2$) crystals, Figure 5.22;
- it may be concluded by the Figures 5.21, 5.22 and 5.23 that it is the case with ettringite ($3CaO*Al_2O_3*3CaSO_4*21H_2O$), although it was not confirmed by the X-ray analysis (see Section 5.4.4);
- aluminate crystals or allides in Figures 5.20, 5.24;
- amorfous substance in Figures 5.21, 5.22 and 5.23.

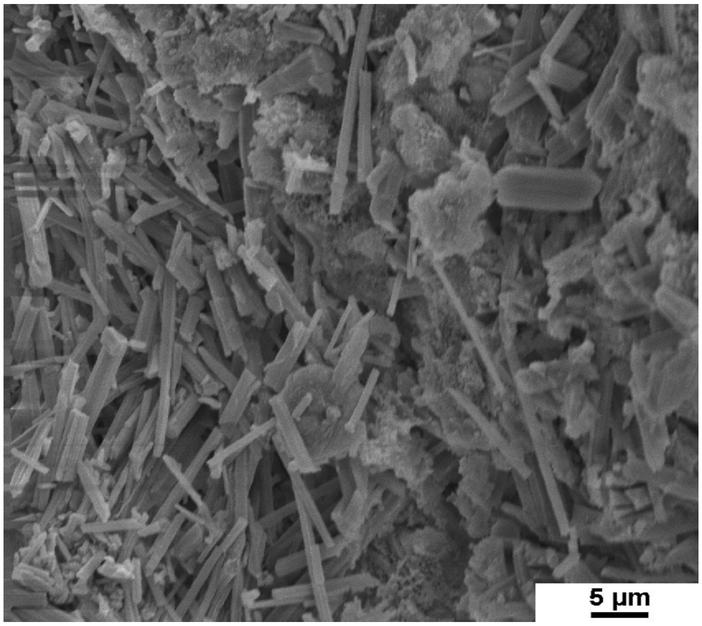


Figure 5.20 Concrete fracture surface taken in the depth of 40-50 mm of the hydroplane hangar shell

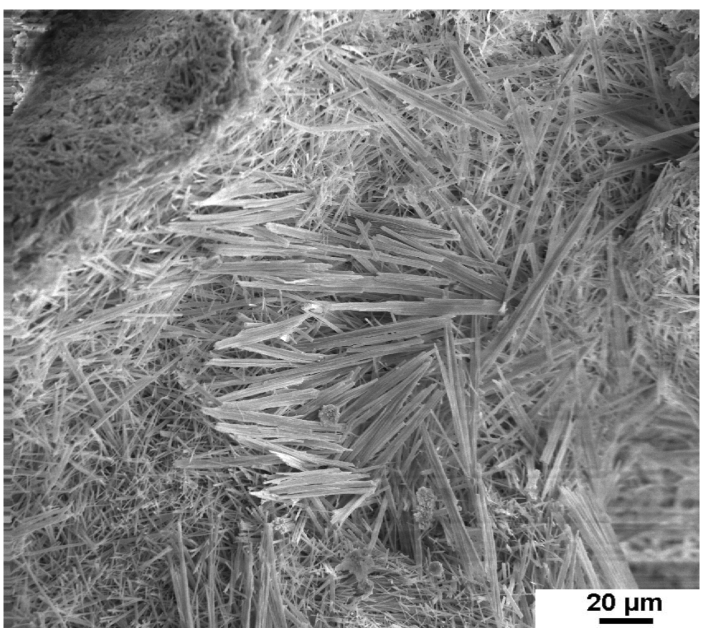


Figure 5.21 Concrete fracture surface taken in the depth of 40-50 mm of the hydroplane hangar shell

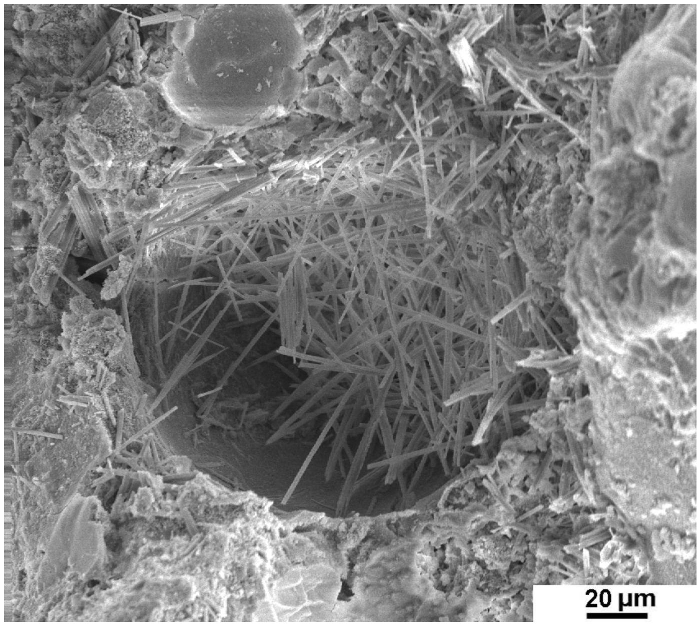


Figure 5.22 Concrete fracture surface taken in the depth of 40-50mm of the hydroplane hangar shell

Figure 5.23 Concrete fracture surface taken in the depth of 40-50 mm of the hydroplane hangar shell

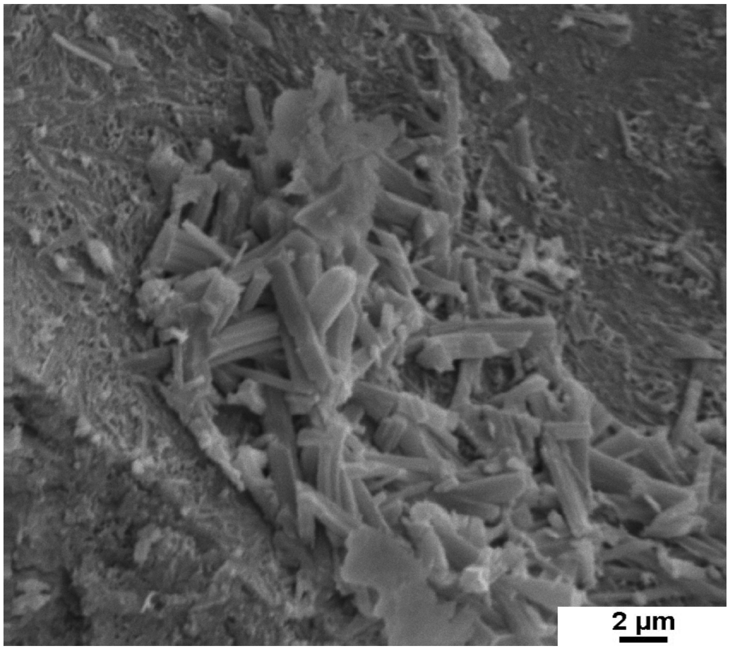


Figure 5.24 Concrete fracture surface taken in the depth of 40-50 mm of the hydroplane hangar shell

- **Samples of reinforced columns of the Tallinn hydroplane hangar**

Only a few photos are presented here to characterize concrete from reinforced concrete columns of Tallinn hydroplane hangars in the depth of 20-40 mm and 80-100 mm from the outer surface of the column. Figure 5.25 shows a general view of the concrete, giving evidence of porous concrete, whereas pores are filled with crystals. The general view presented in Figure 5.27 gives evidence of high density cement stone, revealing few pores.

According to the photos made by electron microscopy, the following substances can be identified by experience in the concrete from Tallinn hydroplane hangar reinforced concrete columns:

- calcite ($CaCO_3$) crystals, Figures 5.26, 5.28;
- portlandite ($Ca(OH)_2$) crystals, Figures 5.26, 5.28;
- ettringite ($3CaO*Al_2O_3*3CaSO_4*21H_2O$) crystals, Figures 5.26, 5.28;
- on the basis of the photos it could be concluded that aluminide crystals or allites can also be found in the samples, although it was not confirmed by X-ray analysis, Figures 5.26, 5.28.

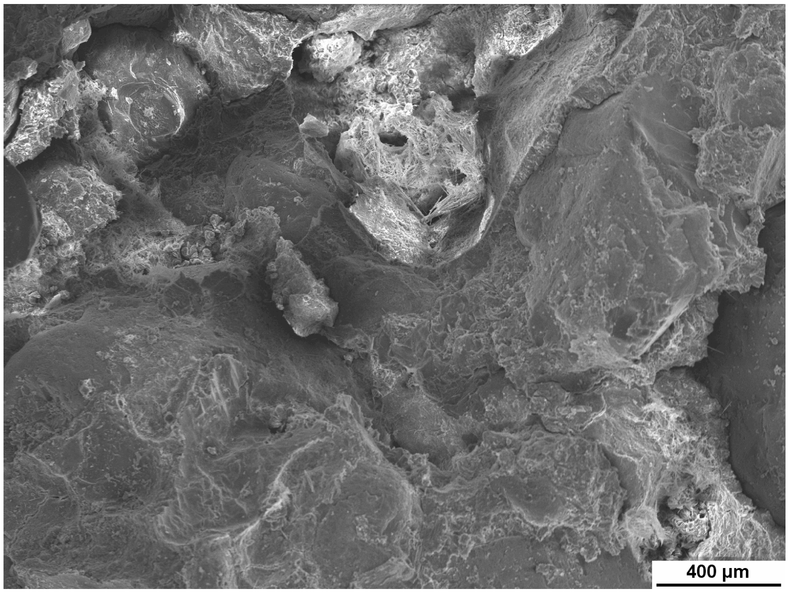


Figure 5.25 Concrete fracture surface taken in the depth of 20-40 mm (sample L2) of the hydroplane hangar reinforced concrete column

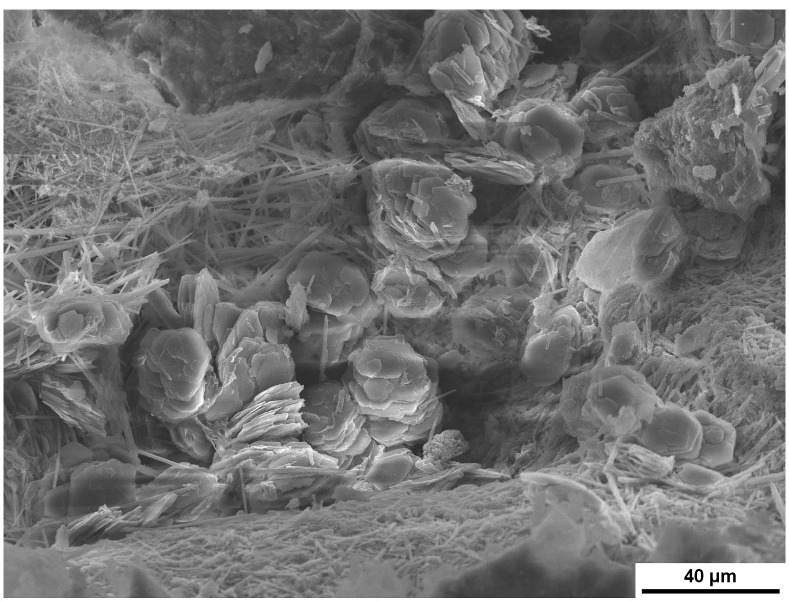


Figure 5.26 Concrete fracture surface taken in the depth of 20-40 mm (sample L2) of the hydroplane hangar reinforced concrete column

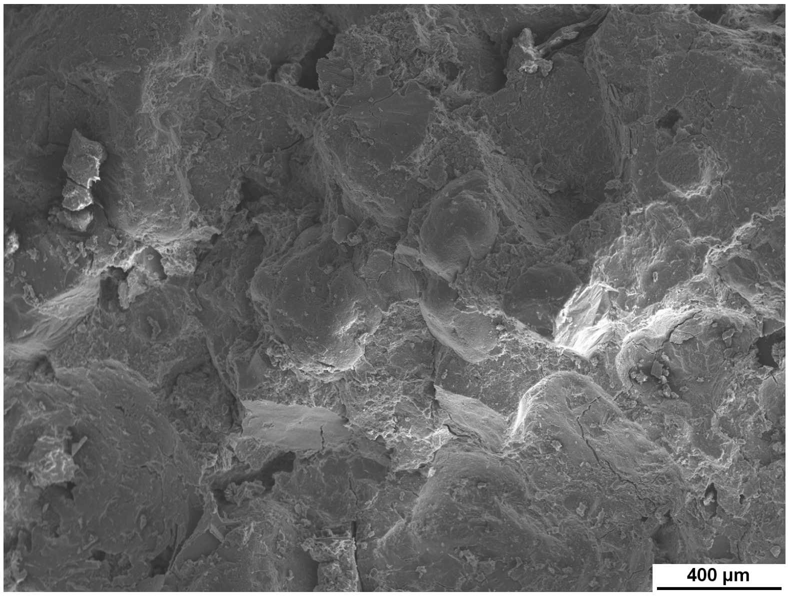


Figure 5.27 Concrete fracture surface taken in the depth of 80-100 mm (sample L5) of the hydroplane hangar reinforced column

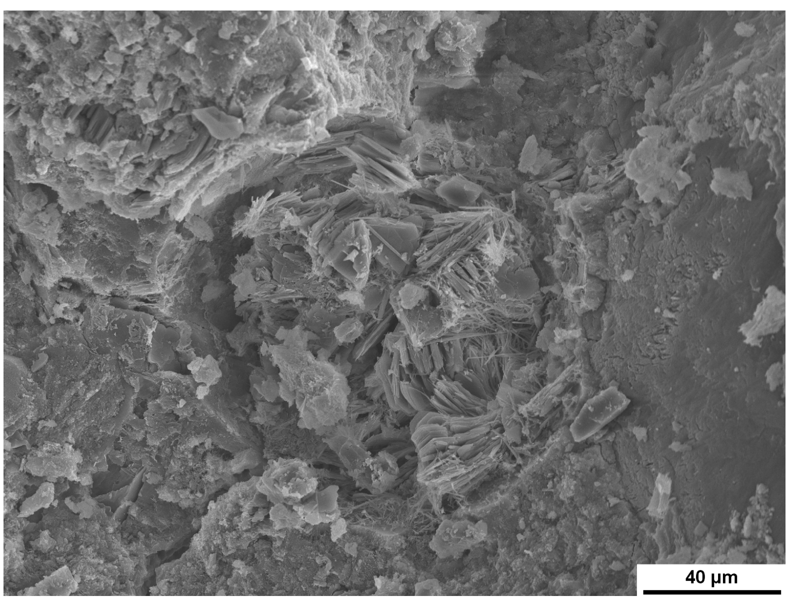


Figure 5.28 Concrete fracture surface taken in the depth of 80-100 mm (sample L5) of the hydroplane hangar reinforced concrete column

- **Samples of Papisaare hydroplane hangar reinforced concrete columns**

Only a few photos are presented here to characterize the concrete of Papisaare hydroplane hangar reinforced concrete columns in the depth of 20-40 mm and 80-100 mm from the outer surface of the column. A general view of concrete is presented in Figures 5.29 and 5.32, giving evidence of few pores and high density binder that is surrounding the aggregate particles. At the same time there is a fracture between the binder and the aggregate.

According to the Figures taken by electronmicroscopy, the following substances were identified by experience in the concrete of Papisaare hydroplane hangar reinforced concrete column:

- calcite ($CaCO_3$) crystals, Figure 5.32;
- portlandite ($Ca(OH)_2$) crystals, Figure 5.32;
- ettringite ($3CaO*Al_2O_3*3CaSO_4*21H_2O$) crystals, Figures 5.30; 5.32;
- on the basis of Figure 5.32 it may be concluded that the samples also contain aluminide crystals or allides, although X-ray analysis did not confirm that.

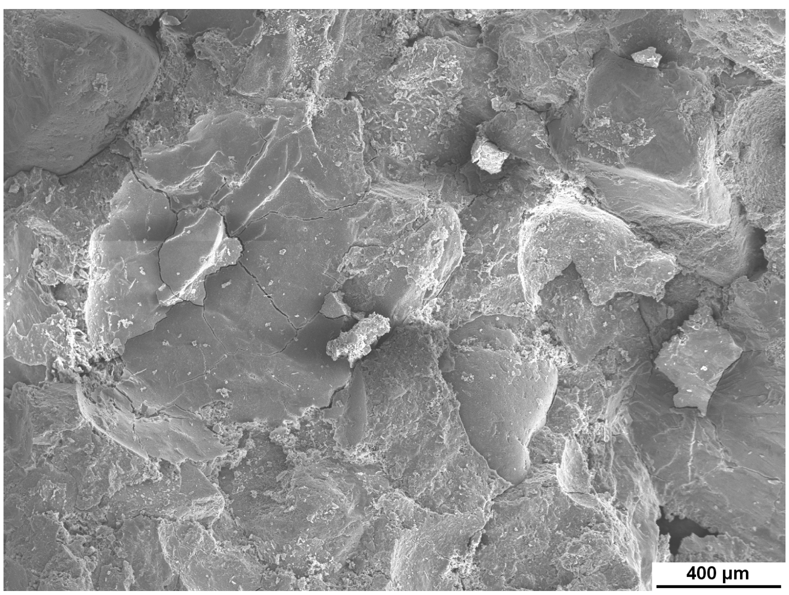


Figure 5.29 Concrete fracture surface taken in the depth of 20-40 mm (sample P2) of the Papisaare hydroplane hangar reinforced concrete column

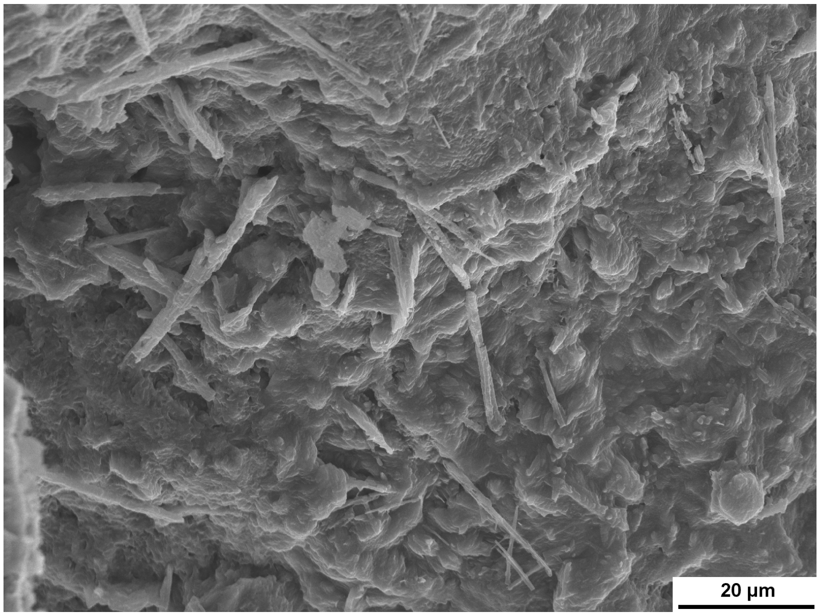


Figure 5.30 Concrete fracture surface taken in the depth of 20-40 mm (sample P2) of the Papisaare hydroplane hangar reinforced concrete column

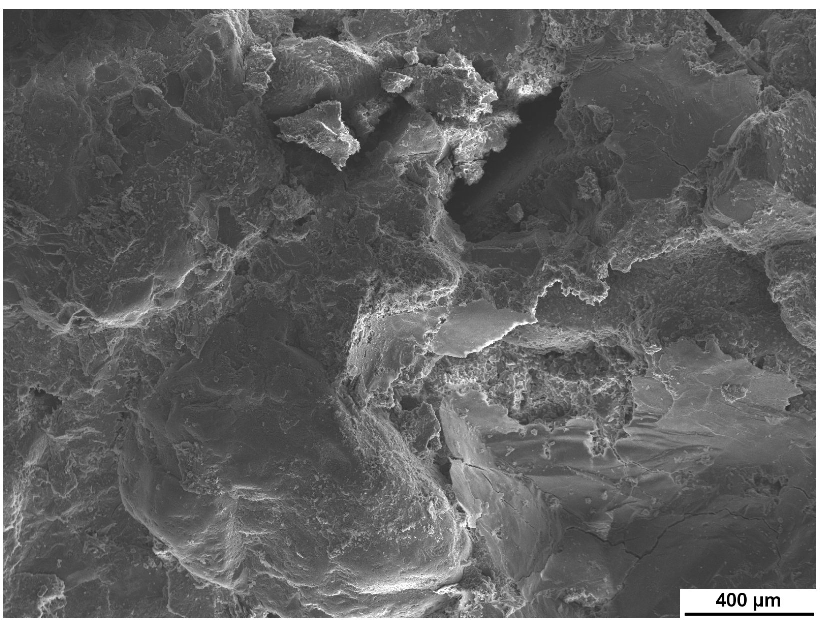


Figure 5.31 Concrete fracture surface taken in the depth of 80-100 mm (sample P5) of the Papisaare hydroplane hangar reinforced concrete column

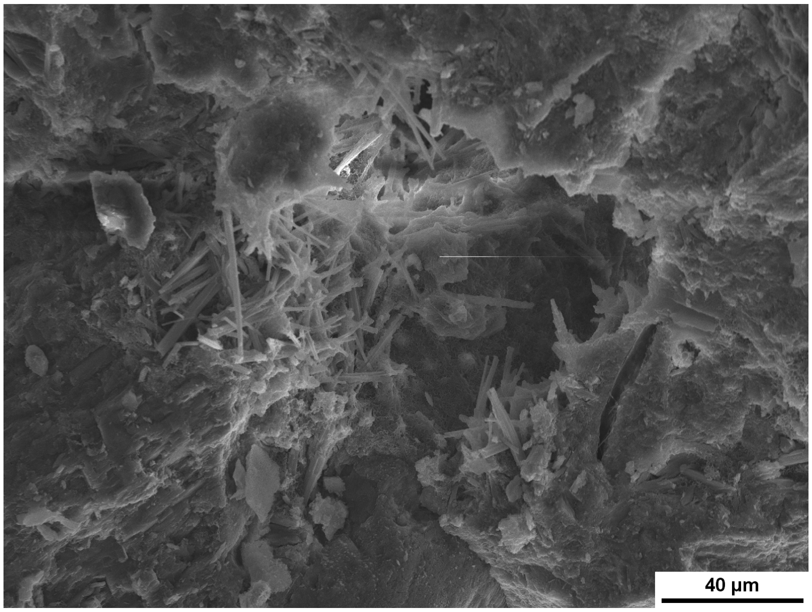


Figure 5.32 Concrete fracture surface taken in the depth of 80-100 mm (sample P5) of the Papisaare hydroplane hangar reinforced concrete column

5.4.7 Water absorption of concrete

Methods of determination of water absorption of concrete

Water absorption is a property that enables one to characterize porosity of concrete. The following parameters are essential in the estimation of porosity:

1. water absorption in relation to dry mass of a sample;
2. speed of water absorption.

Absolute water absorption was determined according to the German DIN 52 103 norm and the Finnish standard of SFS 5513.

Since neither the estimation of water absorption of concrete in percentage nor the resulting porosity have been standardized according to information available, the estimation is made through expert estimates (based on experience).

For purposes of comparison, in addition to the water absorption of concrete from Tallinn and Papisaare hydroplane hangars, the water absorption of concrete from the Helsinki Olympic Stadium (built in the 1930s) was estimated. The stadium originates from the same period and is likely to have been constructed from similar components and by a similar technology.

Sample parameters were as follows:

1. cubes of 20 x 20 x 20 mm sawn from reinforced concrete shells of the Tallinn hydroplane hangar;
2. cylinders drilled out of a diameter of 45 mm, samples L =12 mm sawn from the reinforced concrete frame of the Helsinki Olympic Stadium;
3. seven samples from the depth of 0-150 mm of reinforced concerete column of the Tallinn hydroplane hangar. A sample of 70 mm in diameter, *L*=150 mm was drilled out of the column, then sawn into samples of 20 mm in thickness, a total of 7 samples, from the outer surface towards the centre L1- L7;
4. seven samples from the depth of 0-150 mm of the reinforced concrete column of the Papisaare hydroplane hangar. A sample of 70 mm in diameter, *L*=150 mm was drilled out of the column, then sawn into samples 20 mm in thickness, a total of 7 samples were studied from the outer surface of the column towards the centre P1 – P7.

Water absorption of the samples p. 1, 2 was determined according to the German norm DIN 52103. The procedure was as follows:

1. drying in the thermostat at 105^0 C and weighing (dry weight);

2. placing to water, the outer surface of the samples being 1-2 mm above the water level;

3. visual inspection after damping (samples had wetted from the edge to the centre), re-weighting (wet weight);

4. sample placing time into water and damping time were recorded, accordingly, the mean water absorption time was determined.in hours.

Water absorption of samples p. 3, 4 was determined by a non-standard testing method, proceeding from a modified Finnish standard SFS 5513 [39], the procedure being as follows:

1. the sample was broken;

2. a water drop was spilled by a pipet on the cement stone fracture surface and the absorption time, the speed of water absorption in seconds, was measured by a stop watch.

Results

- **Concrete samples of the reinforced concrete shells of the Tallinn hydroplane hangar**

Table 5.7 shows the results of water absorption studies of concrete from hydroplane hangars.

The mean water absorption time of the hydroplane hangar shell concrete was 0.7 hours.

Table 5.7 Water absorption of concrete in reinforced concrete shells of the Tallinn hydroplane hangar

No	Wet weight, g	Dry weight, g	H_20, g	%
1.	20.00	18.69	1.31	7.0
2.	15.05	14.02	1.03	7.3
3.	16.86	15.73	1.13	7.2
4.	16.95	15.84	1.11	7.0
5.	16.57	15.60	0.97	6.2
6.	12.38	11.79	0.59	5.0
7.	15.16	13.98	1.18	8.4
8.	14.88	13.98	0.90	6.4
9.	12.30	11.61	0.69	5.9
10.	12.93	11.98	0.95	7.9
11.	13.42	12.32	1.10	8.9
12.	7.58	7.20	0.38	5.3
13.	11.50	10.74	0.74	7.1
		Water absorption, %		6.9 +/- 1.1

- **Samples of concrete of the reinforced concrete frame from the Helsinki Olympic Stadium**

Table 5.8 shows the results of water absorption studies of concrete from the reinforced concrete frame of the Helsinki Olympic Stadium.

The mean water absorption time of concrete from the Helsinki Olympic Stadium was 16 hours.

Table 5.8 Water absorption of concrete from the Helsinki Olympic Stadium

No	Wet weight, g	Dry weight, g	H_20, g	%
1.	52.30	50.09	2.21	4.4
2.	51.35	49.38	1.97	4.0
3.	48.24	46.17	2.07	4.5
4.	52.74	50.59	2.15	4.2
5.	51.52	49.10	2.42	4.9
		Water absorption, %		4,4

- **Samples of concrete from the reinforced concrete column of the Tallinn hydroplane hangar**

The mean speed of water absorption of concrete from a reinforced concrete column of the Tallinn hydroplane hangar was 2–4 seconds.

Table 5.9 shows the results of water absorption studies of concrete from a reinforced concrete column of the Tallinna hydroplane hangar.

Table 5.9 Water absorption of concrete from a reinforced concrete column of the Tallinn hydroplane hangar

Tallinn hydroplane hangar								
Test no.	1	2	3	4	5	6	7	Mean water absorption speed, sec
Sample no.	Speed of water absorption, sec							
1	3	2	2	3	3	2	2	2
2	1	1	1	1	2	1	2	1
3	2	3	2	2	2	1	1	2
4	4	5	5	3	3	3	4	4
Fine fraction from sample no. 5 increased								
5	5	4	4	3	4	4	5	4
6	3	4	4	3	3	3	4	3
7	4	3	5	3	3	3	3	3

- **Samples of concrete from a reinforced concrete column of the Papisaare hydroplane hangar**

The mean speed of water absorption of concrete from a reinforced concrete column of the Papisaare hydroplane hangar in sample P1 was 80 seconds, in sample P2 21 seconds and from sample P3 on up to P7 it was 9–2 seconds.

Table 5.10 shows study results of water absorption speed for the Papisaare hydroplane hangar.

Table 5.10 Water absorption of concrete from a reinforced concrete column of the Saaremaa hydroplane hangar

Saaremaa hydroplane hangar								
Test no.	1	2	3	4	5	6	7	Mean water absorption speed, sec
Sample no.	Speed of water absorption, sec							
1	45	96	85	82	94	79	81	80
2	26	28	20	13	26	23	13	21
3	11	10	8	9	10	13	5	9
4	9	6	4	5	9	5	6	6
5	3	3	6	6	4	3	6	4
6	1	2	1	4	4	4	2	3
7	2	1	2	2	1	2	1	2

5.4.8 Mechanical properties and chemical composition of reinforced steel

Studies of chemical composition of reinforced steel and tensile tests were conducted according to the standard EVS-EN1002-1 and methods of Spectro guidelines.

Study results are presented in Tables 5.11 and 5.12.

Table 5.11 Mechanical properties of reinforced steel in Tallinn and Papisaare hydroplane hangars

Papisaare hydroplane hangar

Sample			Cross section area S, mm^2	Max load P, N	Tensile strength R_m, N/mm2	Load on yielding point P_{02}, N	Yielding point R_{el}, N/mm^2	Relative elongation by fracture A, %
Mark	Dimensions, mm							
	Diameter							
N1	14		154	4500	292	2950	191	11

Tallinn hydroplane hangar

Sample			Cross section area S, mm^2	Max load P, N	Tensile strength R_m, N/mm2	Load on yielding point P_{02}, N	Yielding point R_{el}, N/mm^2	Relative elongation by fracture A, %
Mark	Dimensions, mm							
	Diameter							
N1	18		254	8300	326	7000	275	7

Table 5.12 Chemical composition of reinfoced steel in Tallinn and Papisaare hydroplane hangars

Tallinn hydroplane hangar

diam. 18mm, number of samples 3

C	Si	Mn	P	S	Cr	Mo	Ni	Al	Co	Cu
%	%	%	%	%	%	%	%	%	%	%
0.03	<0.01	0.13	0.012	0.019	0.14	<0.01	0.37	<0.0010	0.11	0.04
Nb	Ti	V	W	Pb	Sn	As	Ca	Ce	Sb	Se
%	%	%	%	%	%	%	%	%	%	%
<0.00	<0.0010	<0.00	<0.01	<0.002	0.002	0.011	0.0002	<0.002	0.001	<0.0010
Te	B	Zn	Fe	N						
%	%	%	%	[cnt]						
<0.001	<0.0003	<0.0010	>99.12	5771						

Papisaare hydroplane hangar

diam. 14mm, number of samples 3

C	Si	Mn	P	S	Cr	Mo	Ni	Al	Co	Cu
%	%	%	%	%	%	%	%	%	%	%
0.16	<0.01	0.60	0.014	0.066	0.02	<0.01	0.02	<0.0010	<0.01	<0.01
Nb	Ti	V	W	Pb	Sn	As	Ca	Ce	Sb	Se
%	%	%	%	%	%	%	%	%	%	%
<0.00	<0.0010	<0.00	<0.01	<0.002	<0.001	0.003	0.0003	<0.002	<0.001	<0.0010
Te	B	Zn	Fe	N						
%	%	%	%	[cnt]						
<0.001	<0.0003	<0.0010	>99.09	3560						

On the basis of the studies, a conclusion can be made that the properties of reinforcement steel bars comply with the requirements set in original design conditions.

Mechanical properties and chemical composition of the reinforced steel of the Tallinn and hydroplane hangar is comparable to the corresponding parameters of the contemporary reinforcement steel Fe 360B.

A low content of carbon (C) is characteristic of the Papisaare as well of the Tallinn hydroplane hangars.

5.4.9 Summary

Materials studies both in the Tallinn and Papisaare hydroplane hangars may be summarized as follows:

- The cubic compressive strength of main girders, beams and columns of Tallinn and Papisaare hydroplane hangars exceeded the lower value of compressive strength of 15 N/mm^{2,} as required in the original design conditions, the actual compressive strength of different elements ranging from 12.9 – 28 N/mm^2.
- Water absorption of concrete from reinforced concrete shells of the Tallinn hydroplane hangar is higher by 2.5 % and the mean speed of water absorption is 32 times higher than that of concrete from the Helsinki Olympic Stadium.
- The mean speed of water absorption of concrete from a reinforced concrete column of the Tallinn hydroplane hangar was 2–4 seconds, that is an indication of high concrete porosity.
- Mean speeds of water absorption found for concrete from a reinforced concrete columns of the Papisaare hydroplane hangars vary from 80 up to 2 seconds from the outer surface of the column towards the centre. The speed of water absorption in samples P1, P2 (i.e. from the outer surface of the column into a depth of 40 mm) was low, from sample P3 on the speed of water absorption was increasing.
- Frost resistance of concrete samples from the Tallinn hydroplane hangars was 110 and 150 cycles, respectively. For a comparison, according to the standard EVS 814:2003, F150 is comparable to frost resistance class KK3 and F110 to class KK2 (the highest required class being KK4).
- Concrete from the Papisaare hydroplane hangars complies with the requirements of frost resistance set for class KK3 $S_{56}<0.20$ kg/m^2.
- According to the results of X-ray and chemical analysis, concrete from reinforced concrete shells of the Tallinn hydroplane hangar was found carbonized across the whole cross-section, concrete from reinforced concrete columns was found carbonized up to 20 mm from the outer surface
- On the basis of X-ray and chemical analysis concrete from a reinforced concrete column of the Papisaare hydroplane hangar was found carbonized up to 40 mm from the outer surface of the column towards the centre.
- The photos taken by an electron microscope confirm the results of water absorption, frost resistance as well as the results of X ray analysis concerning concrete from reinforced concrete shells and columns as follows: cement stone is of relatively high porosity; it is characterized by high content of amorphous crystals.
- Photos taken by an electron microscope confirm the results of water absorption, frost resistance and X-ray analysis of concrete from columns of the Papisaare hydroplane hangar as follows: the outer surface of the cement stone is of high density, higher porosity occurs on the inner surface; crystals of the binder are distorted by shape.

- Strength properties of reinforced steel comply with the terms set in the design requirements.

5.5 Summary and conclusion

Based on the structural condition assessment and the materials analysis of the Tallinn and Papisaare hydroplane hangars, the following conclusions can be drawn:

- **Reinforced concrete structures of Tallinn hydroplane hangars:**
  - The reason of early damage formation and the results of concrete from reinforced concrete shells is that the shell structures have no acting roof covering, as a result, corrosion of cement stone in concrete has occurred. As a result of corrosion, cement stone had decomposed, concrete porosity had increased, carbonation had speed up considerably, and thus an extensive corrosion of the reinforcment and loss of protective layer of concrete on all the structural elements had occurred.
  - As a result of concrete porosity, carbonation of cement stone has occurred across the whole extent of the sample (shell cross-section) and therefore no protective (alkaline) environment is present to stop corrosion in progress in the reinforced steel; the corrosion speed of the reinforced steel depends in the main on the access of O_2 and moisture (water) open to it.
  - Because of high porosity of shell concrete, its components (cement stone and aggregate material) will be moisturized rapidly in contact with water and that leads to ever increasing decomposition of reinforced steel and cement stone.
  - Concrete from reinforced concrete columns was carbonated across the outer surface area of 20 mm, as a result, corrosion of reinforced steel occurred and the reinforcement lost its protective layer. In addition, concrete is characterized by high water absorption and frost resistance. However, the compressive strength of concrete exceeds that required in the original design conditions.
  - Along with the decomposition of cement stone minerals, contact between the aggregate and the binder material has weakened and shell concrete strength has decreased. In spite of that, according to the calculations of strength made, concrete compressive strength, properties of reinforcement steel bars are satisfactory to ensure restoration works and prolong service life of hydroplane hangars.
  - Urgent action should be taken to start restoration works of the hydroplane hangars in order to avoid rapid further corrosion of the reinforced steel, measures should be taken to restrict access of water to the reinforced steel bars – install a roof covering to the shells and a water outflow system and cover the lower shell surface with a layer of spray concrete.

- The compressive strength of concrete and the properties of reinforced steel bars are adequate to ensure restoration works and a prolong service life of hydroplane hangar structures.

- **Reinforced concrete structures of Papisaare hydroplane hangars:**
  - Based on the studies carried out, a conclusion can be drawn that concrete from reinforced concrete structures of Papisaare hydroplane hangars is carbonated across cross-sections in the area of 40 mm from the outer surface. That has led to extensive corrosion of the reinforced steel and loss of the protective layer on all structural elements.
  - Water absorption of the outer surface of concrete is slow, porosity is higher and frost resistance has increased. It was caused by the technology of concrete manufacture, the content of aggregate material and the structure of higher density resulting from the carbonation process, cement stone pores of concrete are filled with $CaCO_3$ crystals the volume of which is larger than that of the replacable $Ca(OH)_2$ (this process was also confirmed by the X-ray analysis).
  - Compressive strength of concrete exceeds the minimum value required in the design conditions.
  - Compressive strength of concrete and the properties of the reinforced steel bars are adequate to ensure restoration works and a prolong service life of hydroplane hangar structures.

6 ASSESSMENT OF RESIDUAL CARRYING CAPACITY: CASE STUDIES

6.1 Introduction

Prior to erecting hangar structures no complicated calculations in terms of today's shell theories, neither those of complex shells, were made. These shells were probably designed according to simpler calculation methods [31] based on the membrane theory. Unfortunately the statics and strength calculations of hangars have not preserved.
In this thesis the method of finite elements according to the linear-elastic calculation scheme in compliance with the STAAD/Pro and Robot Millenium programs was used to determine the stress and strain state of the shell roof.

6.2 The stress and strain state calculations and analysis

6.2.1 Introduction to the verification of the carrying capacity

The carrying capacity of reinforced concrete structures in the hydroplane hangars in Tallinn and Papisaare was verified in terms of the persistent design situation of the ultimate limiting state of the carrying capacity according to the European standard EN 1990:2002 p.6.4.2 3 [50]. The limiting states of breaking for the cross-section, the structural member or the joint were taken into account to verify that:

$$E_d \leq R_d$$

6.2.2 Actions and combinations of actions

General actions - self-weight, densities, snow and wind loads and the classification of loads were determined according to the European Standards EN 1991-1-1:2002; EN 1991-1-3:2003; EN 1991-1-4:2005 [51; 52; 53].

Design values of actions were found according to the EN 1990:2002 [50], Appendix NA, Table NA.1.2 (B) as follows:

- permanent actions $\gamma_{Gj,\,sup} = 1.20$

  $\gamma_{Gj,\,inf} = 1.00$

- variable actions $\gamma_{Q,1} = 1.50$

  $\gamma_{Q,i} = 1.50$

Calculation results concerning the ultimate limiting state of the carrying capacity of the action combination of the persistent design situation were verified according to the European Standard EN 1990:2002, p. 6.4.3.2 [50]:

- design value of the dominant variable actions

- values of the combination of non-dominant variable actions:

$$E_d = \gamma_{sd}E(\gamma_{g,j}G_{k,j};\gamma_p P;\ \gamma_{Q,1}\ Q_{k,1};\ \gamma_{Q,i}\ \psi_{0,i}Q_{k,i})\ \ j \ \geq 1\ ;\ i > 1$$

6.2.3 The stress and strain state calculations using FEM according Staad/Pro and Robot Millenium

Shell roofs and the reinforced steel of the Tallinn and Papisaare hangars were divided into finite elements in the geometry of the calculation scheme made in the AutoCAD program. In the programs Staad/Pro, Robot Millennium the stress and strain state was determined according to the Finite Element Method (FEM), the linear elastic calculation scheme (Figures 6.1, 6.2).

The calculations were made taking into account the effect of the calculation loads and load combinations described in Section 6.2.2.

The elasticity module of reinforced concrete taken was 25 KN/mm^2 and the Poisson´s ratio was 0.17.

6.2.4 Results of calculation

- Tallinna hydroplane hangar

Resulting from the large number and high volume of internal forces and stress diagrams of the shell roof found in the determination of the stress and strain state, as an example, the values of normal stresses and bending moments are presented here as diagrams in Figures 6.3 and 6.4.

Maximal calculated internal forces and stresses:

1) carrying columns and –angle braces, in the angle brace in the axis D-1-2, Figure 5.10,

normal force N_{sd} = 7200 kN.

2) in area A of the shell roof, Figure 6.5.

normal stresses f = 1.61 N/mm^2

normal force $N_{,sd}$ = 161 kN/m

bending moment $M_{,sd}$ = 1.23 kNm/m

- Papisaare hydroplane hangar

Maximal calculated internal forces and stresses:

1) carrying columns, column no. 9, Figure 5.6

normal force $N_{sd} = 760$ kN

2) in main beams, main beam no. 2, Figure 5.6

bending moment $M_{sd} = 850$ kNm

6.2.5 Summary

As a result of the calculations of internal forces, stresses and strains in Tallinn hydroplane hangars and comparisons in the case of different actions and action combinations it was found that the bending moments, normal, shear stresses and principal stresses of the majority of elements are relatively small. According to the calculation results the wind load does not have a significant effect on the work of hydroplane hangars. Structural carrying capacity was verified based on the calculation results from the action combination of self-weight and snow load.

The internal forces and stresses in the framework of Papisaare hangar were as expected. Structural carrying capacity was verified based on the calculation results from the load combination of self-weight, snow and wind load.

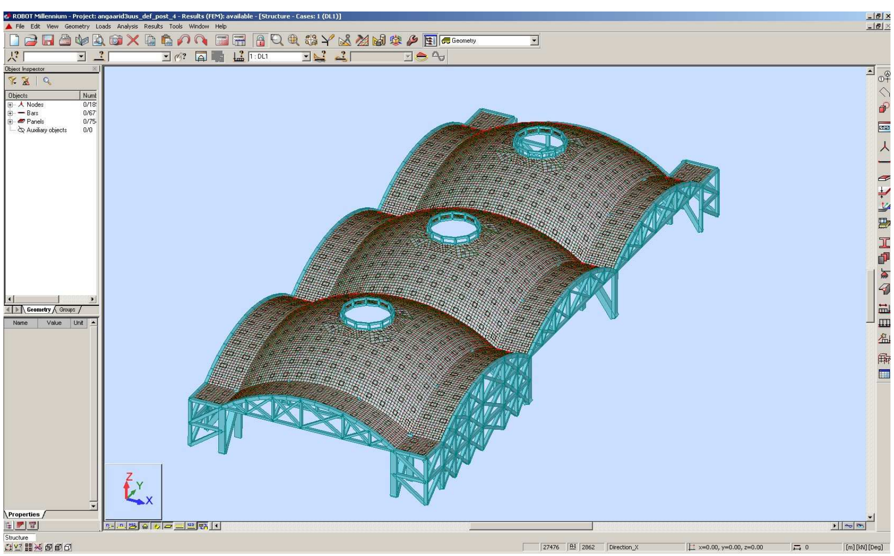


Figure 6.1 Calculation scheme of Tallinn hydroplane hangars in the Robot Millennium program

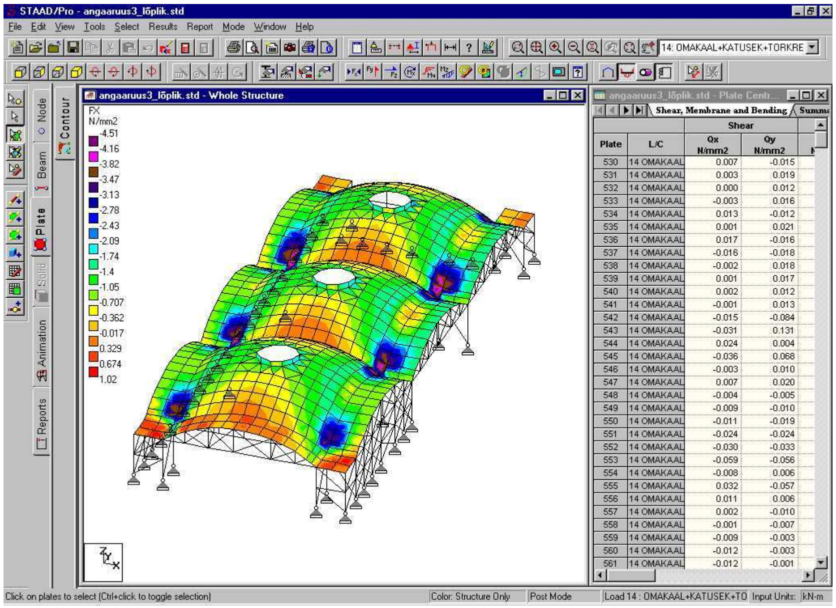


Plate	L/C	Qx N/mm2	Qy N/mm2
530	14 OMAKAAL	0.007	-0.015
531	14 OMAKAAL	0.003	0.019
532	14 OMAKAAL	0.000	0.012
533	14 OMAKAAL	-0.003	0.016
534	14 OMAKAAL	0.013	-0.012
535	14 OMAKAAL	0.001	0.021
536	14 OMAKAAL	0.017	-0.016
537	14 OMAKAAL	-0.016	-0.018
538	14 OMAKAAL	-0.002	0.018
539	14 OMAKAAL	0.001	0.017
540	14 OMAKAAL	0.002	0.012
541	14 OMAKAAL	-0.001	0.013
542	14 OMAKAAL	-0.015	-0.084
543	14 OMAKAAL	-0.031	0.131
544	14 OMAKAAL	0.024	0.004
545	14 OMAKAAL	-0.036	0.068
546	14 OMAKAAL	-0.003	0.010
547	14 OMAKAAL	0.007	0.020
548	14 OMAKAAL	-0.004	-0.005
549	14 OMAKAAL	-0.009	-0.010
550	14 OMAKAAL	-0.011	-0.019
551	14 OMAKAAL	-0.024	-0.024
552	14 OMAKAAL	-0.030	-0.033
553	14 OMAKAAL	-0.059	-0.056
554	14 OMAKAAL	-0.008	0.006
555	14 OMAKAAL	0.032	-0.057
556	14 OMAKAAL	0.011	0.006
557	14 OMAKAAL	0.002	-0.010
558	14 OMAKAAL	-0.001	-0.007
559	14 OMAKAAL	-0.009	-0.003
560	14 OMAKAAL	-0.012	-0.003
561	14 OMAKAAL	-0.012	-0.001

Figure 6.2 Calculation scheme of Tallinn hydroplane hangars in the Staad/Pro program

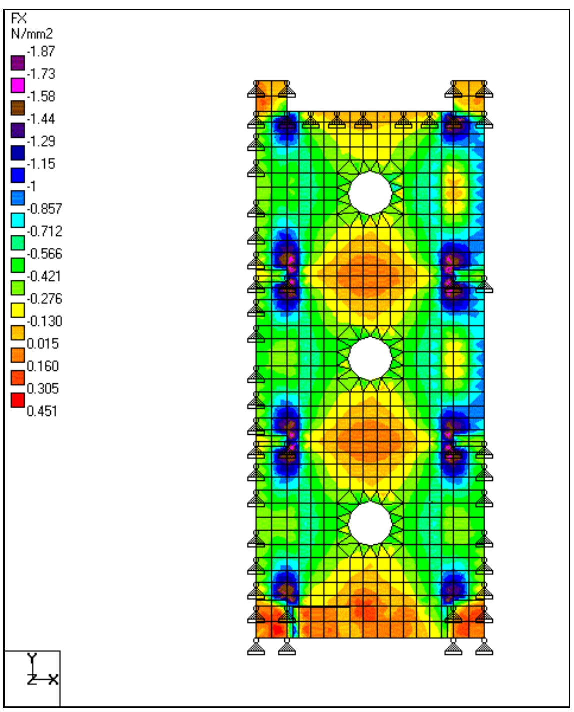


Figure 6.3 Diagram of the normal stresses f_x

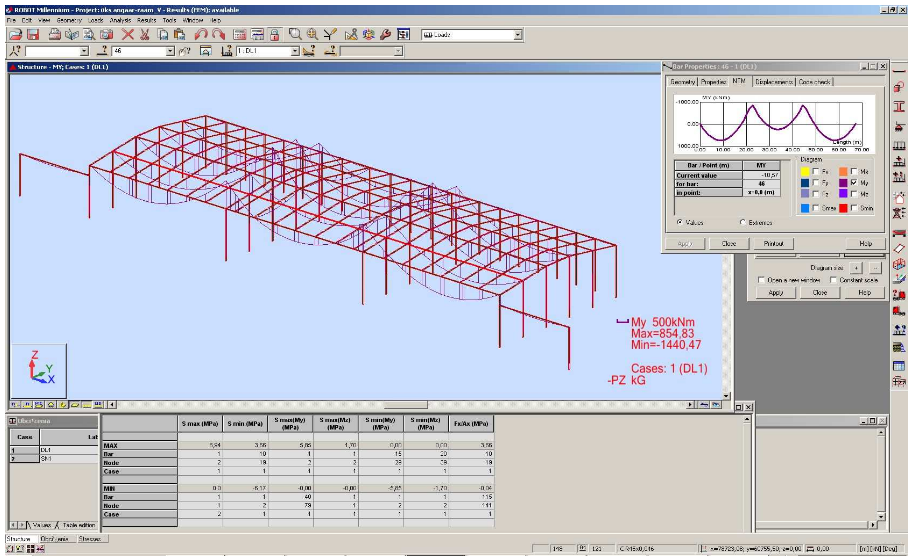


Figure 6.4 Calculation scheme of Papisaare hydroplane hangars in the Robot Millennium program

6.3 Verification of the residual carrying capacity

The residual capacity of both the Tallinn and Papisaare hydroplane hangars was verified according to the European Standards EN 1990:2002 [50]; EN 1992-1-1:2007 [54] requirements of verifying the carrying capacity provided that the geometry of reinforced concrete structures has preserved as in the original design without any damage: reinforced steel has preserved its protective layer, the bond between the steel bars and the concrete has been preserved and the cross-section of the reinforced steel bars has preserved its original dimension. In the damaged zones and elements of reinforced concrete structures studied (reinforced steel bars has lost its protective layer, it has corroded) it is not possible to verify the carrying capacity according to the norms established, according to the norms the residual carrying capacity is not ensured.

Source data used to verify the calculations of the residual carrying capacity were the results obtained from materials research (see Section 5) of the properties of reinforced concrete and reinforced steel.

According to literature [32] and the results of the study show that the spacing of both the lower and upper reinforcement steel bars of the shell roof of Tallinn hydroplane hangars is ~ 200 x 200 mm. The samples drilled out show that the diameter of the steel bars is 8-10 mm.

- **Tallinn hydroplane hangar**

1) The carrying capacity of the reinforced concrete shell area A (Figure 6.5) was verified as an eccentrically stressed cross-section.

Data:

Cross-sectional dimensions (Figure 6.7): b= 1000 mm, h=100 mm, d_1= 77 mm, d_2= 23 mm; A_c= 10 *10^3 mm^2

Reinforcement: f_{yk} = 275 MPa; γ_s = 1.15; $f_{yd} = f_{yk}/\gamma_s$ = 275/1.15 = 239 MPa

$f_{yd} = f_{ycd}$ = 239 MPa; reinforcement 6Ø8 ($A_{s1} = A_{s2}$ = 301.4 mm^2);

Concrete: (C8/10) $f_{ck,cube}$= 12.8 MPa; γ_c=1.50; $f_{cd} = f_{ck}/\gamma_c$=12,8/1.5 = 8.5 MPa;

Calculated internal forces and stresses in the cross-section:

normal stresses f = 1.61 N/mm^2

normal force $N_{,sd}$ = 161 kN/m

bending moment $M_{,sd}$ = 1.23 kNm/m

Calculated carrying capacity:

$(Ne)_{Rd} = \alpha f_{cd} by(d_1 - 0.5y) + \delta_{s2} A_{s2}(d_1 - d_2)$ = 7.31 kNm

The moment of the longitudinal force caused by the load in relation to the axis through the centre of gravity of reinforcement A_{s1} is

$(Ne)_{Sd} = M_{sd} + N_{sd}(d_1 - 0.5h)$ = 2.61 kNm < $(Ne)_{Rd}$= 7.31 kNm

Thus, the carrying capacity of the cross-section is ensured.

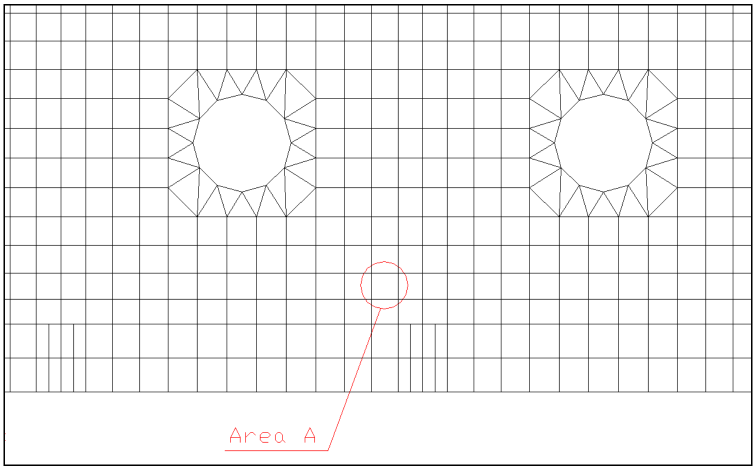


Figure 6.5 Reinforced concrete shell roof – area A

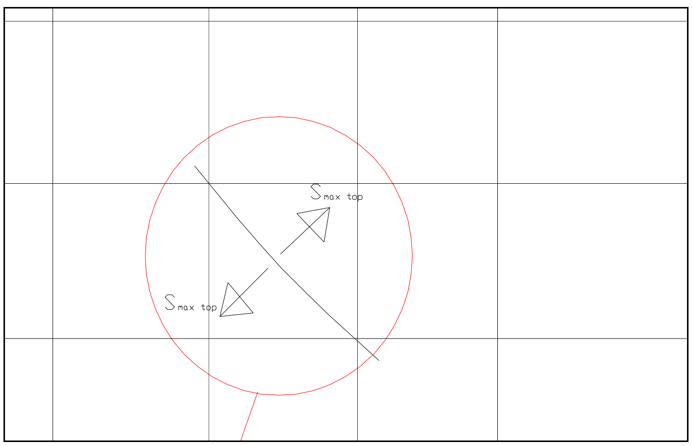


Figure 6.6 Reinforced concrete shell roof – area A

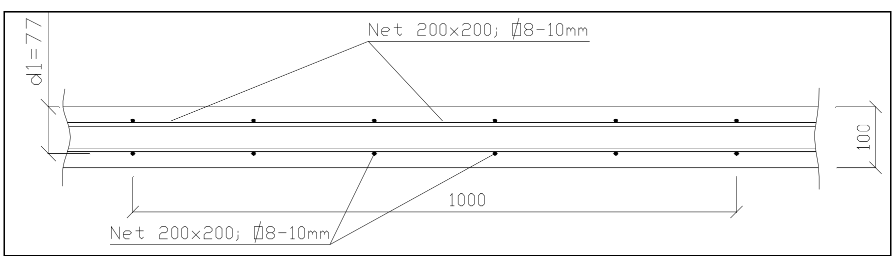


Figure 6.7 Section of the reinforced concrete shell roof

2) Verification of the carrying capacity of the cross-section of reinforced concrete shell area A for principal stresses

According to literature [32] and this study the diagonal reinforcement required to receive principal tensile stresses is absent on the edges of the shell roof. Therefore the principal tensile stresses were divided into force components and the cross-section was verified as drawn eccentrically.
Strength verification for principal stresses was conducted in area A (Figure 6.5) by dividing the principal tensile stresses $S_{max\ top}$ into components $S_{max,\ komp}$ (Figure 6.6).

Data:

Cross-sectional dimensions (Figure 6.7): b = 1000 mm, h = 100 mm, d_1= 77 mm, d_2= 23 mm; $A_c = 10 * 10^3\ mm^2$;

Reinforcement: f_{yk}= 275 MPa; γ_s=1.15; $f_{yd} = f_{yk}/\gamma_s$ = 275/1.15 = 239 MPa

$f_{yd} = f_{ycd}$= 239 MPa; reinforcement 6∅10 ($A_{s1} = A_{s2}$ =471 mm^2);

Concrete: (C8/10) $f_{ck,cube}$= 12.8 MPa, γ_c=1.50; $f_{cd} = f_{ck}/\gamma_c$=12.8/1.5= 8.5 MPa;

Calculated internal forces and stresses in the cross-section:

Max principal stress S_{max}= 3.06 N/mm^2,

Component of the principal stress $S_{max;\ komp} = S_{max} * \sin 45^0$=2.16 N/mm^2,

Component force of the principal stress:

$N_{Sd} = S_{max;\ komp} * A$ = 216 kN/m

Bending moment $M_{,sd}$ = 1.23 kNm/m

Calculated carrying capacity:

$(Ne')_{Rd} = f_{yd}A_{s1}(d_1 - d_2)$ = 6.07 kNm

$(Ne)_{Rd} = f_{yd}A_{s2}(d_1 - d_2)$ = 6.07 kNm

The moment caused by the external forces:

$(Ne)_{Sd} = N_{Sd} * e$ = 5.81 kNm < $(Ne)_{Rd}$ = 6.07 kNm

$(Ne')_{Sd} = N_{Sd} * e$ = 5.83 kNm < $(Ne')_{Rd}$ = 6.07 kNm

Thus, the carrying capacity of the cross-section is ensured.

3) Verification of the carrying capacity of reinforced concrete angle brace no. 1

The carrying capacity of reinforced concrete angle brace no 1 (see Figure 5.10, axis D-1-2) was verified by taking angle brace as a centrically stressed column in a fixed structure. The cross-section was verified as an eccentrically stressed cross-section and the general eccentricity was found as a sum of the eccentricity that caused the residual and the second order eccentricity ($e_{tot} = e_a + e_2 \geq 20$ mm).

Data:

Cross-sectional dimensions: $b = h$= 1300 mm, d_1= 1250 mm, d_2= 50 mm;

$A_c = 1690 * 10^3$ mm^2;

Reinforcement: f_{yk}= 275 MPa; $\gamma_s = 1.15$; $f_{yd} = f_{yk}/\gamma_s$ = 275/1.15 = 239 MPa

$f_{yd} = f_{ycd}$ = 239 MPa; reinforcement 7∅25 ($A_{s1} = A_{s2}$= 3456 mm^2);

Concrete: (C20/25) $f_{ck,cube}$= 28.0 MPa, γ_c=1.50; $f_{cd} = f_{ck}/\gamma_c$ = 28.0/1.5 = 18.6 MPa;

Calculated internal forces in the cross-section:

normal force N_{sd}= 7200 kN

Structure imperfection ν=1/400

Calculation length of the angle brace $l_0 = \beta * l_{col}$= 0.7 *12.1= 8.47m

β = factor depending on the type of attachment of the column and its location, the angle brace inspected had an joint on one end and a fixed connection on the other in the structure thus $\beta \geq 0.7$.

l_{col} = actual length of the angle brace (distance between the end nodes).

Calculated carrying capacity:

$(Ne)_{Rd} = \alpha f_{cd} by(d_1 - 0.5y) + f_{ycd} A_{s2}(d_1 - d_2) = 14259$ kNm

Eccentricity of the longitudinal force in relation to the centre of gravity of the tensile reinforcement:

$e = e_0 + d_1 - 0{,}5h = 0.665$ m

$(Ne)_{sd} = N_{sd} * e = 4788$ kNm $< (Ne)_{Rd} = 14259$ kNm

Thus, the carrying capacity of the cross-section is ensured.

- **Papisaare hydroplane hangar**

1) Verification of the carrying capacity of the cross-section of main beam no. 2 (Figure 5.6)

Data:

Cross-sectional dimensions: b= 450 mm, h=1400 mm, d_1= 1350 mm, d_2= 50 mm;

Reinforcement: f_{yk} = 191 MPa; γ_s=1.15; $f_{yd} = f_{yk}/\gamma_s$= 191/1.15 = 166 MPa

$f_{yd} = f_{ycd}$ = 166 MPa; reinforcement 4∅30 ($A_{s1} = A_{s2}$= 2826 mm^2);

Concrete: (C16/20) $f_{ck,cube}$= 16 MPa; γ_c=1.50; $f_{cd} = f_{ck}/\gamma_c$=16.0/1.5= 10.7 MPa;

Calculated internal forces in the cross-section:

bending moment $M_{,sd}$= 850 kNm

Calculated carrying capacity:

$M_{rd} = a\, f_{cd} by\, (d_1 - 0.5y) + f_{ycd} A_{s2}(d_1 - d_2) = 1509$ kNm

Thus, the carrying capacity of the cross-section is ensured.

2) Verification of the carrying capacity of reinforced concrete column no. 9 (see Figure 5.6)

The carrying capacity of reinforced concrete column (see Figure 5.6) was verified by taking the column as a centrically stressed column in a fixed structure. The cross-section was verified as an excentrically stressed cross-section and the general eccentricity was found as a sum of the excentricity that caused the residual and the second order eccentricity ($e_{tot} = e_a + e_2 \geq 20$mm).

Data:

Cross-sectional dimensions: $b = h$= 450 mm, d_1= 400 mm, d_2= 50 mm;

Reinforcement: f_{yk}=191 MPa, γ_s=1.15; $f_{yd}= f_{yk}/\gamma_s$=191/1.15= 166 MPa

$f_{yd} = f_{ycd}$= 166 MPa; reinforcement 4∅30 ($A_{s1}= A_{s2}$= 2826 mm^2);

Concrete: (C16/20) $f_{ck,cube}$= 16.0 MPa, γ_c=1.50; $f_{cd} = f_{ck}/\gamma_c$ = 16,0/1.5 = 10.7 MPa;

Calculated internal forces in the cross-section:

normal force N_{sd} = 760 kN

Calculated carrying capacity:

$(Ne)_{Rd} = \alpha f_{cd} by(d_1 - 0.5y) + f_{ycd}A_{s2}(d_1 - d_2)$ =1050 kNm

Eccentricity of the longitudinal force in relation to the centre of gravity of the tensile reinforcement:

$e = e_0 + d_1 - 0{,}5h = 0.065 + 0.40 - 0.225 = 0.240$ m ,

$(Ne)_{sd} = N_{sd} * e = 760*0.240$ = 182 kNm < $(Ne)_{Rd}$= 1050 kNm

Thus, the carrying capacity of the cross-section is ensured.

6.3.1 Summary

Verification of the residual carrying capacity of both, Tallinn and Papisaare hydroplane hangars, was conducted provided that the geometry of the reinforced concrete structures has preserved as in original design without any damage – the reinforced steel has preserved its original state with no damage – the reinforced steel has not lost its protective layer, the bond between the reinforcement and the concrete has been preserved, and the cross-section of the reinforced steel bars has preserved its original diameter.

In this study the verification of the carrying capacity was conducted according to the conditions described above. In the elements and zones studied, the carrying capacity of the reinforced concrete structures in the Tallinn and Papisaare hydroplane hangars is ensured and the residual carrying capacity is sufficient to do the restoration works.

In the elements and zones in which the preconditions above are not satisfied, a pre-failure state exists and the residual carrying capacity is not ensured.

6.4 Summary and conclusions

On the basis of the stress and strain state and the analysis of strength calculations it can be concluded that the bending moments, normal, shear and principal stresses of the majority of elements of the Tallinn and Papisaare hydroplane hangars are relatively small. For these reasons, in spite of extensive damage neither local nor overall collapse has taken place. At the same time it should be stressed that verification of actual strength in each smaller area is almost impossible since in spite of everything the source data are not accurate and the whole of the remaining reinforcement, the strength of concrete and the working cross-section are variable. Thus, only a general assessment can be provided based on the data of some zones.

As a result of the verification of the carrying capacity of the Tallinna and Papisaare hydroplane hangars it can be concluded that in the elements and zones where the geometry of reinforced concrete structures has preserved as in the original design, without any damage, the carrying capacity of the structure or its elements is ensured and the residual carrying capacity is still sufficient to do the restoration works. In the damaged areas a pre-failure state has occurred, the residual carrying capacity of the structure is not ensured and immediate strengthening in these zones is required.

7 CASE STUDY - RESTORATION OF THE HYDROPLANE HANGARS IN TALLINN

7.1 Introduction

So far it has not been possible to perform a complete restoration of the hydroplane hangars. In the autumn of 2001, the restoration process started with the middle shell, because that shell was in the worst condition, but only inevitable repairs were carried out then. Taking into account the financial possibilities it was decided to repair the area settled down by stitching cracks, gluing the cracks with epoxy resin and by installing a roof coverage.

Damaged concrete was cleaned, renovation works were prepared, materials and methods to be used were selected according to the description in Section 2.3 and literature reports in [5; 6; 7; 9; 22; 23; 40-49].

7.2 Structural solutions of restoration works and their application on the site

- **Reinforcement of deformed areas**

The area A of the spherical shell, which was settled down and shifted (Figure 5.10), was reinforced with a rebars of B500K Ø 3 mm and Ø 5 mm, with a spacing of 100 mm (Figure 7.2). Reinforcement bars-anchor were fixed to canals (Figure 7.2, 7.7), which were milled from one side and fixed from the other side to drilled holes with a glue mixture. Then the shell, previously from old damaged roof coverage, was covered with bond varnish and monolithed with special mortar (Figures 7.1 – 7.7). In the same way - the monolith®thing of the crack-hole of the joint cylindrical and spherical shell (area B in Figure 5.10) was reinforced.

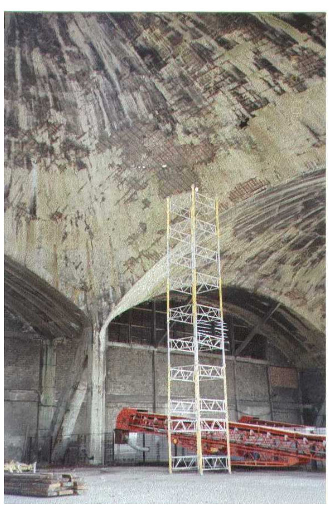


Figure 7.1 Supports to the deformed area of the spherical shell

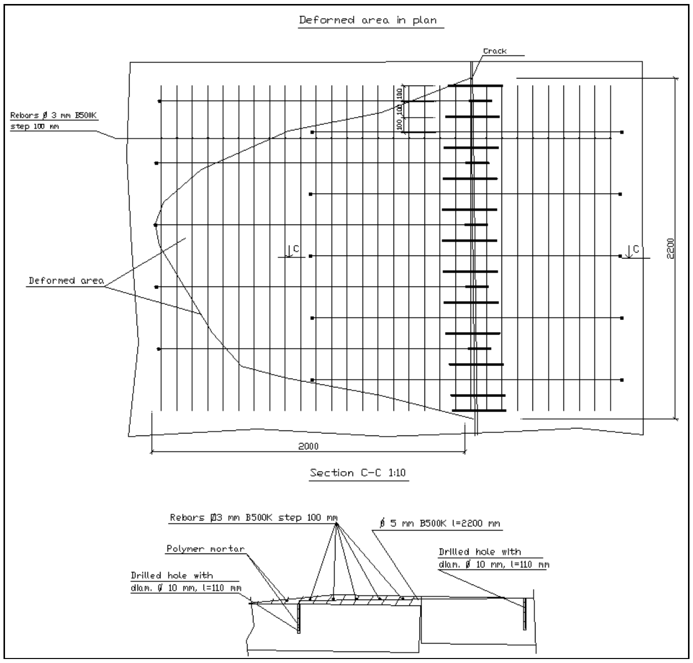


Figure 7.2 Structural solution – deformed area A

Figure 7.3 Reinforcement bars-anchor fixed to the canal by glue mixture in the deformed spherical shell area

Figure 7.4 Reinforced and monolithic area of deformation in the spherical shell

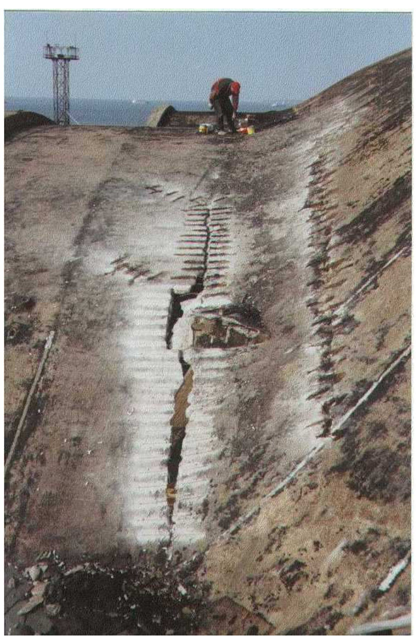


Figure 7.5 Reinforcement of the interface area of the spherical and cylindrical shell

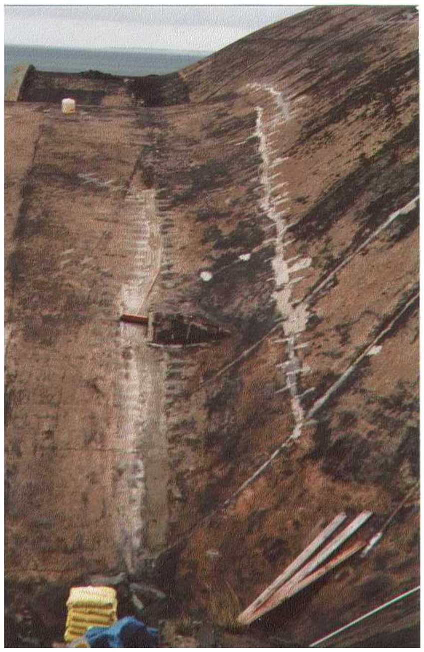


Figure 7.6 Monolithicizing of the interface of the spherical and cylindrical shell

- **Stiching and injecting the cracks**

Before stitching and injection, the cracks were cleaned and treated with a biological repellent. Then the cracks were covered with bond varnish. Reinforcement bars-anchors of Ø 5 mm were fixed to canals with a spacing of 250 mm. Reinforcement bars-anchors_were milled on one side and fixed on other side to the drilled holes with a glue mixture. Then the cracks were injected with epoxy resin (Figures 7.7 – 7.11).

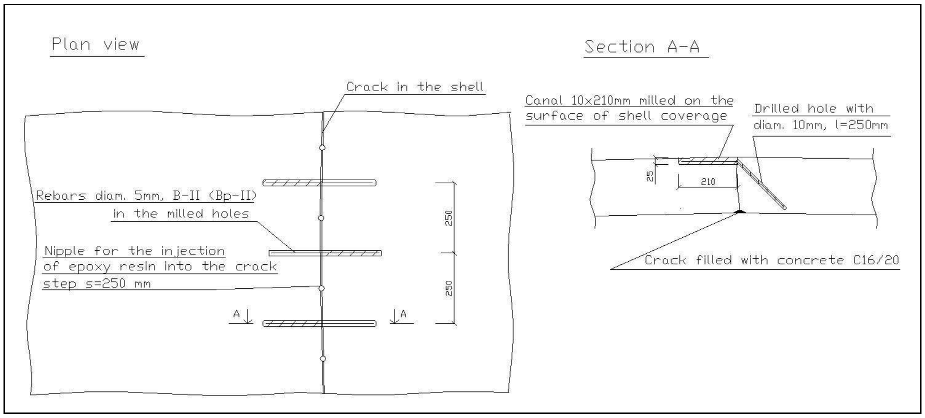


Figure 7.7 Reinforcement and injection of cracks

Figure 7.8 Canal milling to the shell surface

Figure 7.9 Cleaning of cracks in the shell before injection of epoxyl resin

Figure 7.10 Crack injection by epoxyl resin

Figure 7.11 Cracks stitched and injected by epoxyl resin

- **Installation of reinforcement on the lower shell surface**

Protective layer spalling and plaster are to be removed from the inner surface, according to the methods described below in the concrete spray works. Next, an additional reinforcement net of Ø 3 mm with a step of 150 x 150 mm, with dowels of Ø 4 mm, a step of 750 x 750 mm shot in is installed.

Because of possible electro-chemical corrosion between carbon-steel reinforcement in the shell coverage and the new reinforced steel to be installed, it is recommended [28; 29; 33; 34] to use a net from stainless steel number EN1.4301 or EN1.4401 as a new reinforcement on condition that its their relative total area is smaller than that of the carbon-steel reinforcement. Because of the dangers described above, in any case use of carbon-fiber (grafite) as additional reinforcement must be excluded.

The sprayed concrete layer should be sprayed on the shell ground layer according to the technology reported in [5; 6; 7; 9; 22; 23; 40-49].

- **Restoration of reinforced concrete columns, angle braces, tensile beams and girders**

Protective layer spalling and plaster must be removed from the reinforced concrete elements according to the gunning method described below. If the cross-section of reinforcement of the reinforced elements has decreased over 20 % as a result of corrosion, additional reinforcement is to be installed or the corroded reinforcing rods are to be replaced. Each of the corrosion cases should be considered separately and a relevant solution should be proposed by the designer in the work process.

- **Installing the roof cover**

Before covering the shells with the SBS type roof covering, the old bitumen was scratched out and the fogged areas were treated with a biological repellent (Figure 7.12). The cleaned shells were covered with bond varnish. The new SBS type roof cover had two layers: it was armed with polyester fiber and covered with bulk. The roof covering was fixed additionally from edges with rivets (spacing 0.5 m) in the parts of shell, which had larger incline and window lighting surroundings (Figures 7.13, 7.14). Additionally, the water drain system was repaired and new pipes were installed. The edge elements of the cylindrical shells were rimmed with sheet metal.

Figure 7.12 Cleaning of shell cover from old roof coverage

Figure 7.13 Installation of roof cover

Figure 7.14 Shell coverage with SBS roof covering materials

7.3 Summary and conclusions

All of the restauration solutions selected and worked out by us were verified in reality in the site and can be used in further works in the future.

8 CONCLUSIONS AND RECOMMENDATIONS FOR FURTHER WORK

8.1 CONCLUSIONS

The aims of the work were achieved:

Reasons of inadequate durability of reinforced concrete were studied in the zone of old damaged reinforced concrete structures and an analysis of their estimation and study is presented. The object of studies was the structural conditions of reinforced concrete material and structures on the example of the historic Tallinn and Papisaare hydroplane hangars as well as an assessments of residual carrying capacity. Taking into consideration the results of the studies above, an analysis of possible restoration solutions for the supporting structures of Tallinn hydroplane hangars was made and the applicability of the selected restoration solutions on the site is described.

Statements:

- Before diagnosing the causes of deterioration or failure of a concrete structure, a sound understanding of the physical, chemical and mechanical actions that lead to defects is necessary.
- Methods for studies of the structural condition and residual carrying capacity should be composed separate for each structure, taking into account the properties of the materials, calculation methods, designed exploitation purposes applicable in the period of building, extent of damage and a possible effect on the carrying capacity of the structure.
- Reinforced concrete durability is influenced by continual maintenance level and compliance of the conditions of concrete exploitation and designed environment.
- The main impact factor of reinforced concrete as a composite material is air, gases and water (vapour) permeability in the outer surface open to the environment. Permeability of the outer surface open to the environment, in turn, is influenced by the following factors: structure of concrete stone, properties of constituents, strength, and concrete preparation technology.
- Reinforced concrete carbonation, however, has some positive effects: outer surface of concrete is strengthened, permeability of the outer surface is decreased, moisture transport in the cement stone is reduced and resistance to those attacks which are controlled by permeability (incl.carbonation speed etc.) is increased.
- In terms of the carrying capacity of reinforced concrete structures, the geometry of structures, as well as physical-mechanical properties of the constituents of reinforced concrete as a composite material are the factors that the durability of structures depends on.

- At the beginning of the 20th century, sufficiently good knowledge of how roof shell structures work had been acquired. That was confirmed by calculations made by help of the LEM method used today, proceeding from the actual dimensions of our protypes, other parameters and structures.

- It is obvious that the engineer who had designed Tallinn hydroplane hangars had made his calculations in view of substantial reserve, since in spite of considerable damage of concrete and reinforcement, the shell structure with its favourable geometry is still prevailing.

- In the analysis of shell structures (incl domes) a century ago and earlier, an analysis method based on the membrane theory was widely used, allowing for the studies of the behaviour of these structures. At the same time, it should be noted that the theory did not take into account, satisfactorily the influence of the bending moments and cutting forces (in terms of principal stresses) in the area of edge members. Thus, for instance, edge zones of the hydroplane hangar spherical shells studied in this work were not adequately reinforced to receive the tension caused by cutting forces. At the same time, no fractures caused by the forces mentioned were observed, that resulting from the fact that a substantially higher shell thickness than in other zones was found there.

- Only after the cause or causes are known can rational decisions be made concerning the selection of a proper method of repair and in determination of how to avoid a repetition of the circumstances that led to the problem.

- Structural restoration should take into consideration the following: preservation of the geometry of the structure according to that designed, physical-mechanical properties of reinforced concrete (strength, porosity, properties of steel), and the extent of damage.

- On the basis of our studies it can be stated and should be emphasized that it is of prime importance to protect such kind of historic structures direct from rain, as over the time water will dissolve constituents of cement stone from concrete, resulting in increased concrete porosity, reduced compressive strength, accelerated carbonation, intensive corrosion of reinforced steel. To sum up, the carrying capacity of the whole structure will thus be decreased.

- In spite of previous inadequate maintenance and extensive damages formed in the cases considered as well as in many other similar instances relating to reinforced concrete structures from the 1990s, it is possible to extend their service life (use) for decades, at the same time essentially preserving their architectural appearance.

8.2 RECOMMENDATIONS FOR FURTHER WORK

- To have more conformity between the suggested and used restoration solutions described, after their realization it is necessary to examine their reliability in exploitation conditions.
- In the future the behaviour of long-span reinforced concrete structures in the absence of thermal joints should be studied (effect of shrinkage etc.).
- It is necessary to elaborate more exact methods for the assessment of damaged reinforced concrete structures.

9 KOKKUVÕTE

Töö eesmärgid on saavutatud:

On uuritud ning analüüsitud raudbetooni puuduliku kestvuse põhjuseid vanade kahjustustega raudbetookonstruktsioonide valdkonnas ning esitatud nende hindamise ja uurimise vastav analüüs. Teostatud uurimisobjektiks valitud ajalooliste Tallinna ja Papisaare vesilennukite angaaride näitel raudbetooni materjali- ning konstruktsiooni tehnilise seisundi uuringud koos jääkkandevõime hindamisega. Arvestades eelpool kirjeldatud uuringutest saadud tulemusi on teostatud Tallinna vesilennukite angaaride kandekonstruktsioonide võimalike restaureerimise lahenduste analüüs ning kirjeldatud valitud restaureerimislahenduste rakendatavust objektil.

Järeldused:

- Enne raudbetoonkonstruktsioonide kahjustuste hindamist on vajalik omada ülevaadet füüsilistest, keemilistest ning mehaanilistest mõjuritest, mis põhjustavad defekte.
- Tehnilise seisundi ja jääkandevõime uuringu metoodika tuleb koostada igale objektile eraldi, arvestades ehituse ehitamise ajajärgul kasutusel olnud materjalide omadusi, arvutusmetoodikat, projekteeritud kasutusotstarvet, kahjustuste ulatust ning võimalikku mõju konstruktsiooni kandevõimele.
- Raudbetooni kestvust mõjutab konstruktsiooni järjepideva hooldamise tase ning ekspluateerimisel betooni kasutus- ja projekteeritud keskkonnatingimustele vastavus.
- Raudbetooni, kui komposiitmaterjali kestvuse peamiseks mõjufaktoriks on betooni keskkonnale avatud välispinna läbitavus õhule, gaasidele ja veele (veeaurule). Seejuures betooni keskkonnale avatud välispinna läbitavust mõjutavad järgnevad faktorid: betoonkivi struktuur, koostisosade omadused, tugevus ning betooni valmistamise tehnoloogia.
- Raudbetooni karboniseerumisel on ka postiivsed tagajärjed: suureneb betooni välispinna tugevus, väheneb betooni välispinna läbitavus, väheneb niiskuse liikumine betoonkivis ning suureneb vastupidavus kahjustustele, mis on mõjutatavad betooni läbitavusega (näiteks karboniseerumise kiirus jm.).
- Raudbetoonkonstruktsioonide kandevõime osas on olulisel kohal konstruktsioonide geomeetria, samuti raudbetooni, kui komposiitmaterjali koostisosade füüsikalis-mehaanikalistel omadustel, millest sõltub ka konstruktsioonide kestvus.
- 19.-nda sajandi lõpul ja 20.-nda sajandi algul oli piisavalt hea ettekujutus katuse koorikkonstruktsioonide tööst, mida kinnitasid arvutused

tänapäeval kasutatava LEM meetodi abil, lähtudes meie prototüübi tegelikest mõõtmetest, muudest parameetritest ja konstruktsioonist.

- Ilmselt arvestas Tallinna vesilennukite angaarid projekteerinud insener arvutuste teostamisel märkimisväärse varuga, sest vaatamata betooni ja sarruse olulistele kahjustustele püsib soodsa geomeetriaga koorikkonstruktsiooni veel püsti.
- Sajand ja vanemate koorikkonstruktsioonide (sh kuplite) kasutati laialdaselt membraanteooriale rajatud analüüsi meetodit, mis kajastas üsna hästi nende konstruktsioonide tööd. Samas tuleb märkida, et see teooria ei arvestanud vajalikul määral ääreliikmete piirkonnas paindemomentide ja lõikejõudude (peapingete osas) mõjuga. Nii näiteks ei ole ka meie poolt uuritud vesilennukite angaaride sfäärilisete koorikute nurkade piirkonnad vajalikult armeeritud lõikejõududest tekkivate tõmbejõudude vastuvõtmiseks. Samas ei ole märgata nimetatud jõududest tekkinud pragusid, mis tuleneb sellest, et nendes piirkondades on ka kooriku paksus oluliselt suurem, kui muus osas.
- Alles peale kahjustuste põhjuste välja selgitamist on võimalik valida sobiv restaureerimismeetod ning otsustada, kuidas vältida kahjustuste kordumist tulevikus.
- Konstruktsioonide restaureerimislahenduste valikul tuleb arvestada: konstruktsiooni projektikohase geomeetria säilitamisega, raudbetooni füüsikali-mehaaniliste omadustega (tugevus, poorsus, terase omadused), kahjustuste ulatusega.
- Meie uuringute alusel võib väita ja tuleb rõhutada, et esmane ülesanne seda tüüpi ajalooliste konstruktsioonide puhul on kaitsta neid otsese vihma eest, kuna see vesi lahustab aja jooksul tsementkivi koostisosi betoonist välja, mille tulemusena suureneb betooni poorsus, väheneb survetugevus, kiireneb karboniseerumine, intensiivistub terasarmatuuri korrodeerumine. Kokkuvõttes alaneb kogu konstruktsiooni kandevõime.
- Vaatamata eelnenud puudulikule hooldusele ning tekkinud ulatuslikele kahjustustele on meil vaadeldud juhtumitel, aga ka paljudel muudel taoliselt juhtumitel, kus tegemist ajaloolist väärtust omavate 90 aastaste raudbetoonkonstruktsioonidega, saavutada veel suhteliselt mõistlike vahenditega selle eluea (kasutuse) pikendamine aastakümneteks, seejuures tema arhitektuurilist välisilmet oluliselt muutmata.

10 BIBLIOGRAPHY

1. Neville A. M. Properties of Concrete. Fourth Edition. Essex, UK: Pearson Education Limited, 2005. 844 p.
2. Hewlett Peter C. Lea's Chemistry of Cement and Concrete. Fourth Edition.Oxford, UK: Butterworth-Heinemann, 2001. 1057p.
3. Son L. H., Yuen G.C.S. Building Maintenance Technology. London: The Macmillan Press, 1993. 357p.
4. Taylor H. F. W. Cement Chemistry. London: Telford, 1997. 459p.
5. Dobrowolski J. A. Concrete Construction, Handbook. Fourth Edition. New York: McGraw- Hill, 1998.
6. Warner J., Murray M. A., Pfeifer W. D., etc. Concrete Repair. Volume 3. Illions, USA: The Aberdeen Group, 1993. 47p.
7. Gaughen D. C., Brandon J. H. Overview of industry guidance on concrete condition assessment and concrete repair for use prior to coating. California, USA: Naval Facilities Engineering Service Center, 2000. 15p.
8. Keil F. Zement: Herstellung und Eigenschaften (in German). Berlin: Springer ,1971. 439p.
9. U.S. Army Corps of Engineers. Evaluation and Repair of Concrete Structures: EM 1110-2-2002. Washington DC: Department of The Army, U.S. Army Corps of Engineers, 1995. 185p.
10. Raado L. Corrosion Protection of Construction (in Estonian). Tallinn: Tallinn University of Technology, 2002. 94p.
11. Raupach M. Concrete repair according to the New European Standard EN 1504 in M. Alexander, H.D. Beushausen, F. Dehn and P. Moyo (eds) Concrete Repair, Rehabilitation and Retrofitting. London: Taylor &Francis Group, 2006. pp. 6-8.
12. Malhorta V.M. Durability of Concrete, Chapter 26,
Uhlig's Corrosion Handbook Second Edition, R.W. Revie Ed., Wiley, 2000.
13. Rahman A. A. Characterization of the porosity of hydrated cement pastes in The Chemistry and Chemically-Related Properties of Concrete, Ed. F. P. Glasser, British Ceramic Proceedings No. 35, pp. 249 – 263 (Stoke-on-Trent, 1984).
14. Larbi L. A. Microstructure of the interfacial zone around aggregate particles in concrete. Journal Heron, Vol. 38, No.1, 69pp. Delft University of Technology, 1993.

15. Papadakis V. G., Vayenas C.G. and Fardis M. N. Physical and chemical characteristics affecting the durability of concrete. ACI Materials Journal, Vol. 88, No.2, pp. 186-196 (1991).

16. Sakuta M. Measures to restrain rate of carbonation in concrete, in Concrete Durability, Vol. 2, ACI SP-100, pp. 1963-1977. Detroit, Michigan, 1987.

17. Parrot L. J. Review of Carbonation in Reinforced Concrete. Slough, UK: Cement and Concrete Assn., 1987. 42p.

18. Broomfield J. P. Corrosion of Steel in Concrete: Understanding, Investigation and Repair. London: E&FN Spon, 1998. 239p.

19. Wiering H. J. Longtime studies on the carbonation of concrete under normal outdoor exposure. *In:* RILEM Symposium on Durability of Concrete under Normal Outdoor Exposure, Hannover, 1984. pp. 182-196.

20. Concrete Society Report. Non-structural Craks in Concrete. Technical Report No. 22. 3rd Edition. London: Concrete Society, 1992. 48pp.

21. Reis E.E, Mozer J. D., Bianchini A. C., Kesler C. E. Causes and control of cracking in concrete reinforced with high-strength steel bars – a review of research. Illions: university of Illions Engineering Experiment Station Bull. No 479.

22. European Standard ENV 1504-9 1997-07-00. Products and systems for the protection and repair of concrete structures - Definitions, requirements, quality control and evaluation of conformity - Part 9: General principles for the use of products and systems.

23. REHABCON IPS-2000-00063. Strategy for Maintenance and

    Rehabilitation in Concrete Structure. DG Enterprise of the European Commission, 2000-2004.

24. Ratay R. T. Structural Condition Assessment. American Society of Civil Engineers. New Jersey, USA: John Wiley &Sons, 2005. 685p.

25. Berke N.S., Hicks M. C., Malone J., Rieder K.A. Concrete Durability. Journal Concrete International, 08/2005. pp. 63 -68.

26. Feldmann G.C. Non-destructive Testing of Reinforced Concrete. Structure magazine, 01/2008. pp. 13-17.

27. Rusalep E. X-Ray analysis (in Estonian). Tallinn: Tallinn University of Technology, 1996. 199p.

28. Tunturi P. I. Korroosio käsikirja. Hanko, Finland: Suomen Korrosiooyhdistus, 1988.

29. Laren M. The Avesta Welding Manual, steels and stainless steel welding. Stockholm: Avesta Welding AB, 2004.

30. Laul H. Reinforced concrete I (in Estonian). Tallinn: Tallinn University of Technology, 1962. 576p.

31. Laul H. Reinforced concrete II (in Estonian). Tallinn: Tallinn University of Technology, 1962. 575p.

32. Kaljundi J. Historical overview: The Hydroplane Hangars in Tallinn. Tallinn: Deparment of Cultural Heritage, 1979.

33. Mattila J. S., Pentti M. J.. 1996. The performance of the differente realkalisation methods. *In:* Proceedings of the International Conference on Concrete Repair, Rehabilitation and Protection, 27 – 28 June 1996, University of Dundee, Scotland, UK. London: E&FN Spon, p 483 – 491.

34. Mattila J. S., Pentti M. J. and Raiski T.A. Durabitliy of electrochemically realkalised concrete structures. *In*: Proceedings of the Fourth International Symposium on Corrosion of Reinforcement in Concrete Construction, 1-4 July, Robinson College, Cambridge, UK. Cambridge: The Royal Society of Chemistry, p 481 – 490.

35. Kulu P. Euro steels (in Estonian). Tallinn: Tallinn University of Technology, 2001.

36. Encyclopædia Britannica/Calcite. http://en.wikisource.org/wiki/1911_Encyclop%C3%A6dia_Britannica/Calcite, 01.09.2008.

37. Portlandite mineral data. http://www.webmineral.com/data/Portlandite.shtml 01.09.2008

38. Ettringite mineral data. http://www.webmineral.com/data/Ettringite.shtml, 01.09.2008

39. Suomen Standardisoimisliito. Standardi SFS 5513. Muurauslaastinen, Muurauskivinen ja muuratun rakenteen testaus.

40. European Standard ENV 1504-9 1997-07-00. Products and systems for the protection and repair of concrete structures - Definitions, requirements, quality control and evaluation of conformity - Part 9: General principles for the use of products and systems.

41. European Standard EN 1504-1:2007. Products and systems for the protection and repair of concrete structures - Definitions, requirements, quality control and evaluation of conformity - Part 1: Definitions.

42. European Standard EN 1504-2:2007. Products and systems for the protection and repair of concrete structures - Definitions, requirements, quality control and evaluation of conformity - Part 2: Surface protection systems for concrete.

43. European Standard EN 1504-3:2005. Products and systems for the protection and repair of concrete structures - Definitions, requirements, quality control and evaluation of conformity - Part 3: Structural and non-structural repair.

44. European Standard EN 1504-4:2004. Products and systems for the protection and repair of concrete structures - Definitions, requirements, quality control and evaluation of conformity - Part 4: Structural bonding.

45. European Standard EN 1504-5:2004. Products and systems for the protection and repair of concrete structures - Definitions, requirements, quality control and evaluation of conformity - Part 5: Concrete injection.

46. European Standard EN 1504-6:2006. Products and systems for the protection and repair of concrete structures - Definitions, requirements, quality control and evaluation of conformity - Part 6: Anchoring of reinforcing steel bar.

47. European Standard EN 1504-7:2006. Products and systems for the protection and repair of concrete structures - Definitions, requirements, quality control and evaluation of conformity - Part 7: Reinforcement corrosion protection.

48. European Standard EN 1504-8:2004. Products and systems for the protection and repair of concrete structures - Definitions, requirements, quality control and evaluation of conformity - Part 8: Quality control and evaluation of conformity.

49. European Standard EN 1504-10:2004. Products and systems for the protection and repair of concrete structures - Definitions, requirements, quality control and evaluation of conformity - Part 10: Site application of products and systems and quality control of the works.

50. European Standard EN 1990:2002. Eurocode – Basis of structural design.

51. European Standard EN 1991-1-1:2002. Eurocode 1: Actions on structures – Part 1-1: General actions – Densities, self-weight, imposed loads for buildings

52. European Standard EN 1991-1-1-3:2003. Eurocode 1: Actions on structures – Part 1-3: General actions – Snow loads.

53. European Standard EN 1991-1-4:2005. Eurocode 1: Actions on structures – Part 1-4: General actions – Wind actions.

54. European Standard EN 1992-1-1:2004+AC:2007. Eurocode 2: Design of concrete structures – Part 1-1: General rules and rules for buildings.

11 APPENDIX I PUBLICATIONS

H. Onton (2007). Estimation of residual carrying capacity and restoration of the historic reinforced concrete shells and frames erected in Estonia. *In*: Structural Studies, Repairs and Maintenance of Heritage Architecture X: Tenth International Conference on Studies, Repairs and Maintenance of Heritage Architecture STREMAH X, Prague, Czech. Wessex Institute of Technology, UK, WIT Press, Volume 95, 407 – 417.

H. Onton, K. Õiger (2006). Damages and their causes of an old concrete of the hydroplane hangars in Tallinn. *In*: proceedings of ESCS 2006: European Symposium on Service Life and Serviceability of Concrete Structures ESCS-2006, Espoo, Finland, 312 – 318.

H. Onton, K. Õiger (2005). Restoration of an Old Concrete Construction of the Hydroplane Hangars in Tallinn. *In*: Proceedings of TICCIH 2005: TICCIH Intermediate Conference 2005 & International Forum for Industrial Tourism in Nagoya, Japan, 112 – 120.

K. Oiger, H.Onton (2004). Restoration of an old concrete construction: the hydroplane hangar in Tallinn. *In*: Proceedings of 6th European Commision Conference on Sustaining Europe's Cultural Heritage: From Research to Policy in London, United Kingdom. University College London, UK, p. 26. Publication of Conference Proceedings electronically: www.ucl.ac.uk/sustainableheritage/ec-conference.

12 APPENDIX II CURRICULUM VITAE

1. Personal data

Name	Heiki Onton
Date and place of birth	01.01.1978, Tallinn
Nationality	Estonian

2. Contact information

Address	Roosikrantsi 13-15, 10119, Tallinn
Phone	(+372) 56 503 394
E-mai	heiki.onton@ramirent.ee

3. Education

Educational institution	Graduation year	Education (field of study/degree)
Tallinn University of Technology	2003	Civil Engineering/ MSc
Tallinn University of Technology	2000	Civil Engineering/BA
Tallinn Pelgulinna Secondary School	1996	Secondary

4. Language competence/skills (fluent, average, basic)

Language	Level
Estonian	Fluent
English	Average
Russian	Average

5. Professional Employment

Period	Organisation	Position
2005 – do date	Ramirent AS	Vice director
2003 - 2005	Ramirent AS	Chief Designer
2001 - 2003	Ramirent AS	Designer
2000 - 2001	ETS Nord AS	Engineer

6. Defended theses

   Investigation, analysis and restoration of the hydroplane hangar reinforced shells. Master's Degree.

   Structural condition assessment of buildings. Bachelor's Degree.

7. Main areas of research work

   Investigation of the causes of deterioration of old reinforced concrete constructions. Restoration of reinforced concrete constructions.

13 APPENDIX III ELULOOKIRJELDUS

1. Isikuandmed

Ees- ja perekonnanimi	Heiki Onton
Sünniaeg ja -koht	01.01.1978, Tallinn
Kodakondsus	Eesti

2. Kontaktandmed

Aadress	Roosikrantsi 13-15, 10119, Tallinn
Telefon	(+372) 56 503 394
E-post aadress	heiki.onton@ramirent.ee

3. Hariduskäik

Õppeasutus	Lõpetamise aeg	Haridus (eriala/kraad)
Tallinna Tehnikaülikool	2003	tehnikateaduste magister
Tallinna Tehnikaülikool	2000	tehnikateaduste bakalaureus
Tallinna Pelgulinna Keskkool	1996	keskharidus

4. Keelteoskus (alg-, kesk-, kõrgtase)

Keel	Tase
Eesti	emakeel
Inglise	kesktase
Vene	kesktase

5. Teenistuskäik

Töötamise aeg	Tööandja nimetus	Ametikoht
2005 - tänaseni	Ramirent AS	Direktori asetäitja
2003 - 2005	Ramirent AS	Juhtiv projekteerija
2001 - 2003	Ramirent AS	Projekteerija
2000 - 2001	ETS Nord AS	Insener

6. Kaitstud lõputööd

Vesilennukite angaari raudbetoonkoorikute töö uurimine, analüüs ja restaureerimine. Magistrikraad

Hoonete tehnilise seisundi hindamine. Bakalaureuse kraad.

7. Teadustöö põhisuunad

Vanade raudbetoonkonstruktsioonide kahjustuste põhjuste uurimine.
Raudbetoonkonstruktsioonide restaureerimine.