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Durability Properties and Actual Deterioration of Finnish Concrete Facades and Balconies

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ABSTRACT

Finnish multi-storey residential buildings have been built of precast concrete panels since the 1960's. Half of these buildings, which for the most part are located in suburbs, were built in the fairly short period 1960-1979. The durability properties and repair needs of existing concrete facades and balconies are key factors contributing to the technical performance of the built environment and the owners' economical decisions.

The general objective of this research was to study the factors that have actually had an impact on the service life, occurrence and progress of deterioration in existing concrete facades and balconies.

The research was based on condition investigation data from existing concrete buildings and measured weather data. The research material consisted of a database on the material properties and deterioration of existing Finnish concrete facade panels and balconies built between 1960 and 1996, and weather observations since 1961 made by the Finnish Meteorological Institute (FMI).

According to the research results, the durability properties of concrete structures are generally quite inadequate both as concerns reinforcement corrosion and frost resistance of concrete. Facade panels generally have only few areas where reinforcement cover depths are small, but they are significant in terms of the economy of cementitious patch repairs. Exposed aggregate, clinker tile and unpainted form finish concrete facades and balcony side panels have the poorest frost resistance. The poor frost resistance of balcony side panels may be considered a significant factor that limits the service life of the entire stock of precast concrete balconies since the side panels are load-bearing structures which cannot be replaced without demolishing the entire balcony structure. The durability properties of concrete structures have improved, especially since the 1980's, as more attention has been paid to the durability of concrete structures e.g. by preparing durability guidelines.

The research results indicate that despite the generally quite poor durability properties of concrete facades and balconies, the concrete structures have suffered remarkably little far advanced and extensive corrosion and frost damage. The corrosion damage is almost entirely due to the carbonation of concrete. As concrete carbonises, corrosion occurs first in reinforcements closest to the outer surface. Local frost damage in facade elements typically occurs in the upper corners of buildings and at the edges of panels. Local frost damage in balconies typically takes place in the upper sections and especially the front edges of side panels. Visible frost damage clearly correlates with the frost resistance of concrete. The facade surface types and balcony elements that have small areas of inadequate frost resistance naturally suffer less frost damage.

The research results show that the stress on concrete structures from rain and sleet has a crucial effect on the initiation and propagation speed of damage. The amount of rain and sleet and prevailing wind directions during them are thus a quite clear cause of quicker deterioration in the coastal area than inland, and for the lesser corrosion and frost damage of concrete on northern to eastern facades of buildings compared to the southern to western facades.

Keywords: concrete, condition investigation, deterioration, carbonation, reinforcement corrosion, frost resistance, frost damage, outdoor climate

FOREWORD

This research has been conducted in the Department of Structural Engineering at Tampere University of Technology during the years 2006-2011. It is continuation of a long series of research projects concerning the repair of concrete facades and balconies, which has been one of the key research areas of this organisation for more than 20 years.

This dissertation is based on extensive material from condition investigations of existing precast multi-storey buildings and related analyses. The key research areas have been the realised durability properties of concrete structures and the actual deterioration of structures under natural conditions. The results of this research allow developing service-life models for the existing already damaged building stock to better prepare for upcoming repair needs, both technically and economically, and to allow optimal scheduling of necessary repairs with respect to the service life of an existing structure.

Scientific research is always more or less team work. It is important for a researcher to be surrounded by people of high professional competence to develop things with and test ideas on. That has also been true with this research, and I take this opportunity to thank them all.

I wish to thank Prof. Matti Pentti, the supervisor of my work, for his patience in allowing me to finalise this dissertation in the middle of the organisational change of our department.

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Tampere, March 8th, 2012

Jukka Lahdensivu

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TERMINOLOGY

Capillarity
: Capillarity refers to the property of a porous material to transfer liquid water by capillary suction pressure.

Carbonation-induced corrosion
: Corrosion which is initiated by carbonation (neutralisation) of the concrete cover of reinforcement.

Carbonation rate
: Rate of pH drop resulting from carbon dioxide-calcium hydroxide reaction, generally [mm/a$^{0.5}$].

Chloride-induced corrosion
: Corrosion initiated by presence of a critical amount of chlorides.

Condition investigation
: Systematic inspection of a structure's condition and performance with respect to different deterioration phenomena by reviewing design documents, visual examination of the structure, field measurements including sampling, and laboratory analyses of samples.

Corrosion
: Degradation of metal due to its electrochemical dissolution into electrolyte.

Corrosion rate
: Degree of speed of metal loss due to electrochemical dissolution expressed either as corrosion-current density, usually [µA/cm^2] or as material loss per time unit, usually [µm/a].

Freeze-thaw cycle
: Falling of the temperature of a material below 0 °C to a given temperature and rising back above 0 °C. In this research the temperature limit of the freeze-thaw cycle was -2 °C, -5 °C and -10 °C.

Frost damage
: Failure of concrete's internal structure due to freezing pressure of water in its pore system. Recurring freezing and thawing may result in total loss of material strength and its weathering.

Frost resistance
: Capacity of hardened concrete to retain its properties during recurring freezing and thawing.

Protective pore ratio
: Share of all pores not filled with capillary water of total concrete porosity. The calculation of protective pore ratio is described in standard SFS 4475 (1988).

Service life of structure
: The period of time after installation during which a facility or its component parts meet or exceed the performance requirements (ISO 15686-1 2011).

Thermal transmittance

Thermal transmittance, U, indicates the heat volume that under a steady-state passes in a time unit through a material layer or an assembly one surface unit in size, when the temperature difference between the atmospheres on opposite sides of the material layer or assembly is one unit.

Thin-section analysis

Examination of the microstructure of concrete using an optical microscope. A translucent slide about 25 µm thick is made of the concrete sample for the examination. Deterioration analyses are made according to standard ASTM C 856 (2011).

Water-absorbing capacity

Volume of capillaries of a porous material as a share of total porosity, generally [wt%].

NOTATION

h_l	thickness of slab
k	carbonation coefficient
n	number of samples or measurements
p_r	protective pore ratio
p_w	degree of capillary saturation
t	time
x	carbonation depth
F	Faraday’s constant
I	electric current
L	span
ΔW	weight loss
W_m	molecular weight
w%	gravimetric percentage
Z	valence of corroding metal

Abbreviations

AAR	Alkali-aggregate reaction
B500K	Cold-formed indented reinforcing wire with nominal yield point of 500 MPa
BFS	Blast furnace slag
C20/25	Concrete with compressive strength of 25 MPa
FMI	Finnish Meteorological Institute
LW	Light weight (concrete)
OPC	Ordinary Portland Cement
PFA	Pulverised fuel ash
pH	Acidity of aqueous solution in material
RH	Relative humidity
TUT	Tampere University of Technology
XF 1, 3	Stress class, frost stress

1 INTRODUCTION

1.1 Background

The growth of European suburban areas was fast in the 1960’s and 70’s. Migration from the countryside into towns and changes in social structure created demand for fast and massive housing production. Large suburbs were built which changed the former pre-war townscape remarkably.

Fig. 1.1 Hervanta suburban area in Tampere was built mostly in the 1970’s and early 80’s.

Due to the massive need for residential buildings and the rapid development of prefabrication techniques of precast concrete panels in the 1960's and 70's, concrete soon became the dominant material of facades and balconies in multi-storey residential and office buildings in Finland (Mäkiö et al. 1994).

Since the 1960’s a total of about 44 million square metres of precast concrete panel facades have been built in Finland as well as 900 000 precast concrete balconies (Vainio et al. 2005). As a matter of fact, more than 60% of the Finnish building stock has been built in the 1960’s or later (Statistics Finland 2010). Compared to the rest of Europe, the Finnish building stock is quite young.

Despite the relatively young Finnish building stock incorporating precast facades and balconies, the repair need of these structures is nevertheless relatively high (Pentti et al.1998). There are several reasons for this, which should be considered from the viewpoint of the construction and exposure of these structures. The facts related to the structures presented in the following apply to the precast panel and balcony structures used since the mid 1960's until the mid 90's, which can also be regarded as the target of this research.

The service lives of existing concrete structures of precast multi-storey buildings vary widely. In some cases the facades and balconies have required, often unexpected, technically significant and costly major repairs less than 10 years after their completion. Concrete structures have been repaired extensively in Finland since the early 1990’s. During that almost 20-year period, about 10 per cent of the stock built between 1960 and 1980 has been repaired once. It is estimated that the total annual value of the building repair business in Finland is about €5 500 million, of which about 30% involves external structures (facades, balconies, roofs, windows, etc.). The total annual volume of facade renovation is about 15 million m^2. In addition, 40 000 balconies are repaired annually and 4 500 new balconies are added to old buildings. It is estimated that the volume of facade renovation will grow 2% annually (Vainio et al. 2002 and 2005).

1.2 Structural members of concrete facades and balconies

Almost all prefabricated concrete structures in Finland are based on the Concrete Element System (BES 1969). That open system defines, for instance, the recommended floor-to-floor height and the types of prefabricated panels used. In principle, the system allows using the prefabricated panels made by all manufacturers in any single multi-storey building.

1.2.1 Facades

Sandwich panels

The concrete panels used in exterior walls of multi-storey residential buildings were, and still are, chiefly prefabricated sandwich-type panels with thermal insulation placed between two concrete layers. A cross-section of a typical Finnish concrete facade panel and its connection to a floor of hollow-core slabs is presented in Fig. 1.2 (Pentti 1994).

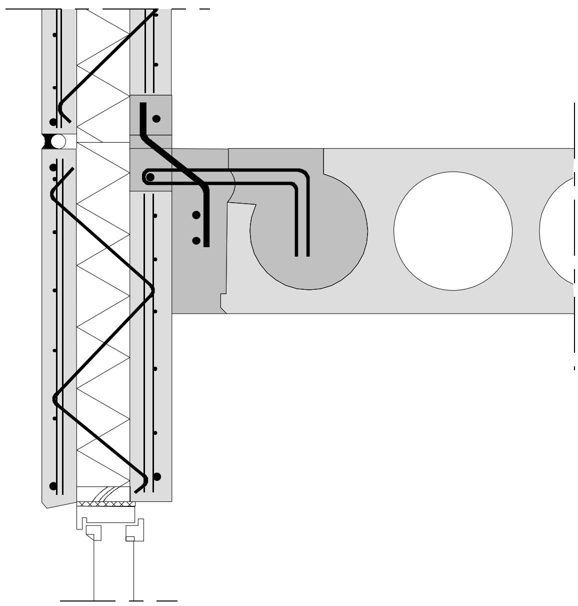


Fig. 1.2 Cross-section of a typical concrete panel facade used in Finland (Pentti 1994).

Facade panels are made up of two relatively thin reinforced concrete layers connected to each other by steel trusses. The thermal insulation between the layers is most often mineral wool of 60 to 145 mm nominal thickness depending on the building regulations in force at the time of design and construction.

The usual nominal thickness of the outer layer has been 40 to 70 mm while 50 and 60 mm are the most usual values (Pentti et al. 1998). The layers are most typically reinforced with steel mesh of a wire diameter of 3 mm and spacing of 150 mm. Rebars 6 to 8 mm in diameter are typically used as so-called edge bars and often also diagonally at the corners of windows and other major openings in the layers. The bars are spliced by lap splices, which increases the overall thickness of the reinforcement.

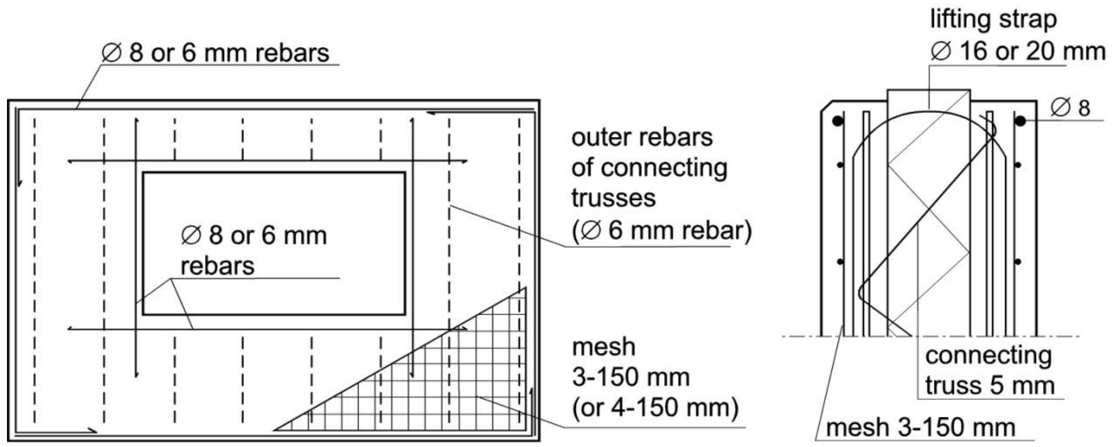


Fig. 1.3 Typical reinforcement of outer layer of a concrete facade panel (Pentti et al. 1998).

The outer layer is generally supported by the inner layer. Sandwich facade panels are connected to the building frame by the inner layer, usually by means of cast concrete joints and reinforcement ties. Panels are typically equipped with lifting straps of 16 mm steel rod for installing them in place. Lifting straps of non-bearing panels are anchored to both the inner and outer layer. After a panel is in place, the straps are to be cut to avoid cold bridges. In the case of a bearing panel, the lifting strap is anchored only to the inner layer and is used to fix the panel to the building frame.

In the Finnish prefabricated concrete building system, the inner layer of end facade panels is load-bearing while that of long facade panels is non-bearing. The outer layers of both element types always have the same dimensions and reinforcement. All vertical and horizontal joints between outer layers are elastic, made primarily with polymer sealants to allow thermal as well as other movement of the layers. It should also be noted that usually there is no ventilation gap behind the outer layers of precast exterior wall panels. Thus, if the thermal insulation gets wet e.g. due to leakage through the joints, the structure dries slowly. The drying of the outer layers is also slow because of the relatively efficient thermal insulation that limits the drying heat flow through the wall. This means that the concrete may remain moist for long periods.

Thin-shell panels

Thin-shell panels consist of a concrete panel 60 to 120 mm, the typical thickness being 80 mm. Thin-shell panels have typically been used in the end facades of concrete building and as the uppermost panels between sandwich panels and the roof.

Thin-shell panels have been connected in many different ways to the frame of a concrete building. They have been attached to a bearing reinforced concrete structure either during its casting or afterwards. In the latter case, it has been possible to leave an air gap for ventilation of the structure, which makes it unnecessary to use insulation material that can withstand casting pressure. When used as the uppermost layer with sandwich panels, shell panels have typically been attached by welding at their edges to pilasters cast in the roof cavity.

Cork, wood-wool slabs (wood-wool cement boards), LW concrete and mineral wool have been used as insulation of thin-shell panel walls. Insulation thicknesses have varied e.g. depending on insulation material and time.

1.2.2 Balconies

Stacked balcony

The most common balcony type in Finland from the late 1960’s until today consists of a floor slab, side panels and a parapet panel of precast concrete. These stacked balconies have their own foundations, and the whole stack is connected to the building frame only to brace it against horizontal loads. All structural members of a precast balcony are load-bearing. The cross-section of a typical balcony constructed of precast panels is presented in Fig. 1.4 (Pentti 1994).

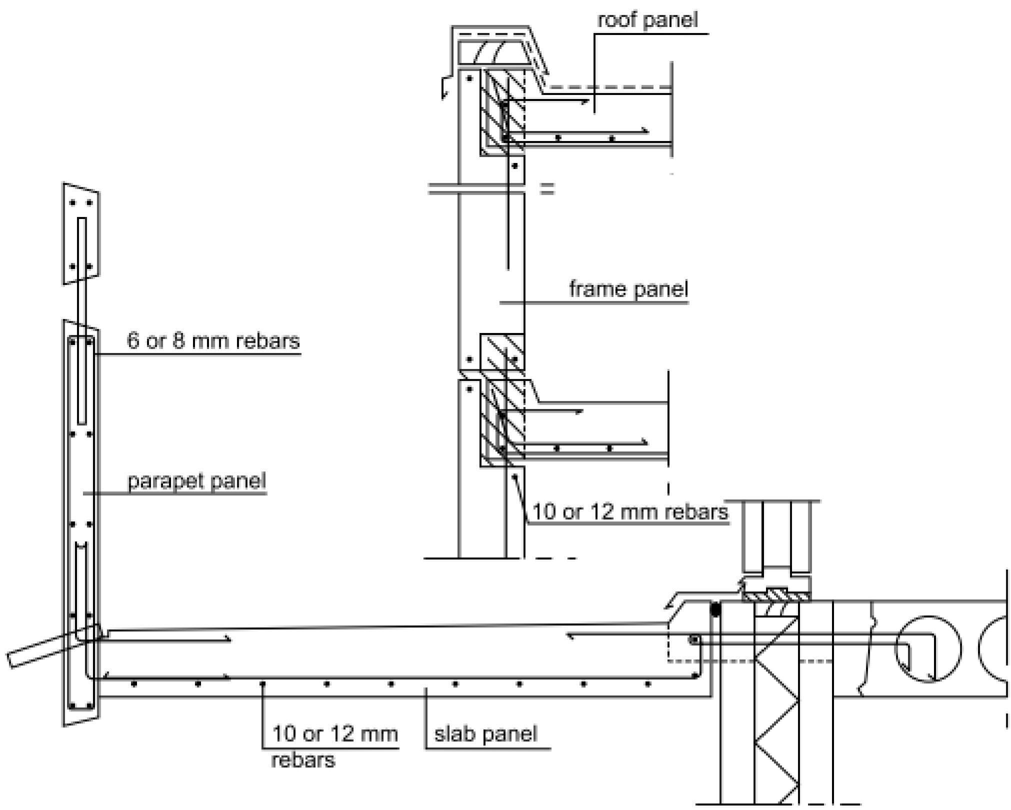


Fig. 1.4 Cross-section of a typical Finnish balcony made of precast structural members (Pentti 1994).

The typical nominal thickness of a load-bearing wall panel is between 150 and 180 mm depending on the number of floors. In general, multi-storey residential buildings have no more than eight floors. This allows using plain concrete side panels as a bearing structure and rebars only 10 to 12 mm in diameter as so-called edge bars to take the forces caused by the shrinkage of concrete and the erection of the balcony.

The nominal thickness of a bearing concrete slab is between 140 and 200 mm. It varies a lot depending on the slope of the upper surface of the slab panel. The bearing reinforcement, typically 10 to 12 mm in diameter and with a spacing of 100 to 150 mm, is in the bottom section of the slab. The upper section of the slab only has tie rods connecting it to the parapet and the frame of the building. The tops of slab panels made in the 1960's and 70's were usually given a dense coat of paint.

The water drainage systems of balconies vary a lot. Generally, the top surface of the slab has a slight slope, which leads rainwater to a drain pipe at the corner of the slab or outside through a spout pipe in the parapet. The water drainage system of some balconies consists of a gap between the slab and the parapet, which allows rainwater to exit the balcony.

The nominal thickness of parapets is from 70 to 85 mm. Parapets usually have quite heavy reinforcement near both surfaces, vertical rebars 6 to 8 mm in diameter spaced 150 mm apart. Parapets are most often joined to the slab by casting them together.

Hung balcony

Hung balconies are typically prefabricated structures of two main types: container balconies and balconies supported on wing walls.

In precast so-called container balconies the parapet and walls form a single element hung by steel lugs (e.g. I-beams) from the upper or lower corners of wing walls and resting on the edge of the external or intermediate wall or intermediate slab. Balconies suspended from wing walls are made up of separate parapet, slab and side elements, but the entire balcony structure is hung from the building frame using side panel starter bars to counteract both vertical and horizontal forces. Attachment may have been implemented e.g. by having the lowest balcony element support all balcony elements above it, which are secured only against horizontal forces to the building frame.

Cantilever balcony

Cantilever balconies may protrude from the building frame or be recessed. They are usually supported on steel rails or joists resting on a cast-in-situ intermediate slab – they are also normally cast-in-situ. In some balconies the reinforced slab or beams may extend through the external wall, or the main bars of the slab may penetrate to the intermediate slab through slit insulation. Cast-in-situ balconies are generally of the same grade of concrete as the building frame.

The top surface of a bearing balcony slab is often waterproofed either by bitumen spraying or a bitumen membrane. So-called floating concrete slab, usually sloping outwards, is cast on top of the waterproofing. The parapets of old cantilever balconies are usually of steel or concrete. Drainage is normally through a gap between parapet and slab or a small channel at the joint between the balcony slab and parapet.

1.2.3 Durability requirements of design standards

In the early years of prefabrication of concrete panels durability and service-life issues were not considered as important as efficient production. As was typical from the early 1960's until the mid-70's, all durability requirements concerned only load-bearing concrete structures, which the outer layer of a facade panel is not.

The concrete grade used in facade panels as well as most structural members of balconies was C20/25 since 1965 until the late 1980's (Vikström 1991). Thereafter, the compressive strength of concrete was raised to C25/30 (Guidelines for durability and service life of concrete structures 1989) and again in 1992 to C35/45 (Guidelines for durability and service life of concrete structures 1992). The basic idea was to improve the durability of outdoor concrete structures. However, it was still possible to use lower grade concrete for outdoor structures by increasing reinforcement cover depth respectively.

In the early 1960's, the requirements for cover depths of reinforcement depended only on strength of material and deformation of the concrete structure. Durability issues were taken into account only in the 1980 official concrete code, which defined the basic cover depth requirement based on exposure class (Finnish concrete code 1980). The basic value was set at 25 mm, but an exception was made in the case of auxiliary rebars, which required only a 15 mm cover. However, reinforcement cover depth requirements have varied over time between 10 and 25 mm depending on whether the reinforcement is of smooth or ribbed bars (Vikström 1991). Various other interpretations are also presented, depending on whether the cover depth requirements are intended to apply only to load-bearing reinforcement or all reinforcement.

An official recommendation concerning the air-entrainment of concrete to enhance frost resistance was given in 1976 (Durability of concrete 1976), and an official requirement concerning frost resistance of concrete in outdoor climate was set as late as 1980 (Finnish concrete code 1980). Air-entrainment of concrete used for outdoor structures started already in the 1960's, but was used only occasionally. According to the recommendation and later requirements, the protective pore ratio of frost resistant concrete should generally be over 0.15 and in a severe climate over 0.20.

In the early days precast concrete panels were sometimes manufactured in unheated off-site or temporary on-site prefabrication plants. Chlorides were mixed into concrete with the purpose of accelerating the hardening process. Acceleration was needed especially in winter. Until 1980 the maximum allowed chloride content of concrete was 2 per cent of cement weight and was subsequently limited to 1 per cent (Finnish concrete code 1980). The 1989 guidelines for durability of concrete structures set the allowable chloride content at 0.4 per cent, and only three years later to 0.2 per cent of cement weight (Guidelines for durability and service life of concrete structures, 1989 and 1992).

On service-life estimation

Traditionally durability design of concrete structures has been based on implicit rules given in the valid Finnish concrete code. The rules governing durability design of concrete structures have typically covered different material and structural characteristics such as concrete grade, concrete cover, water/cement ratio, air content, chloride content, crack widths, etc.

In 1980 (Finnish concrete code) concrete structures were divided into several classes on the basis of the environmental conditions they were exposed to: indoor climate, freezing and driving rain, contact with chlorides, etc. The severity of the environmental conditions was taken into account by applying different rules to different types of exposure.

A generally accepted deterioration model for corrosion of steel was presented by Tuutti (1982). According to his model, the service life of a concrete structure can be divided in two periods: the initiation period and the propagation period. During the initiation period, carbon dioxide or chloride ions penetrate the protective concrete cover. The initiation phase ends when harmful substances finally reach the steel and corrosion can start. It must be noted that no actual deterioration has occurred in the structure until this point. The propagation period follows immediately after the initiation period. The reinforcement corrodes until a certain acceptable limit for corrosion depth is reached. The limit is mostly aesthetical as far as concrete facades and balconies are concerned.

The first guidelines for estimating the service life of concrete structures were presented in Finland as late as 1989 (Guidelines for durability and service life of concrete structures, 1989). The service life of a concrete structure can be calculated separately for carbonation-induced corrosion and frost-resistance of concrete. The calculation is based on material and structural characteristics such as air content of fresh concrete, concrete grade and concrete cover. Also factors for environmental exposure and active corrosion time are given as well as coefficient of variation for all factors.

According to the valid Finnish concrete code (2004), service-life estimation is done using the so-called factor method which takes into account several different factors related to the durability of a concrete structure. However, the service life of concrete structures will furthermore expire at the end of the initiation period. According to Tuutti's model (1982), if the concrete has a high concentration of chlorides, there will be no initiation period at all and the propagation phase will start immediately after casting. Likewise, if the frost resistance of the concrete is inadequate, there will be no initiation period and frost damage will propagate if environmental exposure is severe enough.

Until the 1990's structural engineers had no means of estimating the service life of concrete structures. They could only apply the implicit rules given in the valid Finnish concrete code to produce as durable concrete structures as possible. All existing service-life models and estimation methods have been developed for new concrete structures. They cannot be directly adapted to existing concrete structures because of the lack of material properties and the already occurred deterioration of existing concrete structures.

1.3 Objectives

Rational repair of precast concrete facades has been practiced on large scale for about the last 20 years. The varying structural condition of buildings, and the fact that the most significant damage cannot often be observed visibly before it has progressed too far, requires a thorough condition investigation in most facade repair cases.

A large body of data on implemented repair projects has been accumulated in the form of documents prepared in connection with condition investigations. About a thousand precast multi-storey residential buildings have been subjected to a condition investigation, which has produced painstakingly documented material on each building, including the buildings' structures and accurate reports on observed damage and need for repairs based on accurate field investigations and laboratory analyses.

The general objective of this research is to study the factors that have actually had an impact on the service life of concrete facades and balconies and the occurrence and progress of deterioration in them. The sub-goals of the research are:

- To compile into a database the data on concrete facades and balconies of actual buildings gathered in condition investigations.
- To determine the actual durability properties of concrete facades and balconies.
- To determine which factors have actually had an impact on the occurrence and progress of different deterioration mechanisms in concrete facades.

- To find out the relative importance of those factors.
- To estimate the effects of different climate conditions on the deterioration of concrete facades and balconies.
- To provide new reliable data on the service lives of concrete facades and balconies for use in durability design and LCC analyses of concrete structures.

1.4 Scope of the research

The objective of this research is to evaluate the performance of Finnish precast reinforced concrete facades and balconies from the viewpoint of different deterioration mechanisms and determine their actual service lives. There are many degradation mechanisms that can potentially limit the service life of an outdoor concrete structure. Most of them will be shortly reviewed in the next chapter, but the most important ones, carbonation-induced reinforcement corrosion and frost damage of concrete, will be dealt with in more detail.

The structures targeted by this research are Finnish concrete facades and balconies made of precast panels from the 1960's to the 1990's. They have been exposed to normal Nordic climate conditions for many years after having been installed in place. The definition establishes that the typical compressive strength of concrete of these structures was C20/25 until the late 1980’s and thereafter C25/30 (Vikström 1991).

Only the durability properties and actual service life of precast reinforced concrete facades and balconies will be studied here. Calculated service lives, based on various probabilities of the distribution and deterioration of material properties, are not examined in this context. All the input data on existing concrete structures required by service-life models, such as water-cement ratio and concrete curing, are often not available, or they may be unreliable. As a rule, all factors affecting deterioration are not available. The study uses those measured quantities and data on buildings, structures, state of deterioration and recommended repair methods available in condition investigation reports.

The research assesses the impact of climate conditions on the deterioration of concrete structures in different parts of Finland − in the coastal area, inland, etc. The assessment will be based on data collected from various weather stations of the Finnish Meteorological Institute. There are factors in the surroundings of every building that affect the actual exposure conditions of facades and balconies. Such building- or structural member-specific climate data cannot be produced without extensive, long-term measuring arrangements, and are thus not available for this study.

1.5 Implementation of the research

The research is based on condition investigation data from existing concrete buildings and measured weather data. The research material consist of the database on material properties and deterioration of existing Finnish concrete facade panels and balconies built between 1960 and 1996, and weather observations since 1961 by the Finnish Meteorological Institute (FMI). A description and an evaluation of the research material is provided in Chapter 4.

Collection of condition investigation data from sector actors and compilation of the data in a database in the most usable form possible have been the most challenging and time-consuming part of the study.

The core content of the study are the durability properties of precast panels used in actual construction and how the deterioration of buildings made of different precast panels has occurred in actual natural conditions. Each panel and surface type has been examined separately for various degradation mechanisms.

The possibility of achieving the reinforcement cover depths required by the Concrete Code with different panel and surface types has been assessed on the basis of the length of samples drilled from precast panels. The success of concreting and capillary porosity have been evaluated on the basis of analyses of the same samples, which have an impact on both carbonation and wetting of concrete in outdoor conditions.

Reinforcement cover depth distributions have been studied by measurements on samples and non-destructive field measurements. The achievement of the durability properties set in the Concrete Code as well as the occurrence of corrosion damage have been assessed on the basis of the formed panel- and surface type-specific cover depth distributions.

Corrosion of reinforcements has been examined from the viewpoints of concrete carbonation and the chlorides in concrete. Observations about the propagation of carbonation have been reviewed together with reinforcement cover depths and visible damage. Carbonation of concrete proceeds as a function of time, which is why the carbonation of concrete around steel reinforcements has been analysed using a carbonation coefficient calculated on the basis of Fick's first law (Tuutti 1982, Bakker 1988). The impacts of porosity of concrete and building location on carbonation rate have been studied by panels and surface types.

The success of air-entrainment of concrete has been examined by both protective pore tests and thin-section analyses. Frost attack has been evaluated on the basis of visible damage, tensile-strength-test results for concrete samples and thin-section analyses. Achievement of the durability properties set in the Concrete Code has been examined by protective pore tests and thin-section analyses.

The study introduced a new approach to the investigation of reinforcement corrosion and frost attack of concrete by examining detected damage and durability properties in conjunction with long-term weather information. Occurrence of reinforcement corrosion damage was examined in conjunction with prevailing wind and rain and sleet information. Frost attack of concrete was also investigated in conjunction with prevailing wind and rain and sleet information. A new analysing criterion has been the freeze-thaw cycles following rainfall based on different temperature criteria.

The results and discussion are presented in Chapter 5. The results are presented mainly in the form of tables and graphs, separately for facades and balcony structures. Where no risk of confusion exists, or something applies to both facades and balconies, no distinction is made.

Conclusions as well as utilisation of the results and suggestions about areas requiring further research are found in Chapter 6.

2 ON THE DEGRADATION MECHANISMS OF PRECAST CONCRETE FACADES AND BALCONIES

Precast concrete facades and balconies exposed to Finnish outdoor climate are subject to several degradation mechanisms, whose progress depends on many factors related to structure, exposure and materials. Degradation may limit the service life of structures and, therefore, the possibility of retaining the present or original appearance of structural members and buildings. It is important to know the basics of the degradation mechanisms of concrete to be able to successfully apply suitable repair measures. Degradation may, for instance, have detrimental visual impacts or even reduce the bearing capacity of structures.

The different types of degradation mechanisms of concrete facades and balconies are shortly dealt with in this chapter based on a literature review. There is a lot of theoretical knowledge and practical experience from the service lives and degradation mechanisms of reinforced concrete structures.

2.1 Corrosion of reinforcement

2.1.1 Corrosion protection of reinforcement in sound concrete

Hardened concrete is a mixture of cement, aggregates and water. The properties of concrete can be, and usually are, modified by the use of admixtures and other binders like fly ash or slag. In any case, concrete is always a more or less porous material (Neville 1995). Consequently, any steel reinforcement embedded in concrete is usually in contact with moisture and oxygen through its pore system. The high alkalinity of pore water due to the hydration products of Portland cement does not corrode steel surfaces significantly; it rather passivates them if the halide content (usually chlorides) of the concrete is low enough (Bakker 1988).

Corrosion protection of mild steel in concrete is based solely on the high alkalinity of concrete, or rather of the pore solution in the concrete at the steel surface, deriving from small quantities of readily soluble alkali hydroxides, NaOH and KOH, and a large proportion of less soluble lime, $Ca(OH)_2$. These are mainly responsible for the buffering action of concrete (Bakker 1988 and Gjørv 2009). The pH of solid concrete is usually above 13. Such high alkalinity forms a thin and dense oxide layer on a steel surface (Page 1988), which very efficiently protects all embedded steel from corrosion.

Passivation of a steel surface is very efficient corrosion protection, because the passive film is usually self-healing as long as pH remains high and chloride content remains sufficiently low. According to Parrott (1987), the critical pH level of non-chloride-contaminated concrete is somewhere between 11 and 11.5, which cannot be preserved under the passive layer.

The corrosion protection of steel is electrochemically based primarily on alkalinity. On the other hand, physical protection in the form of a sufficiently dense, thick and uniform layer of concrete on top of the steel is needed to prevent harmful substances, like chlorides and acids, from penetrating into the concrete surrounding the steel (Durable concrete structures 1992).

2.1.2 Carbonation of concrete

As earlier stated, the alkalinity of concrete is mainly due to the calcium hydroxide and alkali hydroxide produced by the hydration of cement. These hydroxides can react with acid substances such as the carbon dioxide of the air mixed with pore water. This neutralisation reaction of concrete is called carbonation.

The main process describing the carbonation of concrete is the reaction of the alkaline calcium hydroxide in concrete ($Ca(OH)_2$) with acid carbon dioxide (CO_2), which produces neutral calcium carbonate ($CaCO_3$). The reaction can be presented in highly simplified form as (Bakker 1988):

$$Ca(OH)_2 + CO_2 \xrightarrow{(H_2O)} CaCO_3 + H_2O \qquad (2.1)$$

In addition to calcium hydroxide, many other hydrated ingredients of cement take part in the carbonation reaction (Bakker 1988, Kobayashi et al. 1994).

The most significant consequence of the carbonation of concrete for corrosion protection of reinforcements, and thus also the repair need of concrete structures, is the reduction of pore water pH to 8-8.5 (Neville 1995). At such low alkalinity, the passivity layer protecting reinforcements is destroyed, which allows corrosion of the reinforcing steels embedded in the concrete to start in the presence of sufficient moisture and oxygen. Carbonation reduces the porosity of concrete made with Ordinary Portland Cement (OPC) and increases its compressive strength. Opposite phenomena have been observed in concretes containing blast furnace cement (BFC) (Neville 1995).

Carbonation begins at the surface of concrete and propagates slowly as a front deeper into it only after the alkaline substances on the surface reacting with carbon dioxide have become neutralised (Tuutti 1982). In extremely dry conditions, when the pore system of the concrete contains little water, carbonation proceeds extremely slowly. That allows carbon dioxide to penetrate deeper into the alkaline substance whereby carbonation does not propagate as an even front (Parrot 1987).

Basically, carbonation is diffusion of carbon dioxide through the carbonated layer to the reaction zone according to Fick's law. Propagation of carbonation is most often described by an equation based on Fick's first law (Tuutti 1982, Bakker 1988):

$x = k \cdot t^{0.5}$ where (2.2)

- x is carbonation depth [mm]
- k is carbonation coefficient [$mm/a^{0.5}$], and
- t is time [a].

The basic assumption of the above equation is homogeneity of concrete, i.e. the properties affecting carbonation rate are similar at all concrete depths. However, actual concrete structures are not ideally homogenous, but vary e.g. as to required compaction and curing period and, especially, prevailing moisture conditions. Thus, it is natural that the carbonation of the concrete used in facades and balconies only rarely follows closely the presented model. Since concrete is often denser and hydration has progressed further inside a concrete structure than on its surfaces, carbonation of concrete with respect to time is often slower than in the presented parabolic model. Likewise, the higher moisture content inside concrete compared to the surface layers may also result in slower carbonation (Bakker 1988). The rate of carbonation is influenced mainly by the following:

- factors affecting the diffusion resistance of the material between outdoor air and the carbonation zone
- the amount of substances taking part in the carbonation reaction of the carbonating material
- carbon dioxide content of ambient air
- temperature.

Strength of concrete has only a secondary impact on carbonation rate, through carbonating-binder content and water-cement ratio, as it affects the diffusion resistance of concrete.

Diffusion resistance. The diffusion resistance of concrete is mainly affected by the amount and type of porosity and the moisture content of the pore system (Parrott 1987). The porosity of concrete, i.e. the amount and size of pores, depends on the water-cement ratio and hydration rate of cement as well as the amount of binder used (Neville 1995). As the water-cement ratio falls and the compressive strength of concrete generally simultaneously increases, the hydration rate of cement increases, porosity decreases, and density increases strongly. At the same time the carbonation rate also decelerates. Since carbonation of concrete starts from the surface of a structure, the density of the surface is highly significant. It is impacted e.g. by the absorptive capacity of used forms and, especially, the care taken in curing surfaces that are not form finished. Inadequate compaction of concrete and micro-cracking lower concrete's diffusion resistance and increase the rate of carbonation correspondingly wherever they occur.

The moisture content of the pore system has a major influence on the diffusion resistance of concrete as the diffusion rate of carbon dioxide to air is about 10,000 times greater than to water-filled pores (Bakker 1988). In practice, carbon dioxide cannot penetrate into the pore system of concrete while the surfaces of capillary pores are covered by condensation moisture. According to Parrott (1987), the optimal relative humidity for carbonation is 50-70%. Below that, the pore system does not contain enough water for the carbonation reaction to occur.

Amount of reactive material. The amount of the carbonating substance determines how much carbon dioxide is consumed in carbonation reactions at different levels to enable the carbonation front to penetrate deeper into the concrete. It depends on the type and amount of binder and hydration rate.

The reacting materials of the carbonation process are mainly hydration products of lime-based (CaO) components of cement. The quality of the binder, and consequently the amount of alkaline substances in the cement, has an essential impact on carbonation rate. Ordinary Portland Cement (OPC) contains about 64% calcium oxide (CaO) while blast furnace cement (BFC) may contain only about 44%. Other blended cements, such as those containing fly ash (PFA) contain amounts of CaO that fall between the above-mentioned amounts (Bakker 1988). The hydration speed of these blended cements is also slower than that of Portland cement, which is why they generally need a longer than usual curing period. Therefore, the hydration rate may also be lower, which may increase the porosity and carbonation rate of concrete compared to concrete made of Portland cement (Neville 1995).

Carbon dioxide content. The carbonation process of concrete structures under outdoor conditions is initiated and maintained by atmospheric carbon dioxide. The carbon dioxide concentrations in the air may differ slightly by localities. Generally, higher than average concentrations may occur momentarily in spaces or areas, where ventilation is unable to remove carbon dioxide more quickly as it is produced. A typical example is a busy multi-storey car park.

Atmospheric carbon dioxide content has been measured since the 1950's. During 1957-2007 it increased from about 280 ppm to a good 380 ppm, and growth continues at an average annual rate of 2 ppm. In summer the carbon dioxide content of air decreases slightly due to the large amounts assimilated by the vegetation of the land areas of the Northern hemisphere (Finnish Meteorological Institute 2010). Despite the significant change in the carbon dioxide content, it still accounts for 0.04% of all atmospheric gases. In theory, carbonation of concrete accelerates continually since more carbon dioxide is available. Other factors influencing the carbonation rate, such as the diffusion resistance of concrete and the amount of reacting material, are nevertheless much more crucial.

Temperature. In northern conditions temperature has an impact on the diffusion resistance of concrete. At lower temperatures the relative humidity of air is generally high, allowing moisture to block the pore system of concrete, thereby preventing carbon dioxide from penetrating into the concrete. As temperature drops below 0 °C, water starts to freeze in larger pores, which affects the diffusion of gases. As a rule, chemical reactions slow down as temperature falls due e.g. to the slowing of the movement of molecules. However, the impact of temperature on carbonation rate at normal outdoor temperatures above 0 °C has been found to be minor (Saetta et al. 1993).

Effect of cracks. In principle, carbon dioxide can penetrate more quickly into concrete through cracks and thereby also accelerate carbonation (Bakker 1988). Yet, in practice, cracks do not seem to have any major impact on concrete carbonation or the corrosion induced by it (Tuutti 1982). The most common cracks are relatively narrow (0.1-0.4 mm) and perpendicular to reinforcement steels. Narrow cracks impede the penetration of water and dry out more slowly than an exposed outer surface, which means that the carbon dioxide diffusion resistance of narrow cracks of outdoor concrete structures is actually very high.

2.1.3 Presence of chlorides

Another problem besides concrete carbonation is that the corrosion protection based on the passivity of reinforcement steels can be lost if harmful amounts of chlorides end up on their surfaces (Neville 1995).

Chlorides may penetrate into hardened concrete if the surface of the concrete is subject to external chloride stress e.g. due to de-icing and dust binding salts or sea water splashes (Bakker 1988). In the case of concrete facades and balconies, such chloride stresses occur relatively infrequently and they are generally localised. Sea water splashes affect buildings very close to a sea shore (Neville 1995). On the other hand, it is possible that chlorides have been added to the concrete mix used for facades and balconies during preparation to accelerate hardening of the concrete (Bakker 1988, Pentti et al. 1998, Gjørv 2009). If an excessive amount of chloride has been used initially as an accelerator, it has not been possible for a passive film to form on the steel surfaces to protect them from corrosion (Gjørv 2009).

Chlorides can penetrate into hardened concrete only in water-soluble form, either by diffusing in still pore water or, more generally, through chloride-containing water being absorbed into the pore system of capillary concrete (Bakker 1988). Since chlorides enter concrete in water-soluble form, cracks allow them to penetrate easily and quickly quite deep into concrete compared to penetration only through the pore system. Cracks less than 0.05 mm wide in a concrete structure are already significant as regards the penetration of chlorides (de Rooij et al. 2007). Thus, the structure of the concrete's pore system, the cracking of the concrete structure and, especially, the environmental conditions of the location of the structure greatly affect the penetration of chlorides into concrete.

Part of the chlorides bind physically to the calcium oxide (CaO). The amount of the binding chloride depends on the type and amount of the cement used. Ordinary Portland Cement (OPC) contains clearly more chloride-binding calcium hydroxide than blast furnace slag cement (BFC) or pulverised fuel ash (PFA) cement. On the other hand, these different blended cements produce denser concretes that are more impermeable to chloride-containing water (Bakker 1988, Neville 1995).

The chemical and physical binding of chlorides reduces the concentration of so-called free chlorides in pore water (Bakker 1988, Neville 1995). In the case of chloride-containing concrete, carbonation of concrete releases chloride bound to the cement stone into the pore water leading to significant acceleration of chloride corrosion due to carbonation of chloride-containing concrete (Pentti et al. 1998).

Chloride corrosion of reinforcement steels is typically severe pitting corrosion. That is generally due to the small anodic area, where oxidation takes place, and a correspondingly large cathodic area, which allows a strong corrosion current to the anodic area (Treadaway 1988). Since the corrosion products of chloride corrosion are more easily soluble in pore water than the products of carbonation-induced corrosion (Page 1988), corrosion may proceed far without any outward visual evidence. Moreover, the corrosion may take place at lower than normal humidity since chlorides are hygroscopic. Corrosion may also advance at lower temperatures since salts reduce slightly the freezing point of water (Pentti et al. 1998).

Even small chloride contents may destroy the protective passive film, which allows reinforcement corrosion to initiate also in alkaline (uncarbonated) concrete. It is difficult to set an exact limit value for critical chloride content since it is influenced by numerous factors including binder type and amount and porosity, pH and moisture content of concrete. Critical chloride content also depends on the used chloride testing method and whether limit values are set only for water-soluble chloride content or also for bound chloride content, in which case the test result must indicate the acid-soluble chloride content.

Many different limit values for critical chloride content are presented in literature due e.g. to the above reasons, ranging from 0.17-2.5% of the weight of cement either as acid-soluble or free water-soluble chlorides (Taylor et al. 1999). Alonso et al. (2000) suggested a critical total chloride limit for Ordinary Portland Cement of 1.24-3.98% by weight of cement where the corresponding share of free water-soluble chlorides is 0.39-1.16% by weight of cement. Finnish guidelines consider 0.03-0.07% acid-soluble chloride by weight of concrete critical (Condition investigation manual for concrete facade panels 2002). To avoid confusion, it should be noted that critical chloride content is usually expressed as a proportion of the weight of cement.

2.1.4 Active corrosion

The corrosion of steel in a water-soluble environment has generally been studied from the electrochemical perspective. The surface of rusting steel comprises anodic areas, where positive ions dissolve in the electrolyte, and cathodic areas, where extra negatively charged electrons can migrate to along the steel surface. In the concrete pore water near the anodic area an oxidation reaction takes place, where positively charged iron ions react either with chlorides or hydroxyl ions producing either water-soluble (e.g. ferrous chloride ($FeCl_2$)) or water-insoluble corrosion products, such as rust (Page 1988). Figure 2.1 is a simplified description of steel corrosion in a water-soluble environment.

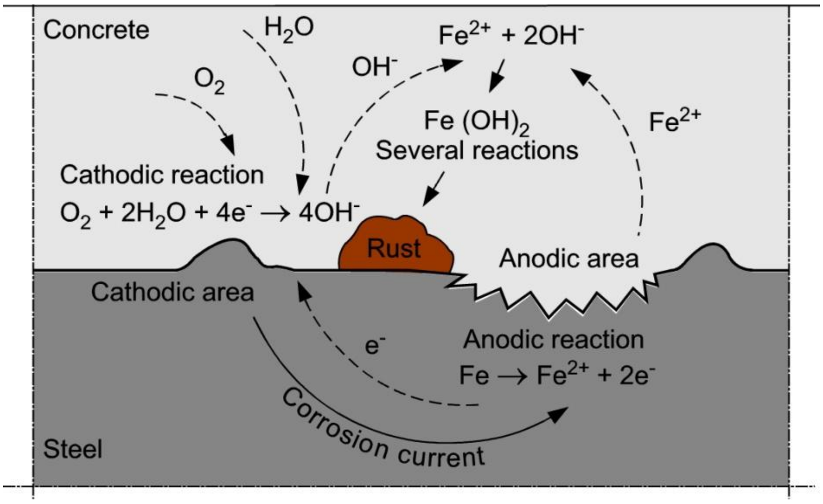


Fig. 2.1 *A simplified schematic presentation of the electrochemical reactions of steel in a neutral or alkaline solution (Mattila 2003).*

In an alkaline or neutral environment containing no chlorides, the partial reactions are (Broomfield 1997):

Anodic:	$Fe \rightarrow Fe^{2+} + 2e^-$	(standard Redox potential -0.440 V)	(2.3)
Cathodic:	$2H_2O + O_2 + 4e^- \rightarrow 4\ OH^-$	(standard Redox potential 0.401 V)	(2.4)

The requirements of electrochemical corrosion are (Page 1988):
- A reactive metal surface which can oxide anodically to form soluble ions
- A reducible substance which acts as a cathodic reactant
- An electrolyte which allows the migration and movement of ions between anodic and cathodic areas
- An electron conductor between anodic and cathodic areas.

As already earlier stated, corrosion cannot occur in steels embedded in solid concrete since the alkalinity of concrete forms a passive layer around them. The passive layer prevents anodic dissolution of ions into pore water (Bakker 1988). The passive layer of steel may be destroyed either by concrete carbonation or chlorides (Treadaway 1988). Carbonation leads to general wide spread corrosion while corrosion induced by chlorides usually appears as local pitting.

General corrosion. As a result of carbonation, the pH of the water in the concrete pore system drops to around 8.5 (Neville 1995). At such a low alkalinity level, the passive layer protecting steels becomes thermodynamically unstable and dissolves in the electrolyte exposing the bare steel surface (Treadaway 1988). Reinforcements embedded in concrete generally corrode in much the same fashion as bare steel exposed to the elements normally does (Philip and Schweitzer 1988). In general corrosion, the anodic and cathodic areas are not stationary, but fluctuate in the corrosion environment on the microscopic level. Therefore, the corrosion of steel is highly uniform across the entire rusting surface.

Pitting corrosion. As the critical chloride content of the water in the concrete's pore system is exceeded, localised breakages may occur in the passive film due to so-called free chlorides. A local breakage in the passive film is more likely when the chloride concentration of the concrete is not uniform. In pitting corrosion, the anodic area remains stationary for long, which means that steel corrosion is local and goes deep while adjacent areas that remain cathodic are unaffected (Treadaway 1988).

Corrosion rate

The control of anodic dissolution of ions with the help of the passive layer on steel becomes irrelevant when the passive layer vanishes either due to concrete carbonation or chlorides. Subsequently, the reactions and reaction speeds of the substances taking part in the corrosion process control the corrosion rate. The corrosion rate is influenced by several factors, the most common ones being:

- Electrical resistivity
- pH of pore solution
- Oxygen availability
- Temperature.

Electrical resistivity. Factors influencing the specific resistance of concrete are the key factors affecting corrosion rate. The specific resistance of concrete depends on the porosity of concrete, which can be changed e.g. by altering the water-cement ratio. By lowering the ratio from 0.7 to 0.5, specific resistance can be increased by about 2.5×10^2 Ωcm. (Gjørv 2009).

Porosity of concrete and the amount and electrical conductivity of pore water influence greatly the specific resistance of concrete. The conductivity of pore water is relatively high due to salts that have dissolved from the concrete, which means that porosity and moisture content of concrete affect its specific resistance (Polder 2002). The concentration of free chlorides in pore water also alters conductivity: the higher the chloride concentration, the higher the conductivity of pore water (Fiore et al. 1996).

The specific resistance of dry concrete is high since the content of electrically conductive pore water is low. Then, the corrosion rate of reinforcement is also at its lowest. The corrosion of steel initiated by carbonation is generally considered to start when the relative humidity of concrete exceeds 65-70%. Corrosion rate increases significantly as relative humidity exceeds the 80-85% level (Tuutti 1982). Corrosion due to chlorides begins already at smaller moisture contents, and is often clearly faster than corrosion initiated by carbonation.

pH of pore solution. As the surface of carbonated concrete dries by evaporation, the concentration of the alkaline salts dissolved in the pore water increases as the amount of water decreases, which temporarily increases the pH level. Higher pH again passivates the steel surface, but the passive layer quickly disappears as the moisture content of carbonated concrete rises again.

Oxygen availability. The oxygen concentration of the atmosphere is around 210 ml/l while that of water is 5-10 ml/l at a maximum (Gjørv 2009). The ingress of oxygen into concrete is limited mainly by the structure and degree of filling of the pore system (Treadaway 1988). Oxygen is present in concrete structures in contact with outdoor air at least intermittently (after the structure has dried) meaning that lack of oxygen does not restrain corrosion rate in practice.

Temperature. The temperature of the corrosion environment has a significant impact on corrosion rate since rising temperature allows the substances participating in the corrosion process to dissolve and move about better, which accelerates many chemical and electrochemical reactions. At low temperatures, the liquid acting as an electrolyte freezes, which considerably affects the electrolyte's specific resistance. Then, corrosion decelerates and becomes insignificant as ice is a poor electrolyte (Philip and Schweitzer 1988). Yet, it must be noted that all pore water does not freeze immediately as temperature falls below 0 °C because pore water contains water-soluble salts. If the concrete contains a significant amount of chlorides, the freezing temperature of pore water drops considerably. It must also be considered that the size of concrete pores has an impact on the freezing conditions of pore water, which means that all water in the pore system does not freeze simultaneously everywhere. Freezing occurs at lower temperatures in smaller pores, in the case of gel pores even at -30 °C (Pigeon and Pleau 1995). Measurable corrosion has still been observed at -15 °C (Mattila and Pentti 2004).

Corrosion current is generally expressed in terms of the thickness of the metallic oxide layer forming in a year [µm/a]. The amount of corroded metal can be derived from the corrosion current based on Faraday's law (Andrade 2002):

$$\Delta W = ItW_m/ZF \qquad \text{where} \qquad (2.5)$$

ΔW is weight loss [g]
I is electrical current [A]
t is time [s]
W_m is molecular weight of corroding metal [g]
Z is valence of corroding metal, i.e. number of electrons involved in the electrochemical reaction (= 2 for steel), and
F is Faradays constant, 96487 C/g.

Experience tells us that the corrosion rate of mild steels in carbonated concrete commonly varies between 0-10 µA/cm^2 (Tuutti 1982). Very low values, typically less than 0.1 µA/cm^2, have been measured for steel in passive state. Values over 1 µA/cm^2 have been measured only rarely (Andrade 2002). The correlation between corrosion current and corrosion rate of reinforcements is presented in Table 2.1.

Table 2.1 Correlation between corrosion rate and corrosion current (Andrade 2002).

Corrosion level	Corrosion rate [µA/cm^2]	Corrosion penetration [µm/a]
Negligible	< 0.1	< 1
Low	0.1-0.5	1-5
Moderate	0.5-1	5-10
High	> 1	> 10

Effects of corrosion

In carbonation-induced corrosion, the corrosion may propagate for a long time before it can be noticed on the surface of a concrete structure. Because corrosion products are not water soluble, they accumulate on the surface of steel near the anodic area (Mattila 1995). This generates an internal pressure, because the volume of the corrosion products induced by carbonation is three to six times bigger than that of the original steels (Tuutti 1982). Internal pressure caused by corrosion products leads to cracking or spalling of the concrete cover. Visible damage appears first at the spots where the concrete cover is thinnest.

The corrosion of steels embedded in concrete structures generally affects their appearance more than their load-bearing capacity. According to Andrade (2002), at the rapid corrosion rate of 1 µm/cm^2 steel corrodes 10 µm/a. Thus, it takes several years of severe corrosion for a steel cross-section to corrode enough to have a significant impact on the load-bearing capacity of a structure. The corrosion of steel causes cracks and chipping in the concrete cover when the cross-section has decreased about 15-50 µm (Alonso et al. 1998). Loss of reinforcement anchorage or setting due to cracking or spalling of concrete may affect structural integrity and performance much faster than a reduction in the cross-section of steels.

Chloride-induced corrosion is typically pitting corrosion, where at least part of the corrosion products are water soluble, which allows corrosion of steels to propagate considerably before visual damage can be detected on the concrete cover.

2.2 Disintegration of concrete

Concrete is a very brittle material, it can withstand only limited tensile stress without cracking. Internal tensile stresses due to expansion processes inside concrete may result in internal cracking and, therefore, disintegration of concrete. Concrete may disintegrate as a result of several phenomena causing internal expansion, such as frost attack, formation of late ettringite or alkali-aggregate reaction.

2.2.1 Frost damage

Frost damage is the result of the hydraulic pressure caused by the freezing dilation of water in the concrete's pore system. Water enters the pore system e.g. as a consequence of driving rain or sleet. In the winter season the structure dries slowly due to low temperature, high relative humidity of air and low level of solar radiation. Temperature may drop below zero rapidly.

Theoretical models explaining frost attack on concrete

A frost attack caused by a heavy moisture load is commonly responsible for the deterioration of concrete structures in Nordic outdoor climate. Concrete is a porous material, whose pore system may, depending on the conditions, hold varying amounts of water. As the water in the pore system freezes, it expands about 9% by volume creating hydraulic pressure in the system. If the level of water saturation of the system is high, the overpressure cannot escape into air-filled pores and consequently damages the internal structure of the concrete resulting in its degradation.

More than 15 different theories or explanations for frost attack on porous materials have been presented over the years (Kuosa and Vesikari 2000). It has been discovered that frost attack is a complex process, and that frost damage can occur in many different ways (Fagerlund 1997).

Probably the most widely known frost damage theory is the hydraulic pressure theory by Powers published in 1949. Accordingly, damage occurs as freezing water expands creating hydraulic pressure within the pore structure of a porous material. The pressure forms when part of the water in a capillary pore freezes and expands forcing the unfrozen water out of the pore. The migration of water causes localised internal tensions in the material whereby its strength may fail resulting in cracking (Powers 1949).

The theory of volume changes in microscopic ice crystals was developed to complement hydraulic pressure theory (Powers and Helmuth 1953). Accordingly, small ice crystals, which tend to grow, form in the capillary pores during the freezing process.

The growth of ice crystals is the result of the lower chemical potential of ice crystals than the super-cooled pore water in smaller gel pores. If there is not enough empty space for ice crystals to grow, pressure will be exerted on the pore structure, which may lead to cracking of the pore structure. It is characteristic of growing microscopic ice crystals that they will continue to grow despite the fact that temperature will stop falling during the freezing phase.

The theory of osmotic pressure complements the two previous theories by taking into account also the migration of dissolved chemicals, mainly the alkalis Na_2O and K_2O, in the pore water. Those dissolved chemicals lower the freezing point of pore water and increase the concentration of salts in the water surrounding the ice. The concentration of dissolved chemicals seeks equilibrium between different pore water solutions causing osmotic pressure on the pore structure (Pigeon and Pleau 1995).

Litvan noticed in the beginning of the 1970’s that the water in the pore structure of a porous material does not freeze immediately as temperature drops below 0 °C. Freezing takes place first in the larger capillary and gravitation pores. In smaller gel pores water begins to freeze when temperature is approximately -15 to -20 °C. The unfrozen pore water is thus super-cooled, which tends to cause drying of the paste because the saturation pressure of super-cooled water is higher than that of ice. According to Litvan, mechanical damage takes place when moisture transfer cannot occur in an orderly manner, i.e. when the rate of freezing is too high, or the distance water must travel to reach an external surface and freeze is too long (Pigeon and Pleau 1995).

The theory of critical degree of water saturation was developed in the early 1970’s (Fagerlund 1977). Its basic idea is that there is a critical degree of water saturation above which a porous and brittle material is damaged while freezing. If actual water saturation is below the critical value, no damage occurs during freezing. Fagerlund’s theory of the critical degree of water saturation is valid regardless of the actual deterioration mechanism. How the deterioration occurs is not taken into consideration, but the presence of water and occurrence of freezing are always assumed.

An increase in facade temperature increases the temperature of the ice in the pore structure. When temperature decreases, the ice in the pore structure shrinks initially, and super-cooled pore water can flow by capillary action to “new” empty spaces in pores, where it freezes rapidly. When temperature increases, the ice expands. That causes considerable hydraulic pressure on the pore structure (Penttala 1998). The temperature factor of ice is 50×10^{-6} per K. It is 3.5 to 10 times bigger than the temperature factor of concrete (Hedlund and Jonasson 2000, Mäkinen 2010). The temperature factor of concrete varies a lot and is influenced by two main constituents of concrete: hydrated cement paste and aggregate (Neville 1995).

Effects of frost damage

As discussed earlier, frost attack of concrete manifests itself initially as internal cracking or surface flaking when the hydraulic pressure of the frozen water in the pore system exceeds the tensile strength of the concrete. Cracking of concrete decreases its strength and accelerates the capillary absorption of water. As freeze-thaw cycles continue under high moisture content conditions, concrete disintegrates (Fagerlund 2002). Incipient frost damage cannot be observed visually or by knocking, but requires adoption of more accurate research methods (Pentti et al. 1998), generally thin-section analysis.

Far advanced frost damage is manifested as reduced strength of concrete, loss of bonding, or crazing or chipping off of the surface. Disintegration of concrete also accelerates carbonation of concrete due to cracking and thereby also steel corrosion.

Fig. 2.2 Far advanced frost attack curls the edge of a facade panel.

The degree of frost damage may vary in different parts of the facade – depending on, for instance, the environmental stress and variation in material properties – as well as across the thickness of the concrete structure. Frost damage due to a high local moisture stress affects only a very limited area. On the other hand, improper surface treatment of a non-frost-resistant concrete facade may result in deterioration across most of the wall surface.

Laboratory tests have shown that frost damage of concrete requires from one (Fagerlund 2002) to several thousand (Neville 1995) freeze-thaw cycles. Neville (1995) found the following significant factors concerning hardened concrete to have an impact on frost damage, and thus also on the number of freeze-thaw cycles required:

- water absorbing capacity; a more water-absorbent concrete takes fewer freeze-thaw cycles to disintegrate than a less absorbent one
- water-cement ratio of concrete; concrete with a low water-cement ratio withstands more freeze-thaw cycles than ones with a high ratio
- air-entrainment of concrete; air-entrained concrete's resistance to freeze-thaw action is many times that of non-air-entrained concrete.

Air-entrained concrete with a water-cement ratio of 0.35 can withstand up to 7,000 freeze-thaw cycles with a loss of mass of 25% while corresponding non-air-entrained concrete can only withstand about 180 cycles (Neville 1995). The number of severe freeze-thaw cycles experienced by actual concrete structures does not generally come close to a thousand during their entire service life (Fagerlund 2002).

Frost resistance of concrete

Frost-induced cracking can be prevented by air-entrainment, which means creating relatively large so-called protective pores in the concrete mix during preparation (see Fig. 2.3), which remain air filled under all conditions and can take the pressure from the freezing of water preventing damage to the structure (Kuosa and Vesikari 2000). Air-entrained concrete is called frost-resistant as it can withstand recurrent freezing and thawing without getting damaged. A basic requirement for air-entrainment is that the amount of capillary pores in the concrete, and consequently the amount of water that freezes under frost attack, does not increase due to air-entrainment (Neville 1995).

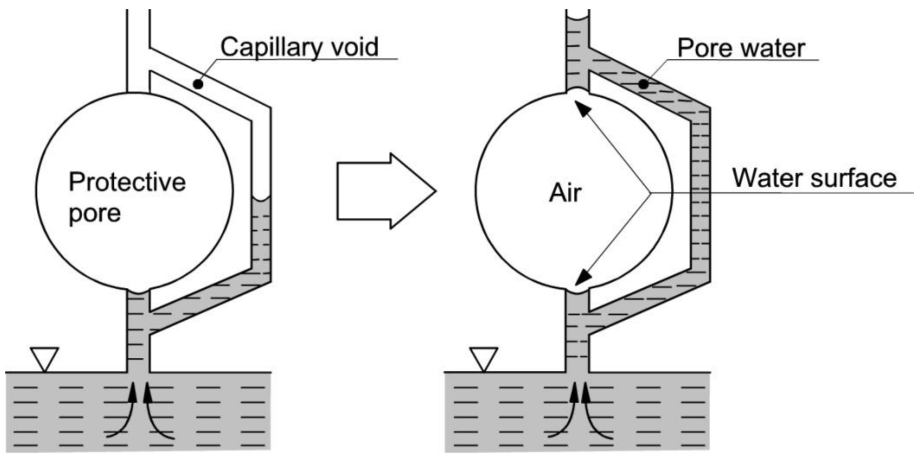


Fig. 2.3 *The principle of protective pores in concrete in contact with water (Guidelines for durability and service life of concrete structures 1992).*

Sufficient protective pore density to ensure frost durability is provided by using a suitable air-entraining agent when mixing the concrete. Pores produced by air-entrainment are typically around 50 µm in diameter (Neville 1995, Pigeon and Pleau 1995). The upper limit of the size distribution of protective pore diameters in air-entrained concrete is about 300 µm (Pigeon and Pleau 1995). Pores larger than about 10 µm may be considered useful with regard to frost durability (Pentti et al. 1998).

Adequate air content of the concrete mix does not guarantee sufficiently close spacing of pores for proper frost durability. This is due the fact that air content is a function of air pore volume, whereas the spacing factor is a function of the amount of air pores (Kuosa and Vesikari 2000). The aim should be to produce a lot of small air pores with a certain air content instead of a smaller amount of larger pores. Half of the average distance between protective pores is called the spacing factor or pore spacing.

In practice, the average diameter of protective pores is about 150-300 µm (Pigeon and Pleau 1995). For protective pores to function properly, they are to be spaced closely enough, i.e. the distance travelled by freezing water to a protective pore must not be too long. Spacing factor values vary slightly by sources. Generally, 200-250 µm is considered a safe factor for frost durability while values below 200 µm are recommended, although frost resistant concrete is known to have been achieved with larger values (300-400 µm) (Pigeon et al. 1985, Neville 1995, Pigeon and Pleau 1995, Dhir et al. 1999, Kuosa and Vesikari 2000). The Finnish Concrete Code sets a maximum spacing factor for each concrete structure based on its environmental stress conditions and planned service life. The minimum requirement is ≤ 270 µm (stress class XF 1, service life 50 years) and maximum ≤ 200 µm (stress class XF 3, service life 200 years) (Finnish Concrete Code 2004).

Air-entraining agents enhance the mixing of air with concrete during mixing. The agents' most important task is, however, to ensure the permanence of small pores in the concrete mix since they tend to combine into larger pores that are not beneficial from the viewpoint of frost resistance (Neville 1995). Most air-entraining agents work on the principle of reducing the surface tension of water. Their pore-system stabilising effect is based either on their chemical or electrochemical properties (Pigeon and Pleau 1995).

Maintaining an appropriate pore system from the viewpoint of frost resistance until the concrete hardens has proven problematic. For instance, the transportation and placing of the concrete mix affect air content, whereby the porosity of the finished structure often deviates from that of a test specimen taken from the same batch during mixing (Neville 1995, Kuosa and Vesikari 2000).

Air-entraining of Finnish concrete facade and balcony structures was introduced only at the end of the 1970's. The frost resistance of earlier so-called non-frost-resistant concretes was determined especially by the water-cement ratio (Pentti et al. 1998). A low ratio results in higher strength and density, which lower the water absorption rate of concrete and reduce the total amount of freezing water. Old non-frost-resistant concrete structures have in many instances been able to resist frost attacks when the concrete has been of adequate quality.

2.2.2 Formation of late ettringite

The mineral ettringite only seldom occurs in nature, but it is an important hydration product of Portland cement that impacts the development of concrete strength in the short-term and concrete's longer-term stability (Clark et al. 2008). Late ettringite is formed by a chemical reaction of sulphate minerals in hardened cement paste. In concrete that is still in the setting phase, ettringite may also form in the pore water. An ettringite reaction involves a strong increase in the volume of reaction products or expansion. The volume of crystallised solid ettringite may increase 130-140% compared to the volume of the raw materials (Deng and Tang 1994).

An ettringite reaction is normally caused by excessive heat curing of concrete during hardening (Clark et al. 2008, Escadeillas et al. 2007). Thus, an ettringite reaction is most likely in the case of panel types subjected to strong heat curing. They typically include balcony elements.

Secondary void filling

The exposure of a structure to prolonged and high moisture stress is also a requirement for the initiation of a late ettringite reaction (Escadeillas et al. 2007). In the initial stage, ettringite formation is not generally harmful to a concrete structure. The crystallising becomes a problem if it progresses so far that the protective pores of the concrete start to fill up, which reduces frost resistance of the concrete.

The forming ettringite mineral crystallises on the walls of air-filled pores, whereby the volume of protective pores decreases weakening the frost resistance of concrete (Stark and Bollmann 1999). Thus, an ettringite reaction may lead to degradation of concrete either as a result of frost damage when the volume of air-filled pores diminishes or cracks in the concrete due to pressure from the filling of pores with ettringite. According to Deng and Tang (1994), the theoretical pressure from ettringite crystallation in concrete made of Ordinary Portland Cement (OPC) may be up to 55.5 MPa.

The disintegration caused by an ettringite reaction looks much like normal frost damage. The ettringite crystallised on the walls of pores can be detected microscopically on a thin section prepared from the concrete (Pentti et al. 1998).

2.2.3 Alkali-aggregate reaction

The alkali-aggregate reaction (AAR) is an expansion reaction of the aggregate of concrete caused by the alkalinity of hydrated cement, which may disintegrate concrete. The existence of AAR was first discovered in the 1940’s, and it is generally divided into three types according to the reacting aggregate: alkali-silicate reaction, alkali-carbonate reaction and alkali-silica reaction (Gjørv 2009). All AARs require reactive aggregate, a sufficient amount of alkali ions in the hydrated cement, and a minimum relative humidity of concrete of 80% (Punkki and Suominen 1994).

For the alkali-silicate and the alkali-silica reaction to take place, the pore water must contain dissolved sodium (Na_2O) and potassium (K_2O) alkalis, and the aggregate must contain minerals that have low resistance to alkalinity. The gel produced by the reaction absorbs much water from its surroundings, which causes its volume to grow leading to internal pressure within the pore system. As the building pressure exceeds the tensile strength of concrete, cracks form in the concrete structure allowing the relatively soft gel to extrude through them (Neville 1995).

The alkali-carbonate reaction is effected by the alkalinity of some limestones and cement and produces a swelling clay-like substance. The gel that forms at high humidity swells about 4% by volume creating pressure within the pore system of the concrete. The cracking of concrete generates a cracking pattern and leads to a loss of bonds between the aggregate and the cement paste (Neville 1995).

AAR generally means slow deterioration of concrete. Degradation rate is influenced by prevailing conditions as well as the quality of aggregate and cement. In the case of silicon-containing rocks, AAR develops sooner, in 2-5 years, whereas with slower reacting rocks like sandstone and limestone the reaction may take 10-20 years to develop. AAR has been reported to occur also with highly stable rocks such as granite, quartzite and sandstone (Gjørv 2009). With blended cements like BFS and PFA, AAR is less common since fewer reacting alkalis are generally involved than with OPC (Punkki and Suominen 1994).

A concrete structure suffering from AAR typically exhibits discolouration due to surface moisture, irregular pattern cracking, swelling and oozing of a gel-like reaction product from the cracks (Neville 1995). The damage from AAR resembles the cracking caused by frost attack and often coincide with it (Punkki and Suominen 1994). The most significant difference between AAR and frost damage is the pattern of cracking, which in the case of frost damage is most intensive close to the outer surface and loses intensiveness with depth. AAR cracking begins deeper inside the concrete and produces a more regular cracking pattern across the entire concrete structure (Pyy and Holt 2010).

In Central Europe and Scandinavia, AAR typically occurs in massive concrete structures like bridges and dams (Punkki and Suominen 1994). Damage due to AAR in Finnish concrete facade and balcony structures has not been reported. The reason could be e.g. the similarity and coincidence of the damage with damage caused by frost attack and the similar repair methods of both degradation mechanisms. It has not been necessary to make a detailed analysis between frost damage and AAR to select the repair method.

2.2.4 Conclusions about the disintegration phenomena affecting concrete

Frost damage is clearly the most significant degradation phenomenon affecting concrete facades and balconies in Finland. Other degradation phenomena are much more rare, but may well occur in singular cases. The visible damage due to various degradation phenomena is quite similar, which means that the determination of the more exact cause of degradation requires laboratory analyses.

It should be noted that the above-described frost damage does not occur merely because the concrete is not frost resistant (according to the code). The frost stress level and moisture stress must also be sufficiently high, and the structure has to be such that moisture stress produces a high moisture content in the concrete as it freezes.

The occurrence of damage is influenced by the stress conditions of the facade in addition to the quality of concrete. The coastal areas of southern Finland, where driving rains are more common during the winter period than inland, suffer more from frost damage. Driving rain stress is heaviest in open areas and the top sections and corners of tall buildings. The frost stress on a structure is also influenced by structural factors, such as (Pentti et al. 1998):

- performance of the structure's connections and details, such as joints, flashings, projections, etc. since they direct the flow of rainwater
- performance and leaks in eaves gutters and downpipes, etc.
- heat flow through the structure, which dries the structure
- surface treatments, which affect absorption of water and its movement on the surface as well as the evaporation of moisture from the structure
- waterproofing of balconies and its condition, and functioning of drainage.

Water that enters the insulation cavity of a wall through poorly performing or damaged structures and joints is pulled down by gravity, which may cause a significant increase in local frost stress on the concrete exterior wall. Moisture stress is also increased by so-called diffusion moisture that has condensed from indoor air on the structure in winter. The structure dries slowly since functioning ventilation and water drainage routes generally do not exist. Moisture must exit slowly through the outer layer, which increases the level of moisture stress it causes, and thereby also frost stress.

2.3 Other deterioration requiring repair of a concrete facade or balcony

The damage to concrete and embedded reinforcements as well as the factors causing the damage dealt with in the foregoing are the actual degradation mechanisms of concrete and concrete structures. The degradation mechanisms investigated below are either results of the frost attack of concrete or corrosion of reinforcements, or may contribute to the occurrence or propagation of these deterioration phenomena.

2.3.1 Weakening of different fasteners or ties of structural members

As earlier stated, the corrosion protection of concrete steels is based on the high alkalinity of concrete. Structural members, such as facade panels or balconies, are usually attached to the building frame by different steel fasteners, which are normally protected by grouting. Carbonation of concrete allows the fasteners to corrode unless they are made of stainless steel. Corrosion can be very slow if the anchoring points are in a relatively dry environment.

Anchoring damage is possible mainly in the case of thin-shell panels, the outer layers of a sandwich construction, balcony parapets and horizontal tying and mounting of the balcony units. If the cover depth of steel trusses of sandwich panels and compaction at the outer layer were inadequate during production, the corrosion of a bar made of non-alloy steel may initiate due to carbonation occurring on the insulation side. The fasteners of thin-shell panels in the insulation cavity may be unprotected and thus exposed to severe corrosion conditions. Any possible concrete protections may have cracked or carbonated due to constraint actions thereby losing their capacity to protect. There are generally few thin-shell panel fasteners, maybe just one per panel for vertical loads, which means that damage to just one may pose an immediate safety risk (Pentti et al. 1998). Welded joints may also be faulty or corroded. Corrosion of the tie bars of anchor plates and degradation of concrete or its chipping by fasteners may also impair the attachment of a panel.

Frost damage of concrete may also weaken the fastening reliability of structural members and the anchoring capacity of steels, fasteners and connecting trusses. Local degradation of concrete around fasteners may thus endanger the usability of the entire structure.

Incipient damage to fasteners and their surroundings cannot usually be detected visually, but requires tearing open structures at least partially (Condition investigation manual for concrete facade panels 2002).

2.3.2 Malfunctioning moisture behaviour of structures

The structural members of facades and balconies are subjected to strong moisture stress due to climate conditions. The moisture flow from indoors may also contribute significantly especially as concerns wet areas. Inadequate moisture behaviour of structures is in itself a major degradation mechanism. The level of the moisture stress of structures also has a significant impact on the activation and propagation of most degradation mechanisms, such as reinforcement corrosion and frost damage of concrete, i.e. the durability of facades and balconies.

Most components of facades and balconies include structural elements or layers intended to control moisture migration, i.e. wetting and drying, so that moisture stress causes as few problems as possible to the use of a structure and the external wall and balcony structures. These structural members and layers include e.g. (Condition investigation manual for concrete facade panels 2002):
- various elastic and mortar joints between panels including window and door joints and joints with other structures, etc.
- structures related to ventilation of structures and drainage of insulation cavities
- eaves structures and different flashings
- various paint and coating treatments of concrete surfaces
- arrangements for draining water from balconies and balcony glazings.

Common shortcomings in the moisture behaviour of facades include the poor driving rain tightness of various connections like panel joints, window sills, eaves flashings and balcony and window joints, the condition and consistency of concrete surface treatment, and inadequate drainage and ventilation of insulation cavities.

The important details of the moisture behaviour of balcony structures are functioning waterproofing of the upper slab surface and its connections to surrounding structures, slopes and drainage arrangements as well as different mortar joints that are often of poor quality and easily lead rain and melt waters inside the structure.

2.3.3 Delamination of tiles and degradation of coatings

The clinker tiles, brick panels and stone slabs used to adorn concrete facades are bonded to the panel surface by concrete. Damage to clinker tiles and stone slabs generally consists of their detaching from the wall panels. Detachment may be the result of e.g. the weakening of bonding due to frost damage of the concrete or the splitting pressure caused by corrosion of the reinforcement steels behind the slabs.

Due to the higher capillary absorption capacity of brick panels, their bonding to concrete is stronger than that of clinker tiles. Sometimes frost damage poses a problem for brick panels. An especially problematic area of brick-panel clad facades is the interface between a painted surface and the brick-panel surface where the topmost panels are subject to strong moisture stress. High moisture content and frost attack lead to frost damage (Pentti et al. 1998).

Peeling of paint from concrete facades is a very common deterioration phenomenon. Organic paints are deteriorated e.g. by ultraviolet and thermal radiation from the sun, which cause chemical changes in the paint coat, strong moisture stress and migration of moisture through the coat, high alkalinity of concrete and accumulation of salts behind the paint coat as the concrete dries, as well as mechanical action on the concrete surface (Pentti et al. 1998). Generally, the deterioration of organic coatings on a concrete substrate involves loss of adhesion of the coating whereby the coating peels off in flakes or sheets.

Deterioration of inorganic coatings is mainly related to the initial formation of adhesion of the coating, and possibly the frost resistance of the coating material. If they are in order, inorganic coatings generally deteriorate slowly due to the wear caused by weather-induced erosion (Pentti et al. 1998).

Damage to a coat of paint is generally only aesthetic and does thus not necessarily indicate the condition of a structure, i.e. the action of other degradation mechanisms on it. Any possible protection provided by paint is, however, lost as it begins to suffer damage. In some instances the protective effect of paint is necessary to slow down the damage to the substrate (Condition investigation manual for concrete facade panels 2002).

2.3.4 Cracking and deformation of concrete

A concrete structure cracks when the actual tensile stress of the structure exceeds the tensile strength of concrete. Tensile stresses can be generated by various factors. These include shrinkages during the plastic and hardening phases, drying shrinkage of hardened concrete and shrinkage differences, external loading of the structure, displacements of supports, temperature changes, frost damage, and internal pressure due to corrosion of reinforcements. Concrete has a low elongation at break of about 0.15 ‰ meaning that prevention of shrinkage generally always leads to cracking (Neville 1995).

The detrimental effects of cracks are proportional to crack width. Crack width is often only measured at the surface of a structure, although it would be important to know how the crack extends inside the structure, and whether it e.g. extends all the way to reinforcing steels. That is, however, difficult to determine in practice without destructive tests. Cracks may have durability and structural and visual impacts. Chlorides or carbon dioxide may penetrate into concrete all the way to the steels through large enough cracks resulting in local corrosion with moisture. Movements of cracks cause fracturing of the finish unless the coating is extremely flexible.

Cracks may form in facade panels already during manufacture and installation due to stresses exerted on them when they are being lifted or transferred or collide against something, etc. Stresses that may cause cracks during the life cycle of a structure include collisions and various restraint actions. Restraint actions are created as trusses and other fasteners attempt to prevent movements of the outer layer in relation to the inner layer or curvature of the outer layer. In practice, detrimental cracks are most often caused by installation period knocks, rigid fasteners or large shrinkages of concrete. Narrow surface cracks always exist on concrete surface, but their impact on the durability and even the appearance of a structure is minor (Pentti et al. 1998).

Balcony structures are completely outdoors, which means that they move with changes in temperature and humidity. The movement is the greater, the larger the structure. For instance, the vertical movement of the upper sections of stacked balconies in relation to the frame is greater than that of lower sections. The movement of balconies can be checked by ties or support beams extending from the frame to the balcony, which may cause very high tensions (Condition investigation manual for concrete facade panels 2002).

Tiles attached to the outer surface of a concrete facade prevent concrete from shrinking, which, on the other hand, is possible on the surface on the side of the thermal insulation. Thus, tiled panels may curve outward due to uneven shrinkage. Curvature is especially pronounced in thin-shell panels that are typically only attached by the corners.

Swelling of concrete due to frost damage may lead to curvature in different directions depending of the location of the frost attack. For instance, should the backing concrete of an exposed-aggregate panel degrade and swell, it will lead to inward curvature. Outer layer movements, offsets at joints joint, etc. are also the result of the yielding of fastenings. Whether the curvature is systematic or localised indicates its cause. Later curvature at points subject to the most stress is likely caused by frost damage.

As a facade panel dries, the outer surface of the outer layer dries before its inner layer, which causes the outer layer to tend to curve inward in the middle and outward at the edges. Curvature is limited by trusses, which at the edges of the panel are subject to pull, and in the middle to compression. The forces generated in the trusses exert a bending moment of the outer layer of the panel, which, again, exerts tensile stress on the outer surface. If the shrinkage tendency of concrete is high, over 0.6 ‰, these flexural tensile stresses easily cause cracks, which the central reinforcement of the outer layer cannot prevent (Neville 1995).

2.4 Available field investigations on durability properties and deterioration of concrete facades or balconies

Laboratory analyses on the material properties and deterioration of concrete are referred to in numerous books and scientific articles and reports. On the other hand, only a few large-scale studies based on samples from actual buildings and field measurements on realised material properties of concrete structures have been conducted.

Domestic literature

Carbonation of concrete. Domestic studies have mainly concentrated on the investigation of carbonation of concrete based on samples drilled from facades and reinforcement cover depth measurements. In the following, the coverage of the samplings of these studies and the key results will be examined.

Concrete facades built in Tampere, Finland, in 1960-1983 were studied in 1988 and again in 1994. The first field investigation analysed 233 concrete core samples drilled from 39 buildings as to carbonation as well as measured the reinforcement cover depths of the targets' facade panels (Mehto et al. 1990). The latter study complemented the former one by drilling from the same buildings four parallel samples next to the previous sampling locations, where possible, and by measuring the carbonation depth of concrete and comparing the values to the corresponding values of the earlier study. A total of 558 samples were taken from 28 buildings in the latter study. The facades of the buildings were exposed-aggregate, painted, and clinker-clad and brick panel-clad concrete (Huopainen 1997). The key observations about the durability and degradation of structures were (Mehto et al. 1990, Huopainen 1997):

- The deviations occurring in the manufacture of concrete panels have been great since the actual thicknesses of the outer layers of the examined panels ranged from 36.3-99.1 mm when the nominal thickness of the structures was 60 mm.
- There was wide variation in concrete carbonation depth both within and between facade types and individual facades.
- The average carbonation coefficient was between 2.5-3.5 mm/√a. Carbonation had been fastest in exposed-aggregate concrete and slowest in clinker-clad and brick panel-clad panels. Painting of concrete surfaces was found to have no effect on carbonation rate.
- Reinforcement cover thicknesses were mostly over 25 mm. In clinker-clad and brick panel-clad panels reinforcement was on average closer to the outer surface being situated near the bottom of panels.
- Microstructure analysis of concrete showed no signs of its frost damage.

The facades of multi-storey residential buildings erected in Helsinki in 1965-1980 and their renovation need and possibilities were studied in field investigations by the National Housing Board in 1992 (Heimala and Punakallio 1993). They involved drilling a total of about 250 concrete core samples from a total of 21 buildings. Besides concrete facades, the investigations also targeted balconies which accounted for about 100 samples. Moreover, about 100 samples had already been taken earlier from the targets, and the laboratory results on them had been available to researchers. The facades of the examined targets were exposed-aggregate, painted, uncoated, and brick panel- and clinker-clad concrete. The most essential observations as to the durability and deterioration of the structures were:

- The deviations occurring in the manufacture of concrete panels had been wide since the actual thicknesses of the outer layers of studied concrete panels typically varied from 25-70 mm when the nominal thickness of a structure had been 60 mm.
- Based on microstructure analyses of concrete, exposed-aggregate concrete is generally non-air-entrained, and the samples generally showed frost damage. Air-entrainment of other facade surface types has also been inadequate, but frost damage was detected in only one target where the paint coat had been defective.
- There was wide variation in concrete carbonation depth within facade types and between individual facades. Carbonation depths of samples varied from 2-25 mm.
- Reinforcement cover thicknesses were mainly above 20 mm. The reinforcements of clinker-clad panels were on average closer to the outer surface near the bottom of panels. The steels immediately behind the clinker tiles had corroded and loosened some clinker tiles.

- The concrete used for balcony structures was for the most part non-air-entrained, yet frost damage occurred only incidentally.
- On the soffits of balcony slabs carbonation had advanced on average 15-20 mm reaching steels at a distance of 15-30 mm from the surface across a wide front.
- Balcony side panels had typically carbonated to a depth of 10-20 mm. The distance of reinforcements from the surface varied from 0-30 mm, the average being 20 mm.

In all of the above studies the buildings were relatively young at the time of investigation: the oldest ones were 28 years old and the youngest ones 5 years old. Considering the age of the buildings, concrete carbonation had on the whole been fast, and no perceptible differences existed between buildings in Tampere and Helsinki. On the other hand, a significant difference in frost damage of concrete in relation to climate conditions is noticeable. In the case of similar structures and facade types, frost damage does occur in Helsinki but not in Tampere, indicating a heavier stress level in southern coastal Finland compared to inland. None of the investigations did, however, analyse factors related to local climate conditions in more detail.

Corrosion rate of reinforcements. Mattila and Pentti (2004) studied the corrosion rate of reinforcements embedded in carbonated concrete under actual natural conditions. Their study monitored for 25 months (1 Dec 2000-31 Dec 2002) the deterioration rate of the facades and balconies of three precast concrete multi-storey residential buildings by 120 sensors installed in the structures. The study determined e.g. the protective effect of three coatings providing moisture protection, balcony glazing and ventilated board-faced balcony side panels compared to unprotected areas. Two of the buildings equipped with sensors were located in Tampere and one in Espoo. In all buildings the sensors were on the southern facade in the solid panel of the uppermost residential floor and the two uppermost balconies of the stack closest to it. The key observations relating to the corrosion rate of reinforcements were (Mattila and Pentti 2004):

- In the case of reference sensors coated with open pore paint, the corrosion rate of reinforcements varied according to the amount of rain received by the facade, so that corrosion rate of the month with the most rain was about five-fold compared to a month of little rain.
- The corrosion rate of reinforcements decelerates significantly as temperature drops under 0 °C. Corrosion does not, however, stop immediately, but may occur to some extent until about -15 °C. The corrosion rate of reinforcements is highest when temperature is 0-10 °C.
- Various protective surface treatments retarded reinforcement corrosion by an average of 75-79% depending on the product used.

A significant finding of the study was the connection between rain stress level and corrosion rate, which is not generally considered in the service-life design of concrete structures. The period of measurement included a year that was more rainy than the long-term average (2001) and one drier than average year (2002). Another key finding was the significant retardation of the corrosion rate caused by finishes that repel rainwater. The effect was nearly the same with all studied finishes. The protection of a finish does, however, end if the coat of paint is damaged.

The propagation of corrosion of reinforcements as temperature falls under 0 °C is the result of the slow freezing of the water in the pore system of concrete due e.g. to the salts dissolved in pore water, as discussed before.

Due to the different locations of sensors in facades and balconies, the effect of frost attack on different facades as well as the action on the lower floors of the building could not be determined in this study.

Foreign literature

Foreign literature, scientific articles and conference publications include numerous studies on concrete structures in contact with seawater and analyses of samples drilled from bridge structures. Typically such studies have investigated the penetration of chlorides into concrete and the initiation of chloride-induced corrosion, corrosion rate and the impacts of corrosion on the service life of a structure. An especially large number of studies have focussed on seawater interfaces and splash-water areas, where the evaporation of water allows chlorides to accumulate locally whereby chloride-contents may become remarkably high. However, it is not sensible to summarise these studies further here since chloride corrosion induced by seawater is not relevant when investigating the deterioration of concrete facades and balconies.

In the case of individual landmarks of modern architecture, such as the Torre Velasca in Milan (Bertolini et al. 2009), the material properties of concrete and reasons for the deterioration of structures have been reported. On the other hand, only a few broader studies on the facades or concrete balconies of existing buildings based on sampling and field investigations have been reported. In the following, two studies based on field investigations of several buildings are summarised.

According to Richardson (1990), the surface type and rain stress received by a concrete facade have a significant impact on its carbonation depth and rate. The findings are based on an analysis of 130 concrete samples drilled from Irish buildings 5-114 years old at the time of investigation. About half of the samples had been drilled from buildings completed in 1955 or later. The key findings were:

- Over 60% of the measured carbonation depths were under 5 mm.
- Average carbonation coefficients were in the 1.4-1.7 range. The carbonation rate of facades exposed to weather was generally lower than that of facades protected from rain and sleet.
- Higher carbonation rates occurred in cities and suburbs than in coastal areas and the countryside.
- In coastal conditions chloride penetration in concrete was found to constitute a higher risk of corrosion than carbonation of concrete.

The Irish climate is mild but rainy compared to Finland and provides favourable conditions for corrosion of reinforcements the year round. The carbonation rates appear lower, also on facades protected from rain, than those measured from Finnish facades (Mehto et al. 1990, Huopainen 1997). In cities and suburbs the carbon dioxide contents of air may be higher locally and occasionally due to heavy traffic, which for its parts explains the differences in carbonation rates.

According to Tilly (2007), corrosion of reinforcements is the biggest single factor damaging concrete structures in Europe. The findings are based on a total of 230 cases. Case material was collected to cover broadly different climatic and service conditions from Greece to Finland. The material included 77 precast concrete buildings, 75 bridges, 36 dams, 12 power plants and 8 multi-level car parks. The remaining 22 cases divided into several smaller groups of different concrete structures. At the time of investigation 60% of the structures were 20-50 years old. In their case, the primary factors causing damage were reported to be corrosion of reinforcements (55%), frost damage (10%), cracking of concrete (10%), AAR (5%), and faulty construction (20%). In many cases more than one type of deterioration was involved, for example, there were several instances of a single structure being cited as suffering from corrosion, frost damage and cracking simultaneously.

Tilly's study made a rough summary of the causes of concrete structure degradation. It does not indicate the numbers of samples drilled from structures or the analyses they underwent. Climate conditions as well as the quality of concrete used for facades and balconies vary a lot between European countries. Therefore, it is interesting to note that reinforcement corrosion has become clearly the most significant degradation mechanism. The study does not reveal in detail whether corrosion results from concrete carbonation or the chlorides penetrating into concrete due to the use of deicing salt, but according to Tilly (2007), faulty construction has contributed significantly to the extent of damage. It includes e.g. incorrectly placed concrete resulting in voids, honeycombing, low reinforcement cover depths, and cracking of concrete. About 60% of the faulty construction instances became evident only several years after the completion of a structure as deterioration had already begun.

2.5 Summary of Chapter 2

The degradation mechanisms of concrete structures and various factors affecting them have been studied widely for a long time, which has made them well known. The following conclusions can be drawn based on the above examination:

- Corrosion of steels in concrete has been studied mainly as chloride-induced corrosion; corrosion due to concrete carbonation has been studied considerably less.
- The factors affecting concrete carbonation are quite well known as are the factors determining corrosion rate. Corrosion rate is clearly correlated with the moisture-content of concrete.
- Degradation of concrete may be the result of frost attack, ettringite reaction or alkali-aggregate reaction. For the above degradation mechanisms to function, the concrete structure must be exposed to a high moisture content.
- Remarkably few research results on the impact of facade-panel or balcony structures on actual service life and damage sustained under natural conditions have been published.
- Various degradation mechanisms have primarily been studied in laboratory conditions using accelerated stress tests. Studies based on broad field investigations have been conducted mainly for pier and bridge structures in contact with seawater. Only isolated broad field investigations on the concrete facades and balconies of the existing building stock have been reported.

3 RESEARCH QUESTIONS AND METHODS

3.1 Research questions

The core content of this study consists of the durability properties of concrete panels realised in actual construction and how the degradation of buildings made of different types of precast concrete panels has occurred in actual natural conditions. The research has sought answers to the research questions posed below.

What are the realised durability properties of concrete facades and balconies?

- Can the required reinforcement cover depths of codes and design standards be achieved with the realised structural thicknesses?
- What is the realised capillarity of concrete?
- Has the compaction of concrete been successful?
- Are the realised reinforcement cover depths in line with requirements?
- Are the steel fasteners of the outer layer of facade and balcony panels always well protected against corrosion or made of stainless steel?
- Do excessive chloride contents from the viewpoint of durability occur in concrete?
- How quickly does concrete carbonation propagate in different panel and surface types?
- How does the capillarity of concrete affect propagation of carbonation?
- Are there regional differences between durability properties of concrete?

What has been the significance of concrete structures' durability guidelines for their realised durability properties?

- Have the reinforcement cover depth requirements had an impact on the actual location of reinforcements?
- Have stainless steel reinforcements been used?
- Has protective air-entrainment succeeded in keeping with requirements?
- Has the heightening of the compressive strength of concrete at the end of the 1980's been reflected in the durability properties and deterioration of concrete?

What types of damage occur in concrete facades and balconies?

- Do safety defects due to damaged fasteners of outer layers of facades and balcony panels exist?
- Is the damage the same with all panel and surface types?
- Does visible reinforcement corrosion and frost damage of concrete appear equally on all facades?
- Are there regional differences in the degree of damage?
- Has the amount of thermal insulation affected the occurrence of damage?
- Does filling of pore structure or alkali-aggregate reaction occur in precast panels used in facades and balconies?

How have the location of a building and structural and climatic factors affected the observed damage to concrete facades and balconies?

- Has the amount of rain and sleet received by a structure had an effect on the propagation of carbonation of concrete?
- Has the amount of rain and sleet received by a structure had an effect on corrosion of reinforcements?
- Has the amount of rain and sleet received by a structure had an impact on frost damage of concrete?
- Has the amount of rain and sleet received by a structure had an impact on the occurrence of damage on different facades of a building?

- Has the received amount of rain and sleet been concentrated on certain areas or sides of building?
- Is there regional variation in the freeze-thaw cycles occurring in outdoor air?

3.2 Methods

The factors affecting the material properties of concrete, and the degradation mechanisms of concrete and concrete structures and factors affecting them, as well as earlier conducted studies and surveys based on extensive field investigations have been examined in a literature study.

The size of the existing precast multi-storey building stock and the number of different facade surface types have been determined on the basis of data of Statistics Finland on suburbs of five large cities (total amount) and by random sampling (facade surface type distribution).

The material compiled from condition investigation reports and meteorological observations is quantitative by nature. The collected research material consists of actual measured values. Various distributions of the realised durability properties of concrete structures have been produced and compared to the durability guidelines for each period and distributions of actual deterioration and prevailing climate conditions. This study will not create a model depicting the deterioration of concrete structures, but analyses the distributions of realised material properties, distributions of damage, and the impact of different material properties and climate conditions on actual deterioration.

4 RESEARCH MATERIAL

The research material consists of the database on material properties and deterioration of existing Finnish concrete facade panels and balconies built up between 1960 and 1996, and weather observations since 1961 by the Finnish Meteorological Institute (FMI). All material properties and deterioration information are based on collected condition investigation reports.

4.1 Condition investigation

The condition investigation of concrete facades and balconies is a systematic method to determine the condition of an existing structure, future propagation of deterioration and recommended repair measures. The content of a condition investigation is based on the properties of the target building, such as used structure types, materials, environmental stress conditions, already visible damage, and the goals set for the investigation. The contents of a condition investigation must always be planned separately for each target (Condition investigation manual for concrete facade panels 2002).

In a condition investigation, the condition and performance of a structural member or group of them is determined systematically in terms of different degradation mechanisms using various research methods including review of design documents, visual inspection of the target, various field measurements and surveys as well as sampling and laboratory analyses. The aim is to find out the causes, extent and impacts of the damage existing at the time of investigation as well as to anticipate future damage at the stage when visible damage does not yet exist. The data are collected as samples while the properties and condition of the structure vary in its different parts. Thus, the condition investigation of an old structure always involves uncertainty, which is why an effort is made to reduce it by using parallel methods in investigating degradation mechanisms and by collecting data from as many sources as possible.

4.2 The database

The database consists of data derived from condition investigation reports on concrete facades and balconies. Condition investigation reports have been collected from owners of rental residential properties and consulting companies conducting condition investigations as well as the Laboratory of Structural Engineering at Tampere University of Technology. It contains condition-investigation report data on 422 targets. A condition investigation report on a single target comprises the condition investigation data of 1-30 buildings. The database contains the investigation results on a total of 947 precast concrete buildings since many investigations looked at several buildings at the same time. Thus, a single condition-investigation report includes the data on an average of 2.2 buildings. The condition investigation targets of the database are presented in Appendix 1. The condition investigations have been implemented largely according to the presented Condition investigation manual for concrete facade panels and are thus mutually comparable.

The targets in the database were built in 1960-1996; mostly in the 1970’s and early 1980’s. They have been divided into three different climatic regions by geographic location: the southern coastal area, inland and northern Finland. Moreover, the metropolitan area has been examined as a separate entity due to its large building stock (see Fig 4.1). Climatically Espoo and Helsinki belong to the southern coastal area, whereas Vantaa and the rest of the metropolitan area are inland.

Inland areas constitute the largest examined area, which also has a lot of precast concrete buildings. Northern Finland has considerably fewer such buildings than the rest of Finland, which means that relatively few condition investigation reports on its buildings were available. These buildings have been made part of the inland building stock in order to be able to include also these targets of condition investigation in the study.

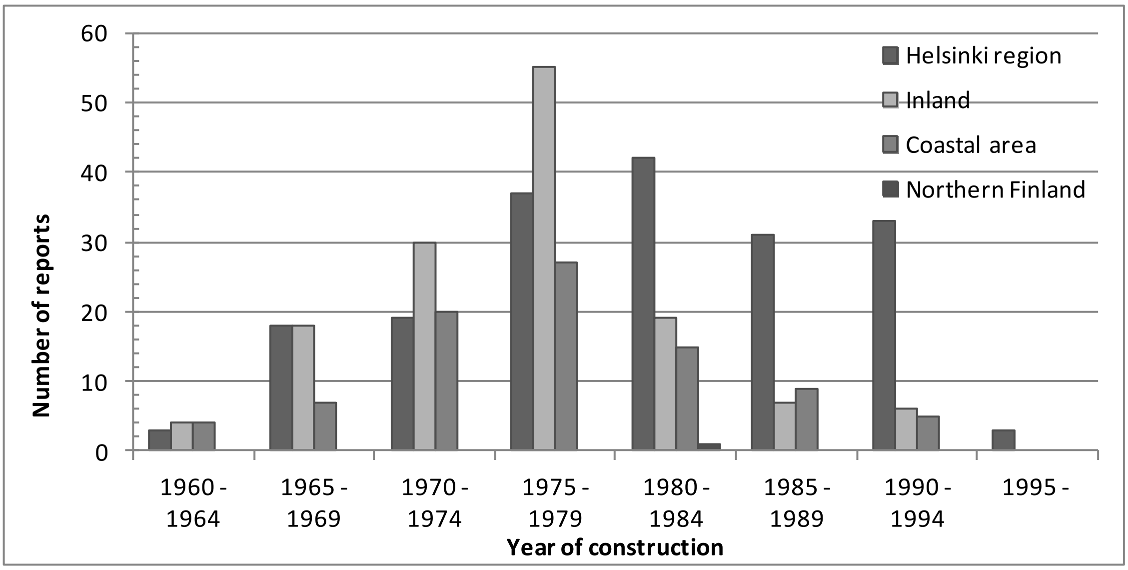


Fig. 4.1 Geographic year-built distribution of condition investigation targets of database.

4.2.1 Surface types of concrete panels

The typical facade types of the time of construction, exposed aggregate and brushed painted facades, dominate among the buildings of the database. According to the facade-type distribution of Figure 4.2, painted form-finish panels were also plentiful. However, they were just parts of other buildings, whose primary facade type was e.g. exposed aggregate or some other widely used type. Painted form-finish panels are generally facade panels of above-ground basement storeys or stair enclosures. The database contains many of them, which is why they have been made here into a separate group.

An actual building typically has facades of at least two types in accordance with the above example. It is also possible that a single panel has two different surface types. A typical example is a clinker-clad panel with unpainted form-finish concrete at the sides. In such cases the degradation of a structure is to be examined separately according to the degradation mechanisms of several facade surface types in order to get the total picture. Consequently, conclusions about the properties and deterioration of a facade type cannot be made on the basis of another type.

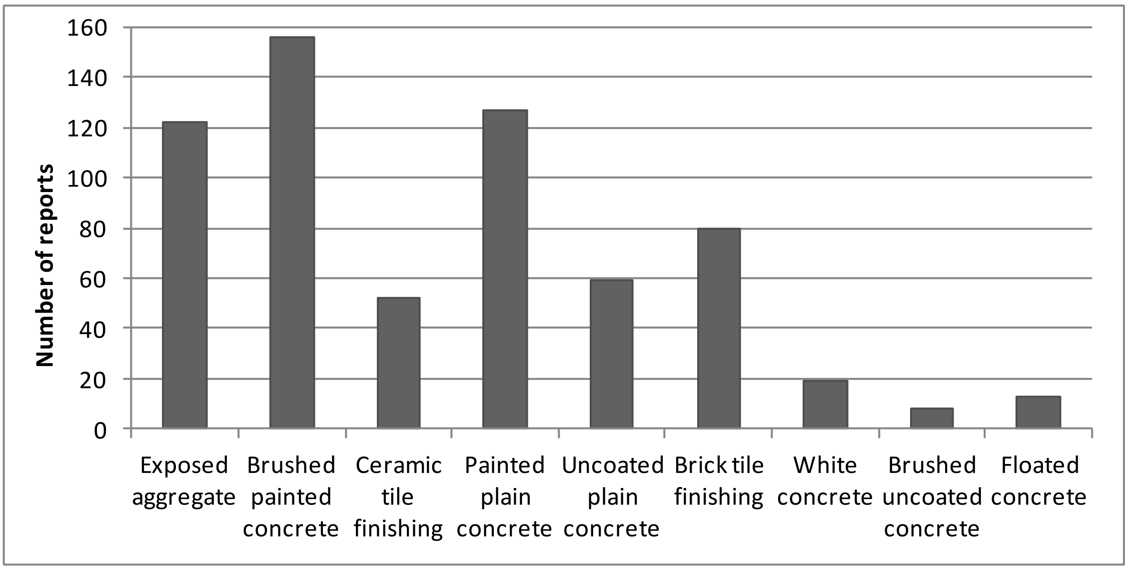


Fig. 4.2 Surface-type distribution of targets of condition investigation in database.

The situation with balconies is more straightforward than with facades in that practically the only surface type used is painted form-finish concrete. The structure and surface type of balconies has remained the same throughout the precast construction period. The database presents balconies by structures: side panels, slabs and parapet panels.

4.2.2 The condition investigation report as research material

Condition investigation reports contain an abundance of detailed information on individual condition investigation targets. They hold all the research data of a condition investigation, all visual observations, and the basic data on a building. The research material includes all results of laboratory tests, such as protective-pore and tensile strength tests, and measurement data on samples and field measurements including reinforcement cover depth measurements. Often also a thin-section analysis report and all the laboratory research results on joint sealants are appended to the report.

Besides research results, the report also includes much information on the building itself, such as year-built, used structure types, and surface types of facades and balconies. The more recent the condition report, the more photo material it generally includes on individual damaged areas as well as general views of the target, which provide a lot of general information missing from the text. Each report also includes the condition investigators evaluation and grounds for suitable repair alternatives.

The main contents of condition investigation reports are generally the same, but despite the uniform guidelines for condition investigations (Condition investigation manual for concrete facade panels 2002), the contents of the reports vary slightly just because of the differences between targets and the reporting practices of engineering firms conducting the investigations. In order to be able to compare the information of different reports, the information had first to be compiled in comparable electronic form. The majority of the condition investigation information is in the form of a paper report written by the investigators, which means that each report has been processed separately for data transfer. Individual pieces of information have been picked manually and entered in tabular data sheets.

Measured data

Condition investigation reports present measured data of both field investigations and laboratory analyses. Data collected in field investigations consists of reinforcement cover depth measurements and thermal insulation layer thickness measurements. The results of the former are generally recorded in reports as distributions, which show the shares of cover depths at 5 mm intervals.

Most measured data of condition investigation reports derives from measurements under laboratory conditions including thickness of the examined structure type (length of drilled core cylinder), diameter and cover depths of reinforcing steels in sample, carbonation depth of concrete (average and maximum), acid-soluble chloride content of concrete, tensile strength of concrete, maximum grain size of concrete aggregate, water absorbing capacity of concrete, and protective pore ratio.

The amounts of measured data and other analyses and observations in the database are presented in Table 4.1 separately for facades and balconies.

Table 4.1 Amounts of different individual measurements and tests and other analyses and observations concerning facades and balconies compiled in the database.

Laboratory tests	**Facades**	**Balconies**
Sample size [no.]	3 868	2 869
Thickness of structure [no.]	3 603	2 351
Reinforcing steel diameter data [no.]	2 174	1 218
Reinforcement cover depth from sample [no.]	4 797	3 098
Concrete carbonation depth [no.]	3 464	2 586
Chloride content of concrete [no.]	496	580
Tensile strength of concrete [no.]	1 776	1 666
Maximum grain size of aggregate [no.]	3 350	2 504
Water absorbing capacity of concrete [no.]	2 311	1 928
Protective pore ratio of concrete [no.]	2 365	1 943
Thin-section analyses [no.]	1 462	973
Field measurements		
Reinforcement cover depth measurements [no.]	249 693	133 514
Structure support method [no. of targets]	509	847
Support/horizontal tying material [no. of targets]	399	215
Parapet fastening method [no. of targets]		212
Thickness of thermal insulation layer [no.]	2 161	

The carbonation depth measured from samples and age of building were used to calculate the carbonation coefficient k [mm/√a], which is an indicator of carbonation rate according to Formula 2.2.

Verbal data

Numerical classification of data and observations presented in verbal form in condition investigation reports has been created in order to make various reports consistent with each other and to allow statistical processing. The classification was accomplished with the help of the Condition investigation manual for concrete facade panels (2002) and condition investigation reports written by TUT researchers. Classification of thin-section analyses is based on the four-tier classification system presented in Koskiahde (2004), see Table 4.2.

Table 4.2 Classification of concrete degradation mechanisms observed in thin-section analysis compiled in the database (Koskiahde 2004).

	Class/designation			
	1	2	3	4
Cracking indicating frost damage	None.	Incipient. Crack widths < 0.01 mm and lengths < 10 mm.	Frequent. Crack widths 0.01-0.1 mm and lengths ≥ 10 mm. Frequency < 0.25 cracks/mm and < 50 % of aggregate loosened.	Severe. Many cracks > 0.1 mm wide and > 25 mm long. Frequency ≥ 0.25 cracks/mm or ≥ 50 % of aggregate loosened.
Air pores	Frost-resistant concrete, pore spacing ≤ 0.25 mm.	Partially deficient air-entraining, pore spacing 0.25-0.40 mm.	Unsuccessful air-entrainment, pore spacing ≥ 0.40 mm or no intentional air-entrainment, but an abundance of small air pores (Ø < 1 mm)	No air-entrainment¸ occasional air pores detectable. Typical air content ≤ 2 %
Degree of pore filling	None.	Incipient filling, small individual crystals.	Continuous, circular filling. Thickness of deposit 0.01-0.05 mm	Wide spread filling of pores, systematic deposit > 0.05 mm

Possible cracking and degree of compaction of cast concrete are evaluated visually from the surface of core cylinders drilled in field investigations. The whole-number grading scale is 1-5 (substandard-poor-average-good-outstanding). Detailed definition criteria are presented in Appendix 2.

The type of thermal insulation (mineral wool/EPS/something else) and its condition (clean/dirty/moulded) and moisture content (wet/dry) are evaluated visually in connection with core drillings.

Visible corrosion and frost damage and flaking of paint coat are evaluated on a three-tier grading scale (none/local/extensive).

The elevation of samples drilled from a building is documented by the number of the floor in question and horizontal position by a marking on the side or in the middle. In the case of facades, edge area refers to a maximum distance of 5 metres from a corner of the building or to the edge of the outermost balcony, if the balcony is closer than five metres to the edge. Only the outermost balcony is considered to be in an edge area, the rest in are in central area. Orientation of the samples is documented to the level of intermediate compass points while the geographic location of the target is documented by post code.

In the case of facades, the fastening of the outer layer to the inner layer is classified as being implemented with stainless steel trusses or something else including a description of the fastening method and/or used material. Data on the elastic sealants for facades are recorded as in Table 4.3. Any asbestos-containing paint possibly used on facades and balconies is classified as does not contain asbestos/contains asbestos.

Table 4.3 Classification of observations on elastic joints between facade panels collected in the database.

	Designation				
	1	**2**	**3**	**4**	**5**
Renewal of joints	Done	Not done			
Condition of joints	Sound	Cracked	Detached from substrate	Cracked and detached from substrate	
Compression of joints	None	Horizontal	Horizontal and vertical		
Ventilation of joints	Pipes	Boxes	Other	Pipes and boxes	Non-existent
Lead compounds	None	Yes			
PCB	None	Yes			

The balcony support method is documented as: stacked/hung/cantilever. The material for horizontal tying of a balcony is classified as: stainless/galvanised/unprotected steel. The balcony parapet fastening methods are divided in three categories: cast-in/structural steel cantilevers and welded joint/structural steel cantilevers and bolted joint). Balcony drainage methods divide into three categories: via pipe through parapet, drainage through gap between parapet and slab and via pipe through slab.

Waterproofing of the balcony slab is classified either as existing or non-existing. Moreover, the treatment of the outer surfaces of balcony parapets is classified as: painted/exposed aggregate/clinker-clad/unpainted/brick panel-clad/brushed/white concrete.

4.3 Meteorological observations

The meteorological observations have been compiled from observations of the Finnish Meteorological Institute (FMI) starting from 1961. Climate conditions vary across Finland, which is why observation data have been collected from several localities. The majority of the Finnish multi-storey residential building stock is located in the southern coastal areas and elsewhere in southern Finland. Based on that fact and FMI's meteorological observatory network, the following stations were selected:

- Turku Airport (on the southern coast)
- Helsinki Kaisaniemi (on the southern coast)
- Helsinki-Vantaa Airport (in southern coastal area)
- Jyväskylä Airport (inland)
- Oulu Airport (only wind speed and direction during rain in northern coastal area)
- Rovaniemi Airport (only wind speed and direction during rain and sleet in northern inland area).

Besides total monthly rainfall amounts, snowfall in the form of water or sleet has been classified since only liquid rain can be absorbed by capillary suction into the pore system of a porous material. Precipitation days have been recorded into five categories by amount so that 0.1 mm represents lightest precipitation. The other categories are ≥ 1 mm, ≥ 3 mm, ≥ 5 mm and ≥ 10 mm.

Wind directions and velocities during rain or sleet have been compiled as part of precipitation data, as well as continuous September-April and corresponding yearly rain-wind observations that also include summer rains. Such observations have been organised by periods of five observation years.

Annual freeze-thaw cycles for the periods have been compiled from the beginning of September to the end of April. Freeze-thaw cycles have been compiled for temperatures < 0 °C, < -2 °C, < -5 °C and < -10 °C. The cycles have also been compiled in cases where rain or sleet has occurred for three days at the most before the beginning of the frost period. Moreover, the numbers of frost periods according to length have also been compiled.

4.4 Evaluation of database

Condition investigation reports have been collected into the database from residential rental property owners, who typically own a large stock of multi-storey buildings, as well as engineering firms conducting condition investigations and the Laboratory of Structural Engineering at TUT. The assessed targets represent normal Finnish blocks of flats. The targets were not specifically selected for this study, but were compiled from the archives of the mentioned organisations and do thus represent quite well the Finnish precast concrete building stock.

According to Statistics Finland (2010), rental dwellings account for 30% of the entire dwelling stock. Thus, most of the Finnish precast residential building stock is owned by private individuals through housing companies. In the database compiled for this research the situation is the opposite: 68 of the target buildings are owned by housing companies and 358 by rental housing companies. However, no data exists about different panels as to durability having been used in the rental buildings. The same norms apply to all Finnish buildings as evidenced by the design documents.

The majority of the condition investigations of the database have been conducted either by the researchers of the Institute of Structural Engineering at TUT or condition investigators of Lauri Mehto Consulting Engineers. Investigations by other engineering firms constitute a small minority. This may, in principle, have significance as to the thoroughness (scope) of condition investigations and the conclusions drawn on the basis of observations and measurements in case different companies have different repair recommendation philosophies.

The quality of made observations and performed measurements depends primarily on the professional skill of the investigators and used research methods and tools. Generally, condition investigations are done according to the Condition investigation manual for concrete facade panels 2002 meaning that in this respect all actors use uniform methods, which makes for quite reliable measurements and field observations.

Visible damage

Of the facades of all buildings in the database, 59% suffered from visible local or extensive corrosion damage at the time of investigation while 43% suffered from frost damage. These observations covered 811 buildings. On the other hand, the respective shares of buildings showing no visible damage at the time of investigation were 41% and 57%.

Thus, both buildings showing visual damage and a considerable share of buildings appearing undamaged were subjected to condition investigations. On the other hand, there may have been visual damage to the facades of a building but not to its balconies, or vice versa. Interviews with condition investigators and property owners indicate that the entire building stock is subject to continuous condition investigations independent of the existence of visible damage. The most determining factor is building age. The typical subject of a condition investigation is a 20-23 year old building (Lahdensivu and Varjonen 2011). On that basis, it can be assumed that the buildings of the database represent quite well the average Finnish precast concrete building stock.

Degradation mechanisms are independent of each other

The most prevalent degradation mechanisms of concrete structures – reinforcement corrosion and frost damage of concrete – are independent of each other. Targets showing incipient or extensive frost damage, or whose frost resistance is low, may suffer from only slight reinforcement corrosion, or none at all.

The various buildings and structural members of the database from various parts of Finland are affected more or less by corrosion of reinforcement and frost damage of concrete depending on the local stress level. The investigations did not reveal any systematic cause for the occurrence of the damage in an area due to material or structural properties. On that basis, the material can be considered independent.

Reinforcement cover depth measurements

The share of under 5 mm cover depths is remarkably small in the reinforcement cover depth distribution. In practice, under 5 mm cover depths are not recorded in field investigations since many meter types do not record < 5 readings, which are entered as 5 mm. Another, perhaps a more important reason, is visible corrosion damage that is first noticed where cover depths are smallest. As stated earlier, cover depths at these spots are not measured in condition investigations, but the amount of visible corrosion damage is assessed either by the naked eye or is measured in so-called linear metres.

The reinforcement cover depth distribution is inaccurate in the 0-4 mm range since the depths are not measured for practical reasons. The red column in Figure 4.3 is an estimation of the share of the smallest cover depths of balcony side panels. The share of the smallest cover depths in the case of different panel types has been studied based on a total of 56 condition investigations conducted by TUT since an abundance of photo material related to them is available. Based on the study, the share of the smallest 0-4 mm cover depths is typically smaller in facade panels than that of 5-9 mm cover depths. The smallest cover depths of facade elements, according to the photos taken during condition investigations, occurred mostly at the edge bars. The share of the smallest 0-4 mm cover depths is greater in balcony structures than facade panels, but never larger than the share of 5-9 mm cover depths.

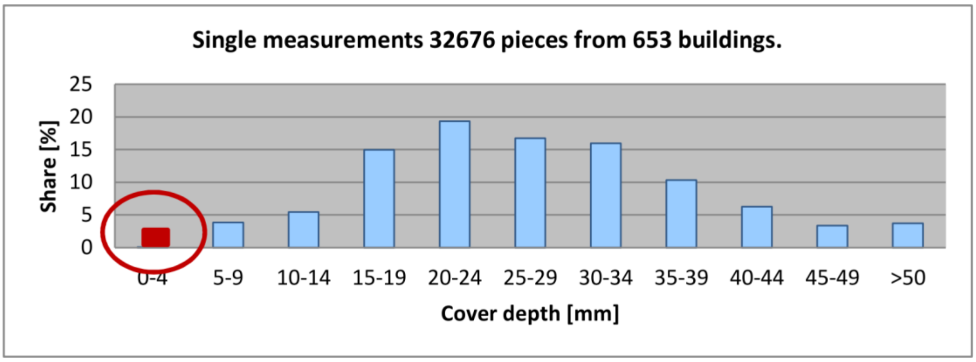


Fig. 4.3 *Cover depth distribution of balcony side panel reinforcements of entire database. The red column depicts the estimated share of the smallest cover depths based on visible corrosion damage.*

In the case of panels including finishing products, the thickness of the brick panel or clinker tile has quite often not been deducted from the cover depth value, or has been deducted only is some measurements. Such an inconsistency was found in the case of brick panel-clad facades, which was yet easy to determine on the basis of too small cover depth values (< 20 mm).

In practice, the smallest unmeasured cover depths are considered to equal visible corrosion damage when selecting the repair method.

Facade surface-type and year-built distribution

The number of available condition investigation reports varies by the age of the building stock: most investigations were conducted on buildings completed between the late 1960's and the late 1970's. The database includes especially few buildings from the early years of prefabricated construction in 1960-1964 since only a small number of such buildings exist. Similarly, buildings completed in 1995 or later are quite poorly represented. This is probably due the fact that renovation has focused on the older building stock, which means that condition investigation of more recent building stock has hardly begun.

According to Statistics Finland, housing construction was liveliest in Finland from the 1960's to the end of the 1980's. By far the most multi-storey residential buildings were completed in the 1970's, see Fig 4.4 (Statistics Finland 2010). The usefulness and reliability of the accumulated database are at their best when studying buildings completed in 1965-1994. The database represents broadly the most typical facade surface types used in that period, see Fig. 4.5.

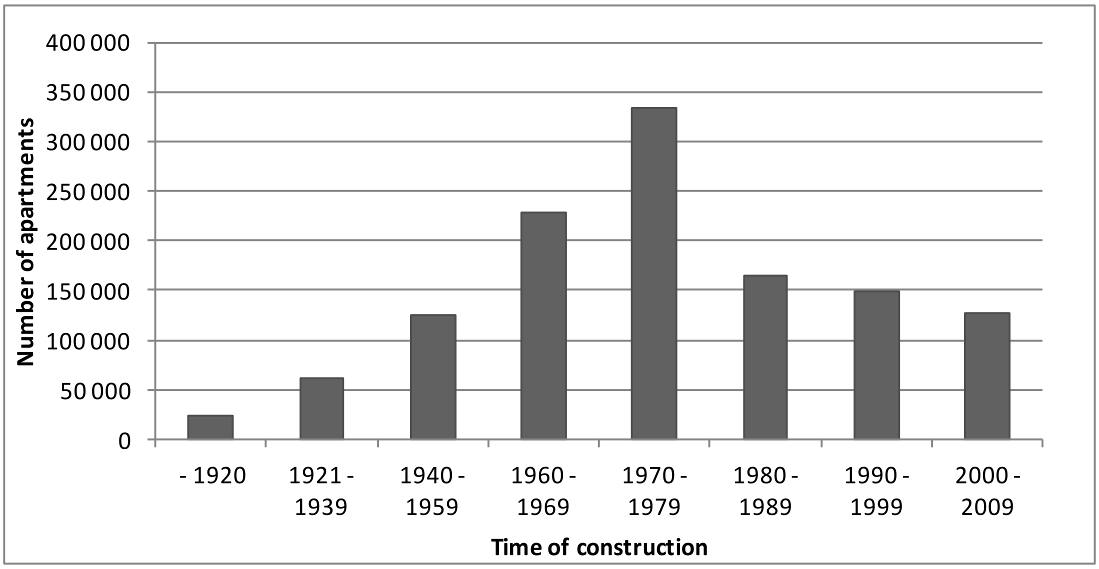


Fig. 4.4 Multi-storey residential building construction in Finland in different periods (Statistics Finland 2010).

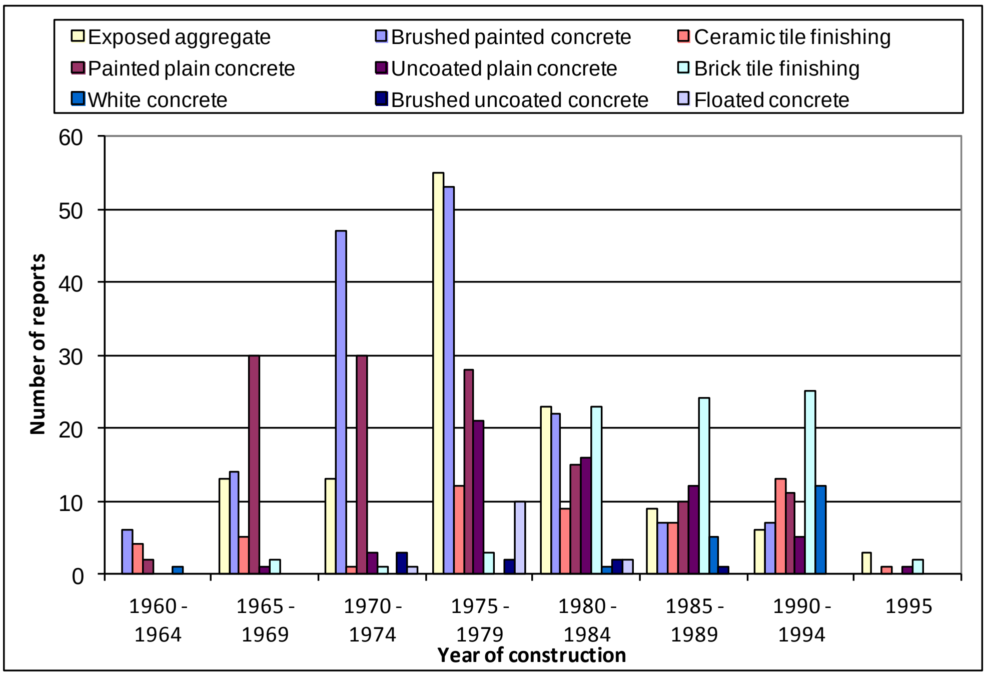


Fig. 4.5 Five-yearly distribution of the surface types of the buildings in the database.

The database does not contain equal amounts of all facade surface types from each year; clear differences do exist. The surface types that stand out in the database are brushed painted, exposed aggregate and brick panel-clad surfaces.

The facade surface type is provided for statistical purposes on a building project notice attached to a building permit application. Statistics compiled on a building permit basis do not, however, classify different concrete facade types accurately enough, which means that the actual surface type distribution of multi-storey buildings' concrete facade types cannot be derived directly from statistics.

The actual surface types of buildings completed in 1965-1994 and their shares in different periods were examined on the basis of a sample of 418 precast multi-storey residential buildings compiled from the suburbs of Helsinki, Turku, Tampere, Jyväskylä and Oulu (Köliö 2011). The facade types of the compiled database are well in line with the surface type survey conducted on said cities, see Fig. 4.6.

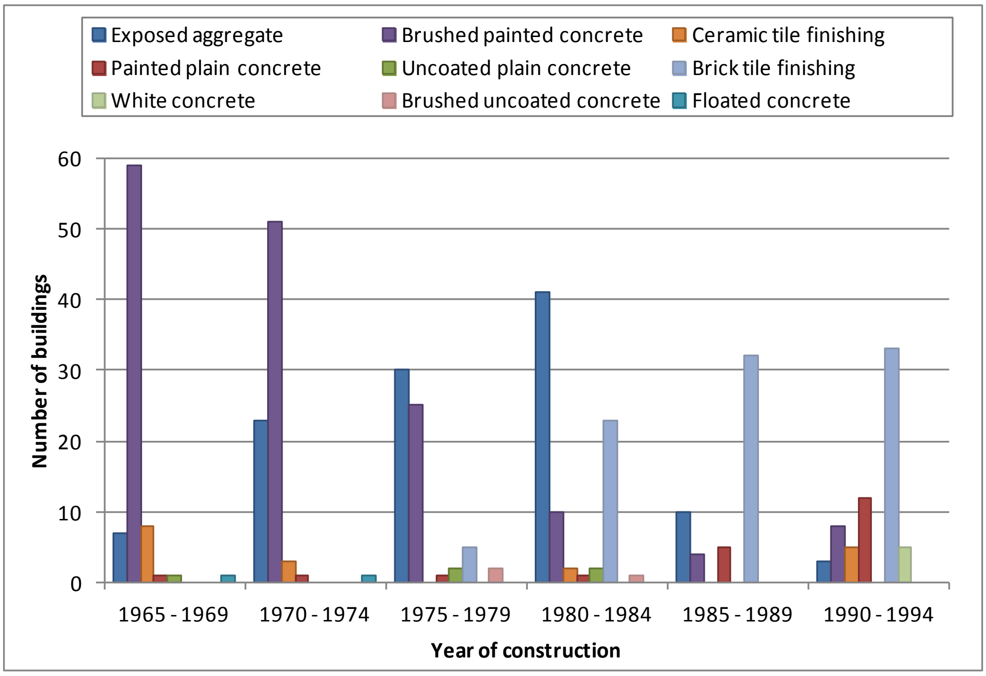


Fig. 4.6 *Facade surface type distribution of sample compiled from suburbs of Helsinki, Turku, Tampere, Jyväskylä and Oulu, n = 418 buildings (Köliö 2011).*

Based on the number of samples drilled from concrete facades, the database includes considerably fewer painted form-finish facades than the multi-storey residential stock in general. The most probable reason for that is the recommendation of the Condition investigation manual for concrete facade panels (2002) to take at least three parallel samples from each different panel type. That results in the drilling of relatively more samples from plinth panels than other facade panels in relation to the total facade area.

5 RESULTS AND DISCUSSION

The results are presented mainly in tables and graphs, separately for facades and balcony structures. Where no danger of confusion exists, or something applies equally to facades and balconies, said separation is not made.

5.1 Structural and material properties

This chapter looks at the durability properties of concrete facades and balconies on the basis of realised dimensions and properties of structures. The analysis is based on observations and examinations of concrete core samples taken from structures and field measurements.

5.1.1 Thickness of structure

Facades

The facades of precast multi-storey residential buildings consist of sandwich panels that are typically non-bearing panels with window openings along the sides and solid load-bearing panels at the ends. The thickness of the outer layer of the concrete sandwich panel has been measured as the length of the samples taken during condition investigation. Measured outer layer thicknesses have been compiled in Table 5.1 according to different surface types of facades.

The outer layer of a sandwich panel must be thick enough to ensure that a sufficient concrete cover can be placed on top of the centrally installed reinforcing mesh and edge bars, and that the bonding of trusses embedded in the inner surface of the outer layer is sufficient and reliable. Some of the panel’s lifting lugs are also installed in the outer layer and also require sufficient cover thickness and anchoring. The design thickness of outer layers has varied in different periods according to facade surface. Most typically it has been 60 mm, but in the early years of prefabricated construction even 40-50 mm (Pentti et al. 1998).

Table 5.1 Thickness of outer layer of precast sandwich panels based on length of samples of condition investigations.

	Bearing panel					Non-bearing panel				
	Length [mm]				No.	Length [mm]				No.
	Min.	Max.	Av.	Std. dev.		Min.	Max.	Av.	Std. dev.	
Exp. aggregate	26	100	62.2	11.1	552	37.5	95	63.2	9.7	339
exp. agg. layer	2	79	26.6	10.3	335	9	85	27.2	11.4	224
backing con. layer	0	75	35.1	13.1	335	0	71.5	36.2	14.2	223
Brushed painted	31	175	59.5	13.2	736	32	109	58.9	11.2	527
Clinker-clad	34	175	67.2	17.6	214	44	105	67.3	11.3	70
Painted form-finish	32	135	65.3	19.1	204	27	111	62.7	13.0	135
Unpainted form finish	44	153	78.0	21.2	155					
Brick panel-clad	44	126	82.3	11.6	350	33	107.5	78.6	13.5	131
Brushed unpainted	36	84	54.8	11.8	34	44	57	50.1	4.9	7
White concrete	46	118	69.0	14.9	48	59	89	69.5	8.9	31
white con. layer	11	55	24.7	11.2	20	11	62	33.1	16.2	15
backing con. layer	21	58	39.6	10.3	20	0	62	37.6	21.5	15
Floated unpainted	50.5	75	62.7	7.5	13		60			1

On average, facade panel outer layer thicknesses are quite close to each other ranging from 55-70 mm. Exceptions in the thicker direction are the unpainted form-finish panel, generally used in the plinth storey of a building, and the brick panel-clad panel, where the around 20-30 mm brick panel increases the total thickness of the outer layer. The thickness of the concrete section in the case of these brick panel-clad panels is nevertheless of the same magnitude as in facade panels in general. Thicknesses of the outer layer vary considerably in all surface types. No significant differences in the average thickness of the outer layer and variation of thickness have been observed between bearing and non-bearing panels with the same facade panel surface type.

The concrete of the outer layer of exposed aggregate and white concrete panels is generally a combination of two layers of different concrete types cast together. The thickness of the surface layer varies considerably from a few millimetres to the thickness of the entire outer layer. The average exposed aggregate and white concrete layers are 25-33 mm.

All facade panels are manufactured in horizontal forms. Brushed and floated panels are manufactured with the outer surface up, which allows finishing the surface of the green concrete. In the case of sandwich panels, thermal insulations are installed on the inner layer cast against the form, and the reinforced outer layer is cast last. In all other cases the outer layer of a sandwich panel is cast against the form, and the other layers are installed on top of the outer layer.

Although average outer layer thicknesses are close to design values, quite small outer layer thicknesses also occur with all facade surface types. Yet, just one sample has been reported to have broken (length 10 mm). A total of 43 (1.2%) under 40 mm outer layer thicknesses were discovered, mainly in bearing panels, whose facade surface was either exposed aggregate (11 samples) or brushed painted (14 samples; six of the samples were from non-bearing panels). In the case of other panel and facade surface types under 40 mm thicknesses occurred only in isolated cases. On the other hand, a total of 101 unusually thick, over 100 mm, outer layer thicknesses were measured. They were most common in form-finish (45) and brick panel-clad panels (26). The greatest individual outer layer thicknesses measured were 175 mm in the case of a brushed painted and a clinker-clad panel.

Most typically, all smallest and largest outer layer thicknesses occur locally and are the result of mistakes in panel manufacture since none of the target buildings had only remarkably small or large outer layer thicknesses. It is probable that the concrete may not have been spread evenly within the form in the case of the form-finish exposed aggregate outer layer. Or a worker may have left a boot print in the fresh concrete during installation of thermal insulations. It is also possible that in the case of the panel with a brushed painted surface cast with the outer layer up, extra strips of thermal insulation may have been left on top of the actual thermal insulation prior to casting the outer layer, which is responsible for thinner areas in the outer layer. Walking on the thermal insulations of panels cast with the outer layer up during reinforcement installation or pouring may also have pressed together thermal insulations, or the concrete mix may have been poured in the middle of the panel in one go and then spread out. Both factors can create remarkably thick areas in the outer layer.

Providing sufficient reinforcement cover depth for corrosion protection is impossible with outer layers less than 40 mm thick. By force of circumstances reinforcements remain either too close to the outer surface, whereby the protective concrete layer requirement is not met, or too close to thermal insulations, which may compromise bonding. In theory, it is possible to install the planned reinforcement with splices and lifting lugs only in a 65 mm thick outer layer, so as to meet the cover depth requirement of 20 mm. A cover depth of 25 mm requires an outer layer of 85 mm considering installation tolerances. Reinforcements of outer layers will not function optimally also in the case of excessively thick outer layers. When reinforcements lie too deep, they cannot prevent cracking due to the shrinking of concrete.

Balconies

The balconies of precast multi-storey residential buildings are stacked balconies made of side panels, slabs and parapet panels supported on their own foundations. The structural thicknesses of various panels have been measured as length of samples taken during condition investigation. Measured structural thicknesses are compiled in Table 5.2 by panel types.

Table 5.2 Structural thicknesses of balcony panels based on lengths of samples of condition investigations.

	Length [mm]			No.	Broken samples [mm]		No.
	Min.	Max.	Av.		Min.	Max.	
Side panel	120	205	159.0	700	43	118	48
Slab	120	329	156.6	786	36	119	89
Parapet	46	148	87.0	696	17	48	3

All balcony panels are bearing structural members dimensioned for bending, deflection or compression as necessary. The Finnish Concrete Code (2004) sets a minimum thickness of 120 mm for balcony side panels. No minimum thickness is set for slabs and parapets. The design thickness of a balcony parapet has generally been 70 mm. Slab thickness is determined primarily on the basis of its length and loading. In practical design the average strength of a slab is calculated tentatively by the following formula:

$h_l = 0.8 \times L/20$ where (5.1)

h_l is thickness of the slab [mm]
L is span of the slab [mm].

This way, the average minimum thickness of a balcony slab 4 000 mm long would be 160 mm. However, the upper surface of the slab incorporates slopes for draining off rainwater, which means that the slab is not of uniform thickness everywhere, but typically grows thinner toward one edge. Thus, it may be only about 120 mm thick at places, which was selected as the minimum slab thickness of Table 5.2.

No samples from balcony slabs have been reported to have been accidentally or intentionally broken. However, all slabs less than 120 mm thick have been considered to be broken. In the case of slabs the sloping of the upper surface for draining off rainwater causes natural variation in slab thickness between slabs of the same building, which makes it difficult to compare the samples. A total of 74 balconies had a slab over 200 mm thick, which were divided quite evenly over various decades. Two slabs over 300 mm thick dated back to 1972. The thickness of a structure did not prevent sufficient reinforcement cover depth. Modes of production may have varied by plants meaning that different methods have been used to support reinforcements.

Condition investigation reports mentioned no samples drilled from balcony side panels having been broken accidentally or intentionally. Comparison of remarkably short samples from a building to other samples from it showed that all samples less than 140 mm long had been broken accidentally or intentionally in connection with core drilling. The total number of broken samples was thus 70. Typical balcony side panel thicknesses are 150-161 mm, the typical design thickness being 150 mm. Only six side panels were over 200 mm thick. Thicker (≥ 180 mm) balcony side panels existed only in buildings erected in the 1990's. Balcony side panels are generally unreinforced and equipped with edge bars and lifting lugs to distribute shrinkage cracking. The cover depths of reinforcements could very well have been realised with typical side panel thicknesses.

Neither was any of the samples drilled from balcony parapets in condition investigations reported to have been accidentally or intentionally broken. When comparing remarkably short samples to others from the same building, it appears that generally samples less than 50 mm long were accidentally or intentionally broken during coring. However, there were only a total of three broken samples. The thinnest balcony parapets were found in a building erected in 1986, where parapet thicknesses were in the 46-52 mm range. Buildings with parapet thicknesses in excess of 110 mm were for the most part completed at the end of the 1980's or later.

The average parapet thickness is quite sufficient for embedding moment reinforcement in both surfaces of a structure while meeting cover depth requirements. On the other hand, adequate reinforcement cover depth cannot be provided in the case of parapets less than 80 mm thick. A total of 144 (21%) such parapets were reported.

5.1.2 Properties of concrete of samples

The quality of the concrete used for facades and balconies could be assessed based on the degree of compaction, maximum size of aggregate, and capillarity of concrete revealed by condition investigation reports. Thin-section analyses have often been used to evaluate the soundness of aggregate-cement contacts. Generally, the bonding of cement to aggregate has been good and the contact surfaces have been sound if cracking of concrete indicating frost damage has not been observed. The water-cement ratio of concrete can be evaluated with some accuracy by thin-section analysis. Such evaluations have not, however, been done of the material compiled for this research. A few isolated comments on normal, higher than normal, or lower than normal water-cement ratios have been made in thin-section reports, but they are too few for statistical analysis.

The compressive strength of the concrete used for facades and balconies has not been tested generally. When compressive strength is mentioned in condition investigation reports, it is stated that the design strength of concrete was C20/25 for facades and balcony side panels and C25/30 for balcony slabs. Design strength of balcony parapets is not mentioned.

Compaction and maximum aggregate size of concrete

Facades. The compaction of cast concrete has been graded on a scale of 1-5 (in whole numbers). The average compaction of different facade types has varied from 2.8-3.9, see Table 5.3. On average, compaction has been poorest with brushed painted/unpainted concrete and brick panel-clad facades. The poorer than average compaction of the brick panel-clad panel is surprising since the significant capillary suction of the brick panel makes the concrete immediately underneath it more compact than usually. The suction of the brick panel lowers the water-cement ratio of the concrete in the bonding area and its immediate surroundings. This has been found to decelerate considerably concrete carbonation under the brick panel (Sulankivi 1993, Pentti and Mattila 1996). The water absorption capacity of brick panel is limited, which is why its concrete compacting influence clearly does not extend more than a few millimetres into the concrete.

The compaction of the concrete of painted/unpainted form-face panels has also been slightly more successful than on average. In the case of layered facade types, exposed aggregate and white concrete, the compaction of the facade surface layer has been more successful than that of backing concrete.

Table 5.3 Average degree of compaction of concrete samples and share of failed compaction.

	Average compaction [1-5]	No.	Poor compaction [1]	
			No.	%
Exposed aggregate				
exp. agg. layer	3.59	467	4	0.86
backing con. layer	3.12	942	15	2.02
Brushed painted	2.84	976	16	1.64
Clinker-clad	3.25	228	2	0.88
Painted form-finish	3.02	265	6	2.24
Unpainted form-finish	3.39	142	0	0
Brick panel-clad	2.89	319	10	3.13
Brushed unpainted	2.94	35	0	0
White concrete				
white conc. layer	3.92	25	0	0
backing con. layer	3.17	57	0	0
Floated unpainted	3.00	14	0	0
Side panel	3.24	789	12	1.52
Slab	3.24	756	5	0.66
Parapet	3.16	568	2	0.35

Wide variation in the compaction of the concrete of all facade types was found between samples from the same target. In general, deficient compaction occurs only in isolated cases, and it is not concentrated by area or surface type.

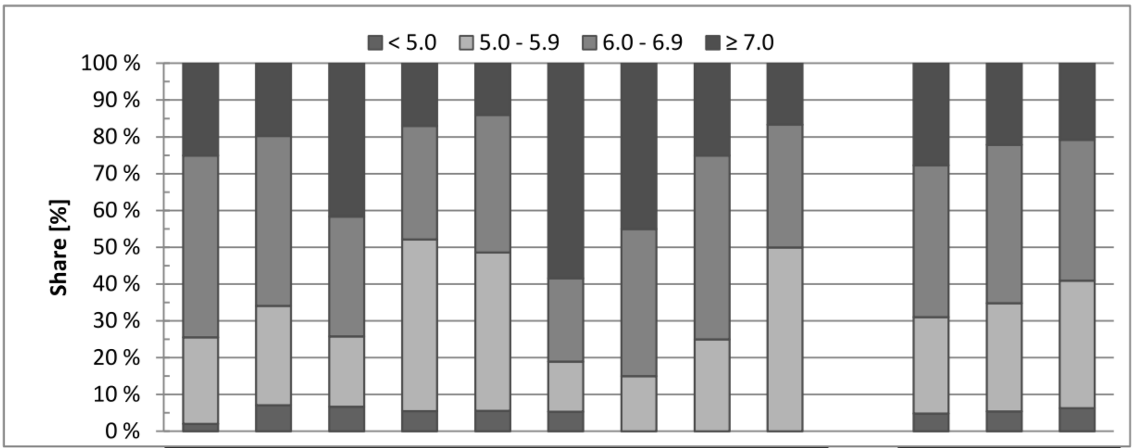
The maximum grain size of the aggregate of facade concrete measured from the surface of drilled core cylinders varies from 2-50 mm (n = 3 847). The mean maximum grain size of different facade types is 7.7-16 mm. Only a few isolated cases of over 40 mm and under 4 mm maximum grain sizes were observed. No correlation between maximum grain size of aggregate and concrete compaction was detected.

Balconies. The compaction of side panels and slabs was on average more successful than that of parapets, see Table 5.3. Wide deviation in compaction of all balcony panels was noticed between samples from the same building. Deficient compaction occurred only in a few isolated instances.

The average maximum grain size of the aggregate of side panels was 19 mm (n = 912), of slabs 18.1 (n = 901), and of parapets 15 (n = 675). The maximum grain size of side panels and slabs varies from 2-59 mm and that of parapets from 4-46 mm. Under 6 mm maximum grain sizes were detected only in a few isolated cases, and a total of 36 cases of sizes over 40 mm.

Capillary porosity of concrete

The capillary porosity of concrete has a major effect on the wetting of concrete in outdoor conditions. The larger the share of concrete pores in the capillary zone, the more thoroughly and quickly the concrete structure gets wet during rain and sleet. Carbonation also advances more rapidly in porous than dense concrete. The shares of concrete samples by facade and panel types at various degrees of capillary saturation p_w [w%] are presented in Figure 5.1. The degree of capillary saturation of concrete is defined as part of the protective pore test according to standard SFS 4475.



	Expo-sed aggre-gate	Brus-hed pain-ted	Clin-cer-clad finis-hing	Pain-ted plain con-crete	Un-coa-ted plain con-crete	Brick panel-clad	Whi-te con-crete	Brus-hed uncoa ted con-crete	Floa-ted con-crete	Balco ny frame panel	Balco ny slab	Balco ny para-pet panel
Average	6,5	6,3	6,7	6,1	6,1	7,2	6,9	6,4	6,0	6,5	6,4	6,3
Min	4,4	1,6	4,4	4,1	4,1	4,2	5,0	5,2	5,1	3,2	0,0	0,2
Max	10,8	9,2	9,6	9,3	8,2	14,3	9,3	7,7	7,6	10,5	14,7	10,5
Total N	536	904	209	236	107	243	40	24	12	719	718	491

Fig. 5.1 Degree of capillary saturation of concrete by classes.

No correlation was found between the capillarity and compactness of concrete. Capillarity of concrete cannot be detected visually since the capillary pores are 4 nm-7 µm in diameter (Pigeon and Pleau 1995).

Facades. The degrees of capillary saturation of facades generally vary between 4.1-11 w%, and those of brick panel-clad facades between 4.2-14 w%. Variation is also wide between samples from different panels of the same building. The capillarity of form-finish and floated panels is lower than the others' since their degree of capillary saturation is less than 5.9 w% in half of the cases. The degree of capillary saturation of 66-74% of other facade types is over 6.0 w%. Extremely low degrees of under 5.0 w% occur generally in a quite small share of all facade types.

A remarkably large portion of brick panel-clad facades has a high degree of capillary saturation: 58% of the samples revealed a degree of at least 7.0 w%. The degree of saturation of these samples is not merely an indication of the capillarity of concrete, but most of the samples have probably incorporated a brick panel. The porosity of burnt brick is for the most part in the capillary zone (Pentti 1988); thus the share of the brick distorts the result to that extent. On the other hand, a capillary brick panel transfers moisture all the way to the backing concrete quite quickly during rain and sleet.

A considerable share of clinker-clad facades also has a high degree of capillary saturation. In the case of clinker-clad panels, carbonation and wetting of concrete can occur only through the joints between clinker tiles since most of the facade is typically covered by dense clinker tiles. Penetration of carbon dioxide and rainwater may occur faster than generally at the joints due to high capillarity.

Balconies. The concrete of all balcony panel types is quite similar as to capillary porosity. The degree of capillary saturation varies from 2.3-14 w%. Variation is also wide between samples from different panels of the same building. The degree of capillary saturation of balcony panels is remarkably high as all samples showed a capillary saturation degree ≥ 6.0 w%.

The high capillary porosity of the concrete used for facades and balconies indicates a quite high water-cement ratio since the share of capillary pores is the greater, the higher the water-cement ratio of concrete. When the ratio of concrete made of Ordinary Portland Cement exceeds 0.7, the capillary pores always form an integrated network independent of the hydration rate of cement (Neville 1995). In panel manufacture the workability of concrete has probably been adjusted by the amount of water used instead of various chemicals such as plasticisers or air-entraining agents.

5.1.3 Steel bars in samples and cover depths of reinforcement

The size and type of reinforcement used in facades and balconies has been assessed on the basis of the steels embedded in samples. Stainless steel grades have not been used for reinforcements of facades and balconies or lifting lugs in any building. Realised reinforcement cover depths have been assessed both by measuring the cover depths of examined concrete samples and by large-scale cover depth measurements during field investigations.

Steels in samples

Facades. The diameter of the reinforcement mesh wire used in all facades has been 3 or 4 mm. Typical edge bar diameters have been 6 or 8 mm, sometimes even 10 mm. Mesh reinforcements have typically been welded meshes while other reinforcements have been ribbed bars. Lifting lugs have normally been of round bar 15-20 mm in diameter.

Besides these reinforcements necessitated by the performance of a structure or its installation, the samples also contain different so-called erection bars used to facilitate installation of the actual reinforcement. These are typically 6 or 8 mm ribbed steels.

Samples also contain steel fasteners of the trusses used to attach the outer layer of facade panels. Typically the bar of a truss is 6 mm ribbed steel and the diagonal 5 mm stainless round bar.

Balconies. The steels of balcony parapets are typically ribbed and 6 or 8 mm in diameter. The steels of the soffits of balcony slabs are generally 8, 10 or 12 mm ribbed steels. The steels of the tops of slabs are usually 6 or 8 mm ribbed steels originating from the reinforcements used to join the balcony parapet and the slab. Both parapets and slabs have individual steels in the form of lifting lugs or those used to fasten handrails. The typical diameter of such, generally round, steels is 15-20 mm.

Balcony side panels are usually unreinforced wall-like panels generally containing only so-called edge bars of 6, 8 or 12 mm ribbed steel. Moreover, the samples contain some 15-20 mm round steels thought to be lifting lugs. Most side panel samples contain no steels.

Samples drilled from balcony panels also contain some steels 3, 4 and 6 mm in diameter whose purpose is not made clear by condition investigation reports.

Reinforcement cover depths

Facades. The concrete cover depth distributions of steels in concrete samples are shown in Table 5.4. Only a small share of under 5 mm cover depths measured from the outer surface of samples were found in painted form-finish and brushed painted panels. In clinker-clad panel samples the share of under 10 mm cover depths was already significant at 6.2%. With other facade surface types, the share of under 10 mm cover depths is at the most 3.6%. Only isolated samples were taken from floated unpainted facades, which makes them incomparable.

Table 5.4 Shares of reinforcement cover depths by classes in concrete samples drilled from facade panels.

		Share of cover depth classes [%]				No.
		0-4 mm	**5-9 mm**	**10-14 mm**	**15-19 mm**	
Exposed aggregate	outer surf.	0.00	0.64	3.81	13.60	472
	inner surf.	2.11	4.68	12.41	13.30	427
Brushed painted	outer surf.	0.47	1.24	4.81	11.32	645
	inner surf.	9.69	13.44	19.73	16.67	588
Clinker-clad	outer surf.	0.00	6.21	14.29	19.25	161
	inner surf.	2.29	3.05	6.10	15.30	131
Painted form-finish	outer surf.	1.06	1.60	13.30	13.30	188
	inner surf.	5.52	9.20	9.20	17.18	163
Unpainted form-finish	outer surf.	0.00	3.61	4.82	16.87	83
	inner surf.	1.52	7.58	9.09	10.61	66
Brick-panel clad	outer surf.	8.86	6.27	9.23	4.42	271
	inner surf.	4.17	3.13	7.29	9.90	192
Brushed unpainted	outer surf.	0.00	0.00	0.00	18.18	22
	inner surf.	6.67	13.33	20.00	13.33	15
White concrete	outer surf.	0.00	0.00	5.36	12.50	56
	inner surf.	2.33	2.33	2.33	4.65	43
Floated unpainted	outer surf.	0.00	16.67	0.00	0.00	6
	inner surf.	11.11	33.33	11.11	11.11	9

The share of small reinforcement cover depths deviates totally from other facade surface types in the case of brick panel-clad facades. The share of under 10 mm cover depths is 15%. However, it should be noted that cover depths have not been measured from the outer surface of samples, but from the interface between concrete and brick panel, thereby ignoring the about 20 mm contribution of the brick panel. On the other hand, the carbon dioxide diffusion resistance of brick panel is so low that it does not retard the migration of carbon dioxide into the backing concrete (Sulankivi 1993), which means that the share of the brick panel should actually be ignored in cover depth measurement.

The cover depths measured from samples taken from the inner surface of the outer layer are considerably smaller than those on the outer surface. Small cover depths occur in especially large numbers in facades cast with the facade surface up. It means that the support of reinforcement for concrete placement has been quite inadequate allowing the reinforcement to sink close to the thermal insulation.

In field investigations the cover thicknesses of facade panels have been measured widely from the outer surface of facades using a non-destructive cover depth meter, which provides a considerably larger sample than merely measuring cover depths from samples. The cover depth distributions of the reinforcements of different facade types are shown in Table 5.5 separately for reinforcement mesh and edge bars.

Table 5.5 Shares of cover depths of meshes and edge bars by cover depth categories measured from the outer surface of facade panels.

		Share by cover depth categories [%]				**No.**
		0-4 mm	**5-9 mm**	**10-14 mm**	**15-19 mm**	
Exposed aggregate	mesh	0.00	0.47	2.60	10.70	39 473
	edge bars	0.00	0.34	1.14	5.62	21 167
Brushed painted	mesh	0.01	0.57	1.90	8.62	68 804
	edge bars	0.01	1.10	2.55	8.17	36 154
Clinker-clad	mesh	0.00	1.97	6.65	22.38	8 582
	edge bars	0.00	1.10	6.16	18.49	4 690
Painted form-finish	mesh	0.13	3.22	4.11	13.09	18 183
	edge bars	0.26	2.44	3.83	11.51	7 991
Unpainted form-finish	mesh	0.01	2.07	4.70	14.72	7 056
	edge bars	0.31	0.89	3.01	11.69	2 265
Brick panel-clad	mesh	1.41	7.42	5.16	2.97	14 988
	edge bars	0.02	3.78	4.32	3.31	9 759
Brushed unpainted	mesh	0.00	1.38	5.01	11.97	1 653
	edge bars	0.00	6.38	10.79	15.09	1 332
White concrete	mesh	0.00	0.10	1.80	11.64	3 869
	edge bars	0.00	0.00	1.61	15.28	2 193
Floated unpainted	mesh	0.00	0.57	1.78	6.75	1 578
	edge bars	0.00	2.11	1.74	4.78	520

The shares of small, under 5 and under 10 mm cover depths, are generally smaller than the corresponding outer surface cover depths measured directly from samples, see Table 5.4. An exception are brick panel- and clinker-clad facades. In the case of clinker-clad facades, at least in some measurements, the reading of the cover depth meter has clearly been reduced – typically by 20 mm – which is the only possible explanation for the under 20 mm cover depths. The share of under 10 mm cover depths in clinker-clad facades is clearly larger based on sample measurements than field measurements. If the typically about 5 mm (often also 8 mm and 12 mm) reduction due to the thickness of the clinker tile is taken into account in field measurements, the cover depth distributions based on sample and field measurements are quite close.

The surface of most facade panels, such as exposed aggregate and brushed panels, is rough, which means that cover depth meters may indicate slightly exaggerated cover depths. Surface roughness is 3-5 mm depending on facade surface type and roughness, which has not generally been considered when measuring and reporting cover depths. Cover depths of areas suffering from already visual damage are not measured in field investigations either. This is another reason why the shares of the smallest cover depths indicated by field measurements are smaller than in reality.

The cover depth of reinforcements can be measured considerably more accurately directly from concrete samples than by using non-destructive methods. Thus, a cover depth distribution measured directly from samples may be considered more reliable although the share of visible corrosion damage is ignored also in that case. On the other hand, the sampling required is much smaller than when using a cover depth meter during field investigations.

Although the cover depths of reinforcement to a quite large extent do not meet the minimum requirements set in codes and durability guidelines, it can be stated that the shares of small, under 5 mm and under 10 mm cover depths, are on the whole quite small in the case of facade panels.

Balconies. The cover depth distributions of steels embedded in concrete samples drilled from balcony panels are presented in Table 5.6. Only quite small shares of under 5 mm cover depths were found in all balcony panels measured from the outer or top surface. Bar spacing in balcony parapets is generally 100-150 mm meaning that a 3.5% share of under 10 mm cover depths is a lot. On the other hand, side panels primarily have steels at the edges, and slabs at the top of the front edge consisting mainly of starter bar ends for fastening the parapet panel.

Table 5.6 Shares of reinforcement cover depths of concrete samples drilled from balcony panels by cover depth categories.

		Share by cover depth category [%]				**No.**
		0-4 mm	**5-9 mm**	**10-14 mm**	**15-19 mm**	
Side panel	outer surf.	0.53	2.63	5.79	10.00	190
	inner surf.	2.16	0.72	4.32	11.51	139
Slab	top.	0.92	3.46	6.91	6.91	434
	soffit	1.75	2.34	7.60	10.72	513
Parapet	outer surf.	1.45	2.31	4.34	8.38	346
	inner surf.	2.63	4.61	13.82	16.12	304

The reinforcement cover depths measured from inner surfaces of samples from parapets were considerably smaller than those measured from outer surfaces. As much as 7.2% of the cover depths were under 10 mm. The share of small cover depths was also significant in the soffits on slabs since the slab is always a bearing structure, where reinforcement plays a crucial role. The parapet is not usually a bearing structure, but with reinforcements on both surfaces, it functions as a cantilever, and the reinforcements play a role in the static functioning of the structure. On the other hand, the reinforcement of side panels is rarely statically functioning and therefore usually causes aesthetic damage.

Reinforcement cover depths of balcony panels have been widely measured from the surfaces of balcony panels using a non-destructive cover depth meter, which allows a considerably larger sample of measurements than when measuring only from samples. The measured reinforcement cover depth distributions of different panel types are shown in Table 5.7. In field measurements the cover depths of balcony side panels have been measured at edge bars and have not been categorised by outer and inner surfaces.

Table 5.7 Shares of reinforcement cover depths by cover depth categories based on field measurements of balcony panels.

		Share by cover depth category [%]				n
		0-4 mm	5-9 mm	10-14 mm	15-19 mm	[kpl]
Side panel	outer surf.	0.06	3.84	5.47	15.10	32 540
Slab	top	0.01	3.64	5.45	13.60	12 957
	soffit	0.04	3.69	5.65	14.41	42 628
Parapet	outer surf.	0.01	1.90	4.12	11.49	26 636
	inner surf.	0.19	7.53	10.01	20.59	18 665

The shares of small, under 5 mm, cover depths are generally smaller than corresponding reinforcement cover depths measured directly from samples. Correspondingly, the shares of 5-9 mm cover depths are generally slightly higher in field measurements than in measurements based on samples, see Table 5.6.

Balcony panels are typically form-finished on at least three sides, which means that the surfaces are quite smooth. Therefore, the cover depth meter readings should correspond to the distribution of cover depths measured directly from samples. However, in field measurements cover depths are not generally measured at places where visible damage already exists. Thus, the shares of the smallest cover depths are smaller than in reality in field measurements. The reinforcement cover depth distribution measured directly from samples can be considered more reliable than field measurements also in the case of balcony structures although the share of visible corrosion damage is ignored also there. The sampling required is in any case smaller.

The reinforcement cover depths of balcony panels do not in large part meet the minimum requirements set for them in codes and durability guidelines. The situation with respect to structural damage is worse with balconies than facades since the steels used in balcony panels are of larger diameter than facade steels, and consequently more easily cause visible corrosion damage as they rust that extends deeper into the structure than in the case of smaller diameter steels. Balconies are also bearing structures, which means that the steels of the slab soffit and parapet are needed to transfer forces due to loading.

The share of small, under 10 mm, cover depths is often significant in the selection of the balcony repair method. In side panels their share is less than 4 per cent, which corresponds to an about 700 mm long section of the edge bars of a single side panel. Cementitious patch repair of that can still be considered economical.

The share of under 10 mm cover depths of the functioning steels of balcony slab soffits is about 4 per cent, that of parapet outer surfaces just under 4 per cent, and that of parapet inner surfaces over 7.5 per cent. The amount of reinforcement in these panels depends on the dimensions and loading of the structures, which means that a similar assessment of the length in metres of reinforcement as in the case of the less than 10 mm reinforcement cover depth of side panels cannot be made.

5.1.4 Type and material of fastenings

Facades. One of the key factors to be determined by a condition investigation is the safety of structures and security of fastenings. The fastening method of facade panels and used materials as well as the condition of fasteners has been examined generally in connection with core drilling through drill holes. The fastening method and material of the outer layer of facade panels has been determined in the case of a total of 414 buildings.

The most typical outer layer fastening method with sandwich panels is the steel truss with stainless steel diagonals 5 mm in diameter and stringers of ordinary ribbed bar. In a total of eight buildings the diagonals that penetrate the insulation layer are mild steel, which may rust unprotected. Five of the buildings were completed in 1975, the others in 1967 and 1978.

In five buildings the outer layer was fastened with stainless structural steel. Three of them were completed in 1965 and the rest in 1963 and 1978. In a total of seven buildings the outer layer was been fastened by structural steel components protected against corrosion by various means. Four of the buildings were completed in the 1960’s and three in the 1970’s.

By way of generalisation, it may be said that there has been more variation in the fastening methods of the outer layer of buildings completed in the 1960’s, and the fasteners have often been of regular non-alloy steel, whose corrosion protection has consisted of dipping it in grout, bituminisation, or they have been left unprotected. These fasterners have often been covered by a thick layer of rust, but their diameter has been reduced by a maximum of 1 mm. Since the 1970’s fasteners that penetrate thermal insulation have been almost without exception of stainless steel.

All in all, the fastening reliability of outer layers has generally been good. According to the database, it has been inadequate only in isolated cases, and even then mainly due to frost damage of concrete. The strong curvature of thin-shell panels is not necessarily a sign of extensive and far advanced frost damage, especially in clinker- and brick panel-clad buildings, where curvature is typically caused by shrinkage differences.

Balconies. The targets of condition investigations had a total of 364 balconies. The balconies of precast members are in most instances supported on bearing side panels, columns or via the bearing outer layer of the external wall on their foundations. Such stacked balconies may protrude from the building frame or be recessed. All or part of the balconies of twelve buildings are hung or cantilever balconies – generally the so-called airing balconies are cantilever balconies while the dwelling-specific balconies are stacked.

Stacked balconies are tied laterally by storeys to prevent tipping over: from side panels to transverse partitions or from balcony slabs to intermediate floor slabs. Tying to transverse partitions has been effected by welded joints using e.g. flat steel components or ribbed bars. The portion of non-alloy fasteners in the insulation cavity may be e.g. concreted. The balcony slab may be tied to an intermediate floor slab by flat or round bar fasteners (e.g. of stainless steel) of by balcony hinges (of stainless or hot-dip galvanized steel). In building completed in the 1960’s, lateral tying is generally by regular steels. Since the 1970’s mainly stainless steels have been used for that purpose.

Fastening reliability of structures has been inadequate only in a few isolated instances in the case of balconies supported on their own foundations. There, far advanced extensive frost damage of concrete has occurred.

In the case of the few cantilever balconies included in the research material, the support structures had suffered damage more often. It had been for the most part corrosion of unprotected tensile reinforcements at the slit insulations of a wall. With balconies supported on railway rails or other structural steels, the capacity of steel components has generally sufficed also in the case of further advanced corrosion.

5.1.5 Thermal insulation

The type, condition and thickness of facade panel thermal insulation was determined from a total of 2 161 drilled holes. The prevalent thermal insulation of facade panels has been mineral wool; others like EPS and wood-wool slab have been used in isolated cases. Thermal insulations were normally dry at the time of investigation.

The nominal thickness of thermal insulations was typically selected to meet existing regulations. Insulation thicknesses vary considerably up and down from nominal thickness as Figure 5.2 shows.

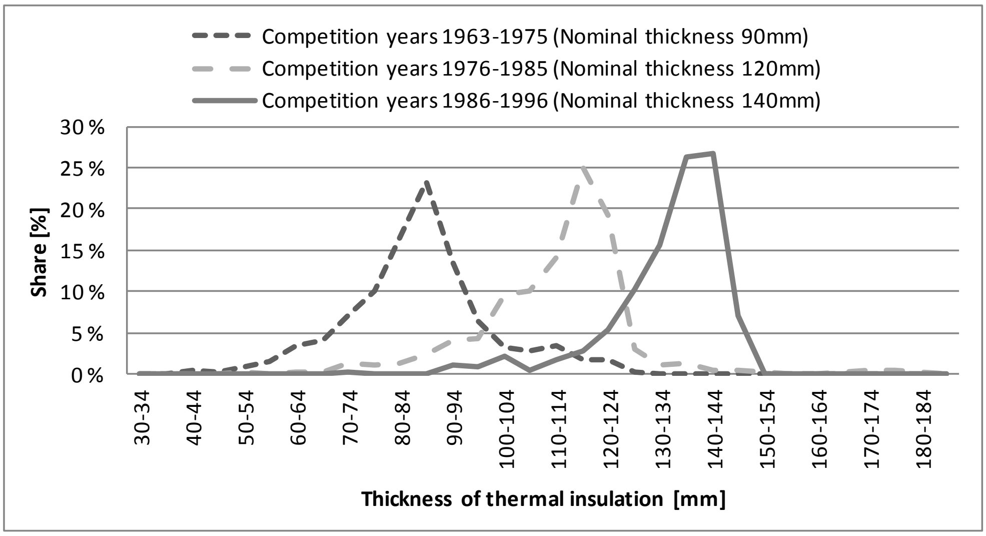


Fig. 5.2 Measured thickness of thermal insulation in sandwich panels according to year-built.

Thermal insulations have typically compressed about 10 mm during panel fabrication. Thicknesses may vary by more than 20 mm between different panels of the same building. Under 50 mm thermal insulation thicknesses were measured in isolated cases. The under 60 mm values were so few in number that they can be ignored. Thermal transmittances of sandwich panels calculated on the basis of actual thermal insulation thicknesses according to standards SFS-EN 6946 and SFS-EN 10456 are presented in Table 5.8.

Table 5.8 Thermal transmittance of concrete sandwich panel (U-value) calculated on the basis of average and 97% fractile insulation thicknesses. Inner layer thickness is assumed to be 80 mm and outer layer thickness 60 mm. Thermal conductivity of 0.044 W/m^{2}K of insulation was used.

Period	Measured average thickness of thermal insulation [mm]	Calculated U-value [W/m^{2}K]	Required U-value [W/m^{2}K]
	97% is more than 60	0.63	0.70 (North Finland), 0.81 (South Finland)
1963-1975	83	0.47	0.70 (North Finland), 0.81 (South Finland)
1976-1985	109	0.37	0.40 (1976) and 0.29 (1978)
1986-1996	131	0.31	0.28 (1985-2003)

The share of trusses is below 3%, which means that their influence need not be taken into account according to standard SFS-EN 6946.

Compared to the presently valid thermal insulation regulations (U ≤ 0.17 W/m^{2}K), the realised U-values are quite modest, but in the case of precast multi-storey residential buildings completed before 1976, the thermal insulation of facades has been clearly better than required at the time, also where the thermal insulation has undergone significant compression.

Since 1976 thermal insulation regulations have been tightened considerably, and the realised average U-values of sandwich panels are slightly below the minimum level set in the Finnish Building Code due to compression. Yet, the thermal insulation capacity of these buildings has improved remarkably compared to earlier erected buildings.

The thermal transmittance of facades is remarkably low compared to windows in all studied buildings. In 1976-2003 the minimum thermal transmittance required of windows was U ≤ 2.10 W/m^{2}K. No requirements on thermal insulation capacity were set for windows of structures built before that.

5.2 Corrosion of reinforcement

This chapter examines the durability properties of concrete facades and balconies with respect to reinforcement corrosion, detected corrosion damage, and factors affecting the initiation and propagation of corrosion. Investigated issues include carbonation of concrete, the chlorides contained by concrete, reinforcement cover depths and visible damage as well as the rain and sleet stress received by facades and balconies.

5.2.1 Carbonation of concrete

As noted in Chapter 2, carbonation of concrete proceeds at a decelerating rate as a function of time. Carbonation of concrete is studied using the carbonation coefficient, k [mm/a$^{0.5}$], instead of measured carbonation depths in order to make condition investigations conducted at different times comparable. Condition investigation reports give the average carbonation depth of concrete samples drilled from facades and balconies, which is used to calculate the average carbonation coefficient, k, of a sample.

Facades

The carbonation coefficient values for the outer surface of a structure and the outer layer of a sandwich element and inner surface of a thin-shell panel are compiled in Table 5.9, and the shares of the outer surface carbonation coefficients of various facade types by categories are shown in Figure 5.3. The outer surface carbonation coefficient has been calculated based on all facade surface type values. Thin-shell panels are not identified separately in the material meaning that structure type is determined on the basis of the carbonation depth of the inner surface. Whenever the carbonation depth of the inner surface has clearly exceeded that of the outer surface, a structure has been categorised as a thin-shell panel. The remainder are thus sandwich panels. Categorisation on this basis involves the risk that sandwich panels that include thermal insulation with ventilation grooves are placed in the same category as thin-shell panels. The risk of confusion is biggest with clinker-clad panels and panels manufactured at the end of the 1980’s. On the other hand, part of the thin-shell panels may also have remained among sandwich panels if ventilation of the inner surface has been limited leading to its slow carbonation.

Table 5.9 Average carbonation coefficients, k, by facade surface types calculated on the basis of average carbonation depths of concrete measured from samples.

		Carbonation coefficient, k [mm/a$^{0.5}$]				No.
		Av.	**Std. dev.**	**Min.**	**Max.**	
Exposed aggregate	outer surf.	1.96	1.28	0.00	11.40	849
	sandwich inner surf.	0.56	0.57	0.00	2.89	686
	thin-shell, inner surf.	2.83	1.48	0.76	11.40	162
Brushed painted	outer surf.	2.71	1.23	0.24	9.83	1285
	sandwich, inner surf.	0.37	0.49	0.00	2.94	1148
	thin-shell, inner surf.	3.44	1.91	1.57	9.83	53
Clinker-clad	outer surf.	0.57	0.78	0.00	4.23	208
	sandwich, inner surf.	0.53	0.61	0.00	3.62	168
	thin-shell, inner surf.	2.69	1.40	0.69	8.54	98
Painted form-finish	outer surf.	2.03	1.49	0.00	7.60	365
	sandwich, inner surf.	0.49	0.61	0.00	3.80	276
	thin-shell, inner surf.	3.67	1.60	1.63	7.75	33
Unpainted form-finish	outer surf.	2.16	1.74	0.00	14.38	160
	sandwich, inner surf.	0.51	0.61	0.00	2.00	115
	thin-shell, inner surf.	3.52	2.41	1.61	14.38	28
Brick panel-clad	outer surf.	1.47	2.20	0.00	12.25	427
	sandwich, inner surf.	0.62	0.70	0.00	4.11	340
	thin-shell, inner surf.	3.42	1.89	0.87	10.51	89
Brushed unpainted	outer surf.	1.91	0.79	0.58	3.47	41
	sandwich, inner surf.	0.24	0.36	0.00	1.39	41
White concrete	outer surf.	0.61	0.92	0.00	4.54	58
	sandwich, inner surf.	0.41	0.47	0.00	1.73	48
	thin-shell, inner surf.	2.45	1.21	0.30	4.62	13
Floated unpainted	outer surf.	2.67	0.62	1.92	3.98	13
	sandwich, inner surf.	0.55	0.65	0.00	2.04	13

Average carbonation of the outer surface is clearly slowest with clinker- and brick panel-clad and white concrete surfaced panels. Clinker tiles prevent effectively the diffusion of carbon dioxide into concrete meaning that carbonation may propagate only through the joints between tiles or in areas where clinker tiles have not completely bonded to the substrate. Brick panels improve the water-cement ratio of concrete and thus the density of concrete at the panel/concrete interface, which has been found to clearly slow down the carbonation of concrete. The slow carbonation of white concrete is most likely due to the higher grade of concrete containing more cement than the concrete used in so-called more conventional facade types.

Carbonation is clearly fastest in brushed and floated panels. Their facade surfaces have been treated while fresh. The carbonation rates of other facade surface types fall fairly evenly between the rate of the above-mentioned facade surface types.

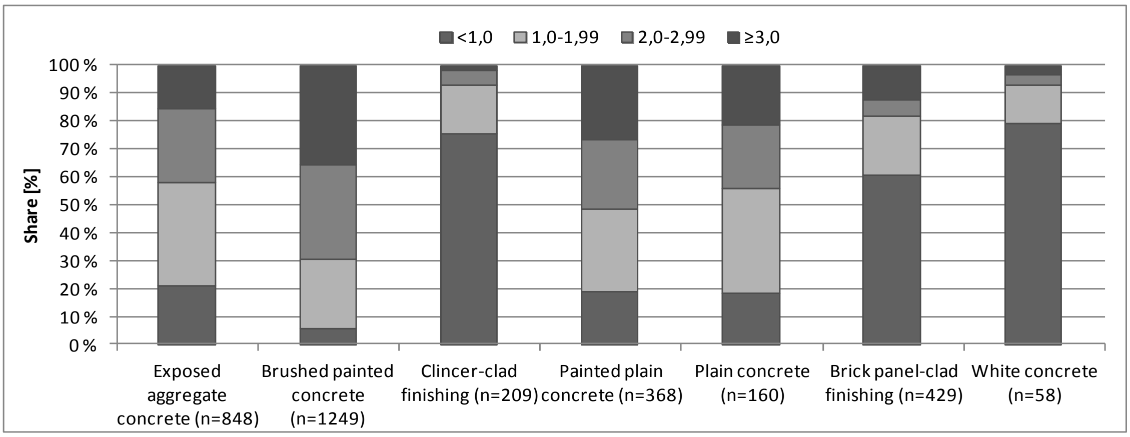


Fig. 5.3 Carbonation coefficients, k, of outer surfaces of facades by facade surface types.

There is wide deviation between concrete outer surface carbonation coefficients as can be seen from Table 5.9. Distribution of carbonation was wide also between samples drilled from different panels of the same building, which points to wide variation in concrete quality between different production batches. The concrete carbonation coefficient exceeds k=6 mm/a$^{0.5}$ with a few samples per facade surface type. The database includes only four samples of through-carbonated outer layers. With clinker-clad, unpainted brushed and floated, and white concrete surfaced panels the carbonation coefficient exceeds k=3 mm/a$^{0.5}$ only in a few isolated cases. Compared to earlier Finnish studies (Mehto et al. 1990, Heimala and Punakallio 1993, Huopainen 1997), the average carbonation coefficients of the material of this study are slightly smaller, but slightly larger than in Irish precast concrete buildings (Richardson 1990).

The carbonation coefficient is clearly linked to the porosity of concrete. Due to lower carbon dioxide diffusion resistance and a more extensive continuous capillary pore system, carbonation proceeds the fastest in concretes, where the share of capillary pores is largest, and the slowest in those with the smallest share of capillary pores, see Fig 5.4 and Appendix 5. On the other hand, the orientation of buildings has not been noted to have an effect on the carbonation rate of facades. The differences in the rain and sleet stress on different facades would thus not seem to have a crucial impact on carbon dioxide diffusion resistance – other properties of concrete, mainly porosity and the amount of carbonating material, appear more determining. Comparison of the carbonation coefficients of painted brushed facades to those of unpainted ones reveals that the paint coat of facades has a minor impact on carbon dioxide diffusion. Neither were any significant differences found when comparing the carbonation coefficients of concretes under solid and broken paint coatings. The thickness of the coat of paint applied to facades has not generally been determined in conditions investigations.

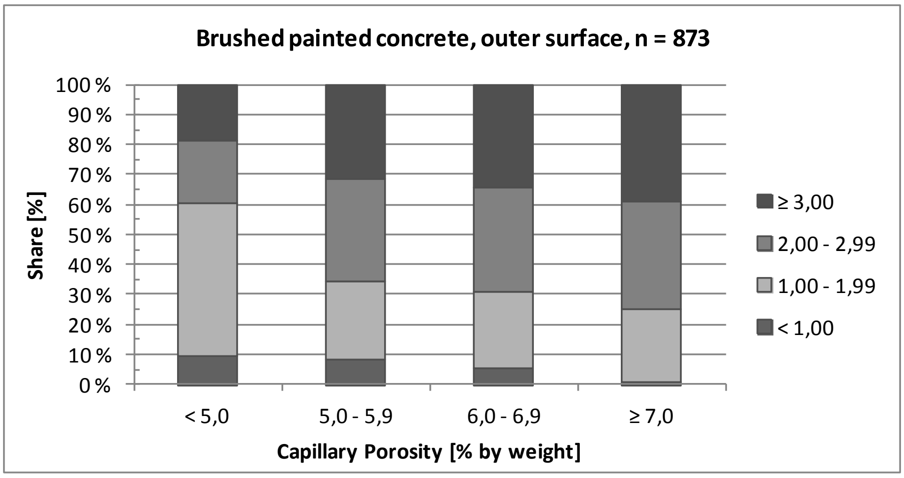


Fig. 5.4 *Shares of carbonation coefficients of concrete outer surfaces in various capillary porosity categories in brushed painted concrete.*

Sandwich panels' inner surface carbonation coefficients are quite low with all panel types compared to those of facade surfaces. This allows concluding that hardly any air circulates across the inner surfaces of panels despite the ventilation pipes in the elastic joints of panels and ventilation channels.

Thin-shell panels inner surfaces' average carbonation coefficients are clearly higher with all panel types than outer surface factors. The air space behind thin-shell panels allows air to circulate although the ventilation pipes in the elastic joints between panels and the ventilation channels are quite small, and ventilation is wind- and gravity-driven as the air in the ventilation gap warms. The significant differences between the carbonation coefficients of the inner surface and facade surface of the shell panel show that climate conditions have a clearly reducing effect on carbonation coefficients since the inner surface is always fully protected from rain and sleet.

Balconies

The carbonation coefficients of the outer and inner surface of the balcony side and parapet panels and the top and soffit of the slab have been compiled in Table 5.10, and the shares of carbonation coefficients of different panel types by categories are presented in Figure 5.5. Calculated carbonation coefficients are based on all values of each panel type.

Table 5.10 *Average carbonation coefficients, k, by panel types calculated on the basis of average carbonation depths of concrete measured from samples.*

		Carbonation coefficient, k [mm/a$^{0.5}$]				**No.**
		Av.	**Std. dist.**	**Min.**	**Max.**	
Side panel	outer surf.	2.61	1.52	0.00	16.63	901
	inner surf.	2.92	1.51	0.00	10.77	741
Slab	top	1.21	1.11	0.00	9.81	931
	soffit	3.08	1.40	0.00	11.19	884
Parapet	outer surf.	2.06	1.45	0.00	13.57	719
	inner surf.	2.17	1.31	0.00	13.57	687

There is wide deviation in the distribution of carbonation coefficients between all balcony elements both as concerns the outer and inner surfaces of structures and the tops and soffits of slabs, see Table 5.10. Deviation in carbonation is also wide between samples drilled from different panels of the same building, which proves the great fluctuation in the quality of concrete between production batches. The carbonation coefficient of concrete exceeds k=6 mm/a$^{0.5}$ only in a few samples per panel type, and the factor of slab tops exceeds k=4 mm/a$^{0.5}$ only in a few isolated instances. Samples of carbonated-through structures were drilled only from balcony parapets – a total of only three.

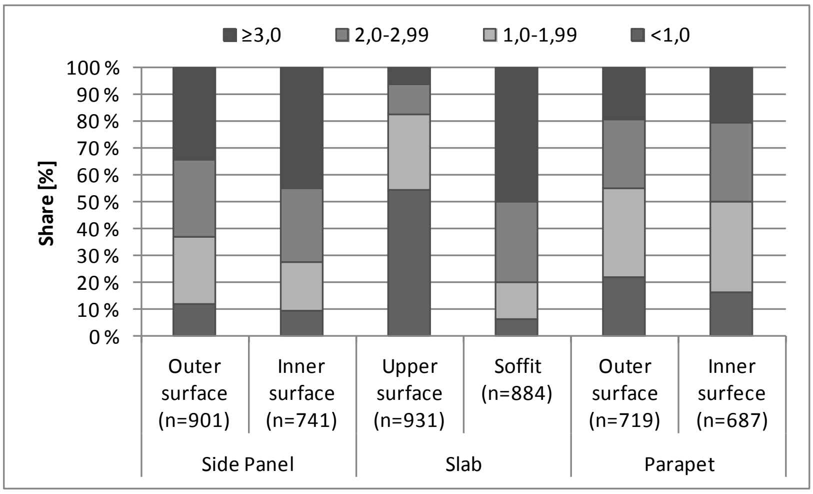


Fig. 5.5 Shares of different surface carbonation coefficients, k, of balcony elements in different categories by panel types.

The carbonation coefficient is clearly connected to the porosity of concrete, see Fig 5.6. An impermeable coat of paint has a noticeable impact on diffusion of carbon dioxide since the tops of balcony slabs are often coated with another type of paint than other parts of balconies. The porosity of concrete is uniform throughout the thickness of the slab meaning that the paint coat on the tops of slabs must have been considerably more impermeable than that on the soffits. Naturally, the paint coat on the topside only retards diffusion while it is unbroken.

Balconies are most often located on the southern-to-western facades of buildings, which has not allowed investigating the impact of building orientation on their carbonation rate. On the other hand, a small difference was detected in the average carbonation coefficients of the inner and outer surfaces of side and parapet panels in that the inner surfaces that are better protected from rain and sleet are slightly higher than those of the outer surfaces. The soffits of balcony slabs are completely protected from rain, which gives them the highest carbonation coefficients.

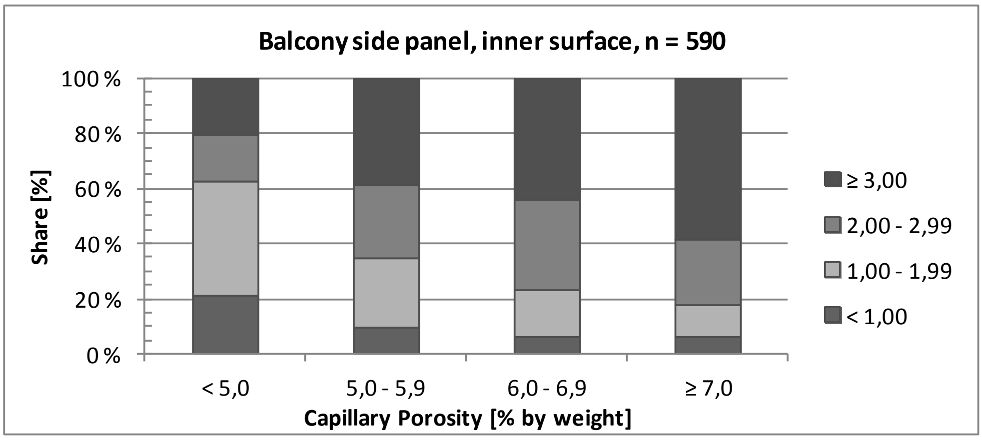


Fig. 5.6 Shares of balcony elements' carbonation coefficients in different capillary pore categories in balcony side panel.

The average carbonation rate of the concrete of the panels of all balconies built in the 1980's is only slightly slower than that of balconies built in the 1960's and 1970's. The carbonation rate of 1990's balconies is clearly lower than in previous decades. The reason is the general raising of the compression grade of concrete for all panel types, which has made concretes denser and increased the content of cement, the carbonating ingredient. Another significant factor is the lower water-cement ratio of concrete as the workability of concrete has been improved by air-entraining agents and plasticisers contrary to the 1960's and 1970's when workability was normally improved by adding water.

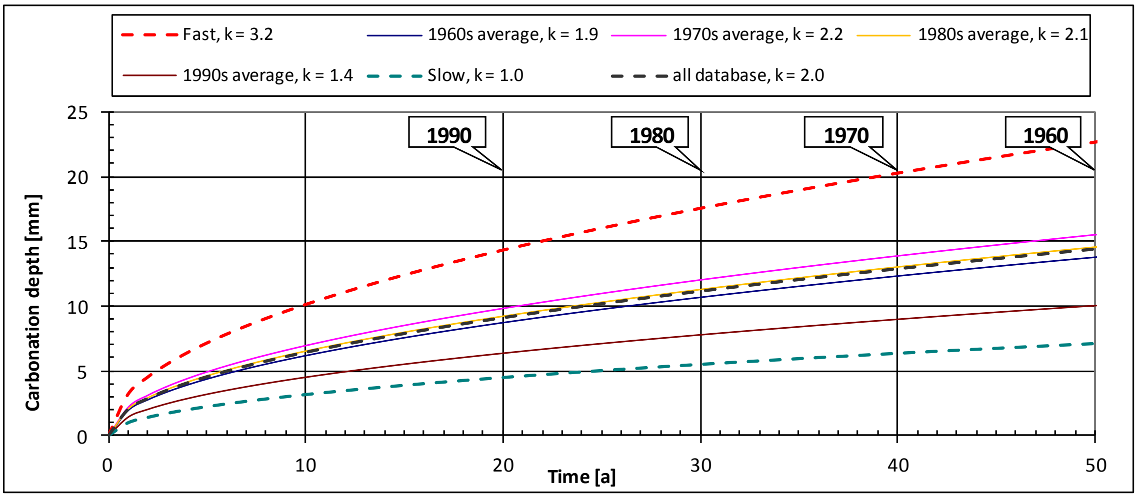


Fig. 5.7 Average carbonation of all balcony panels in various decades.

The phenomenon is similar in the case of facade panels, but it is not as noticeable since all facade surface types have not been produced in every decade.

5.2.2 Presence of chlorides

The chloride content of concrete of a total of 496 facade samples and 580 balcony samples has been determined. The Condition investigation manual for concrete facade panels (2002) considers 0.03-0.07 w% a critical chloride content for corrosion of reinforcements.

Facades. Seven samples from five spots in concrete facades exceed the above lower limit. The chloride contents of the samples are 0.04-0.09 w%, the average being 0.06 w%. The buildings are located in different parts of southern Finland. Three were completed in the 1960's and the other two in 1974 and 1979.

Balconies. Altogether 22 samples from balcony panels of 13 different buildings exceeded the lower limit of critical chloride content. The chloride content of the samples varied from 0.03-0.11 w%, the average being 0.06 w%. Espoo, Helsinki and Tampere are each home to three of the buildings, while Oulu, Lahti, Iisalmi and Hämeenlinna each has one. The buildings are partly the same ones that showed high chloride contents of facades. Nine of the buildings were completed before 1976, the remaining four in 1981, 1983, 1987 and 1988.

All in all, the critical chloride content required for inititiation of reinforcement corrosion was exceeded in only a few isolated buildings completed mainly before 1976. The use of chlorides as a concrete-hydration accelerator has been considered primarily a problem of on-site prefabrication plants of the 1960's. Thus, it was surprising to discover high chloride contents also in concrete panels manufactured in the 1970's, and especially in four buildings erected in the 1980's, where the side panels and slabs of balconies showed high chloride contents from the viewpoint of initiation of corrosion. Chlorides cannot have penetrated into the concrete after completion of any of the buildings, but must have entered into the concrete in connection with fabrication of panels intentionally or accidentally.

Although 3.8% of the samples from balconies (1.4% from facades) exceed the critical chloride content, reinforcement corrosion initiated by chlorides is on the whole extremely rare and is not regionally concentrated.

5.2.3 Corrosion damage

A large amount of corrosion damage has been observed visually in both facades and balconies in connection with condition investigations. Corrosion damage has generally occurred in areas where reinforcement cover depths have been smallest. As stated above, the critical chloride content for corrosion of reinforcements is exceeded only in a few cases meaning that the bulk of reinforcement corrosion is caused by concrete carbonation.

Facades. Visually observable reinforcement corrosion damage existed in 59% of the examined facades at the time of the condition investigation. The majority of the damage was local (54%); extensive corrosion was found only in 5.7% of the buildings. Correspondingly, 41% of the examined facades showed no observable corrosion damage at the time of investigation.

The corrosion damage was almost solely due to carbonation. As concrete carbonises, corrosion initiates first in reinforcements closest to the outer surface. The most typical localised corrosion damage is caused by the ends of reinforcement mesh wires at splices near the outer surface and the edge bars at the sides of window openings.

With the exception of white concrete and clinker-clad facades, carbonation has reached a depth of 5 mm in less than 10 years on average, and 10 mm in 20-30 years. The present average carbonation depth of a precast multi-storey residential building completed in 1970 is 11-20 mm depending on the facade surface type, see Fig. 5.8. In white concrete surfaced and clinker-clad panels carbonation has progressed only to a depth of about 7 mm. Thus, corrosion of steels closest to the surface has been possible in facades completed before 1980 for 20 years or longer.

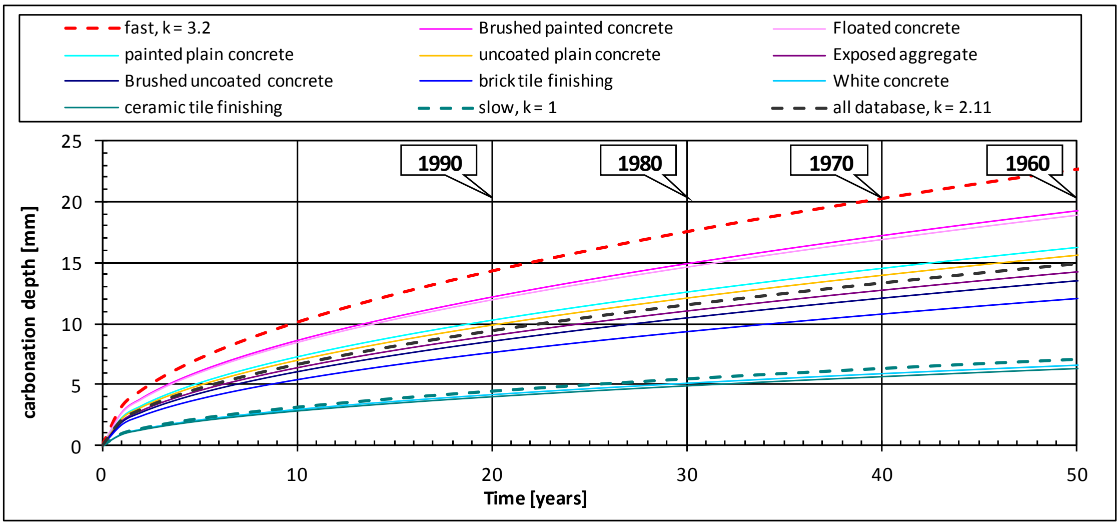


Fig. 5.8 Average carbonation depth of concrete according to surface finishing.

The share of reinforcements at under 5 mm depth is quite small and varies by facade surface types meaning that it naturally has a major effect on the amount of visible corrosion damage. A clear exception is the brick panel-clad facade, where the share of reinforcements with under 5 mm cover depth is remarkably large compared to all other facade surface types, but yet 60% of them have no visible or extensive corrosion damage. Thus, the brick panel retards significantly visible corrosion damage. Generally, the share of steels at less than 10 mm depths is also small: typically less than 2.5%. The share of under 10 mm cover depths in brick panel- and clinker-clad panels is also remarkably large compared to other facade surface types. The amount of extensive corrosion damage in clinker-clad panels is, however, of the same magnitude as e.g. in exposed aggregate or brushed painted panels. In clinker-clad panels concrete carbonation only progresses in the joints between clinker tiles meaning that carbonation can only have migrated to the depth of the steels at isolated points. Clinker tiles may cause capillary rise of water into the facade concrete only through the joints between clinker tiles. In the pore system of the concrete, the water distributes evenly also behind the clinker tiles whereby the clinker-clad concrete of the facade does not get as thoroughly wet to the reinforcement during rain and sleet as e.g. an exposed aggregate or a brushed painted concrete facade. On the other hand, clinker cladding slows the drying of concrete, which means that the moisture content of a clinker-clad panel may be higher than that of an exposed aggregate one after a long period of rain and sleet, as the latter dries more easily.

Most extensive corrosion damage to exposed aggregate and brushed painted panels occurs in the southern coastal areas. Similar correlation between damage in southern coastal areas and inland has not been observed with other facade surface types.

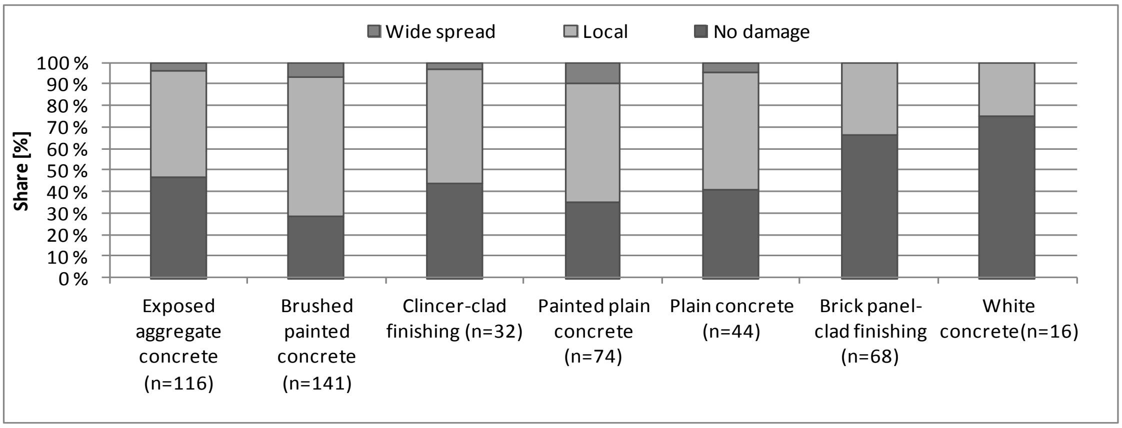


Fig. 5.9 Shares of visible corrosion damage by facade surface types, n= 491 buildings.

Correlation can be noticed between the shares of visible corrosion damage in different facade surface types and the distributions of carbonation coefficient shares and degree of capillary saturation of concrete, see Fig. 5.3, 5.4 and 5.9. The more capillary the concrete, the faster the carbonation of concrete propagates, and the more visible corrosion damage appears.

The carbonation of the inner surfaces of sandwich panels has been so slow that the stringers of the trusses still generally lie in uncarbonated concrete and are thus well protected against corrosion. The fastening reliability of the outer layers of sandwich panels has consequently not been compromised by the corrosion of reinforcements.

Balconies. Visually observable corrosion damage to reinforcements was detected in 66% of all examined balcony panels at the time of investigation. Most corrosion damage is local (51%). However, extensive corrosion of reinforcement was found in 15% of the buildings. Visually observable corrosion damage was not found in 34% of examined balconies.

Despite individual high chloride contents, corrosion damage has generally been caused by concrete carbonation.

The average carbonation rate of balcony elements is faster than that of facades. Carbonation has propagated to a depth of 5 mm in less than six years on average, and to 10 mm in about 15-22 years. Carbonation advances significantly faster in the soffit of balcony slabs than its side and parapet panels. Presently, the carbonation of the soffit of a balcony slab manufactured in 1970 has progressed on average to a depth of about 20 mm, and that of a slab manufactured in 1980 to a depth of about 15 mm, see Fig. 5.10. Reinforcement corrosion has thus been possible in a quite significant portion of the soffit reinforcements of balcony slabs manufactured before 1980. Carbonation of side panels and parapets of the same age has also propagated quite deep subjecting a significant portion of the reinforcements to corrosion.

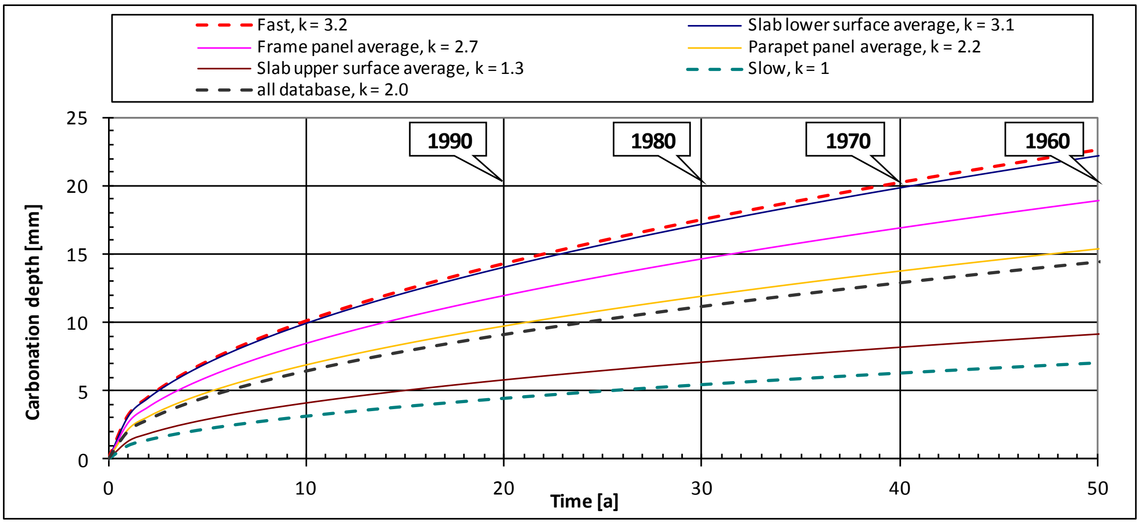


Fig. 5.10 Average carbonation depth of concrete of balcony panels.

Extensive and local visible corrosion damage occurs mostly in balcony side panels, 76% of whose edge bars generally show some degree of corrosion damage. The share of reinforcements at under 5 mm depth in parapet panels is 1.5% on the outer surface, which is nearly three times the reinforcements of side panels at corresponding depth. Yet, no visible corrosion damage has been detected in 33% of the buildings, and the share of extensive corrosion damage in parapet panels is smaller than in side panels. The concrete of balcony parapets appears to be of a higher grade: the average carbonation rate of parapet panels is lower and the share of capillary pores smaller than in side panels.

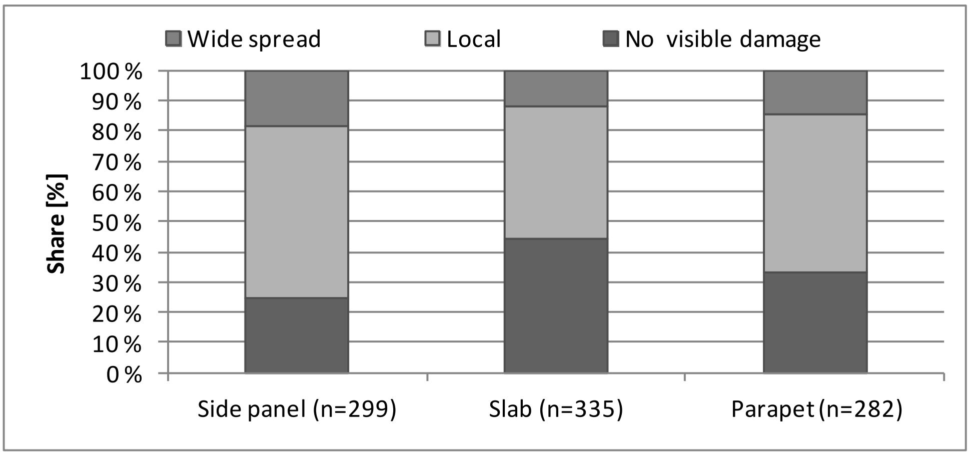


Fig. 5.11 Share of visible corrosion damage in different balcony elements.

Although the carbonation of concrete has progressed considerably in the soffits of slabs, there is remarkably little visible corrosion damage compared to small cover depths: no corrosion damage was visible in 148 buildings out of 335 at the time of investigation. The share of extensive corrosion damage in balcony slabs was 12%. Carbonation of concrete is fastest in the soffits of slabs, which means that the reason for the lesser visible corrosion damage compared to other balcony panels is the low moisture stress level. The soffit of the slab is completely protected against rain and sleet. It can get wet only when water migrates through the slab or flows onto the soffit over the edges or due to a defective drainage system, see Fig. 5.12 and 5.13.

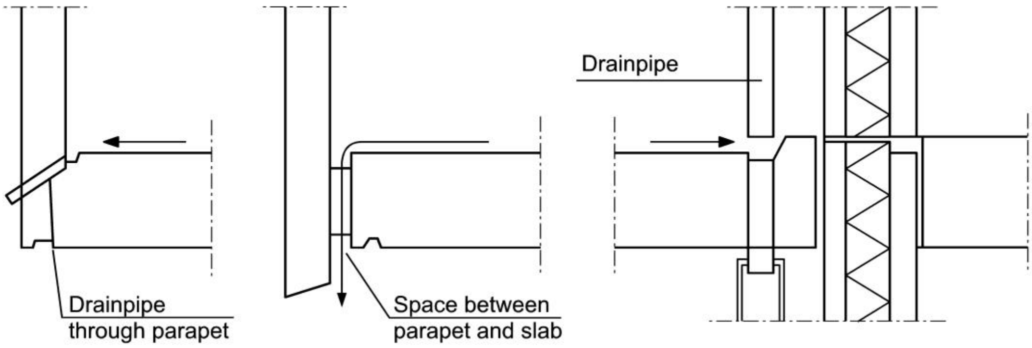


Fig. 5.12 Different balcony drainage methods. From left: Pipe through parapets, gap behind parapet and drain pipe in the corner.

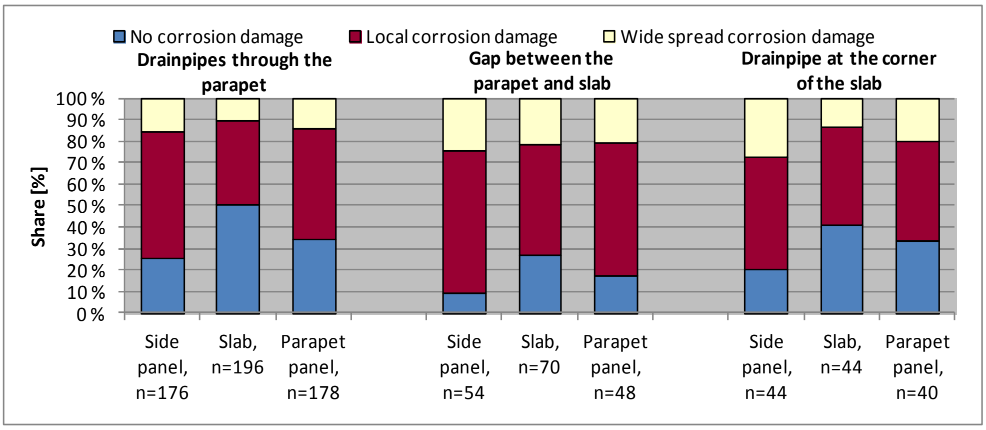


Fig. 5.13 *Share of visible corrosion damage in different balcony panels by drainage systems.*

There is clearly the most visible corrosion damage to all balcony panels when drainage is effected through a gap between the parapet and the slab. Especially the corrosion damage shares to side panels and slabs are much larger than with other drainage methods. Drainage through a gap between the parapet and slab allows rainwater to flow over the front edge of the slab onto to soffit thereby causing a local high moisture stress level. Water flowing down from the slab wets the balcony slabs and parapets below as well as the side panels due to the effect of winds.

Annual amount of rain and sleet

The corrosion rate of reinforcement in carbonated concrete depends strongly on the moisture content of the concrete. According to Tuutti (1982), the corrosion rate increases sharply as relative humidity of concrete exceeds 85%. The moisture stress on a carbonated structure does thus essentially affect the corrosion rate of reinforcements. According to Mattila (2003), the corrosion rate of uncovered concrete is high if the annual amount of rain and sleet is approximately 480 mm or higher.

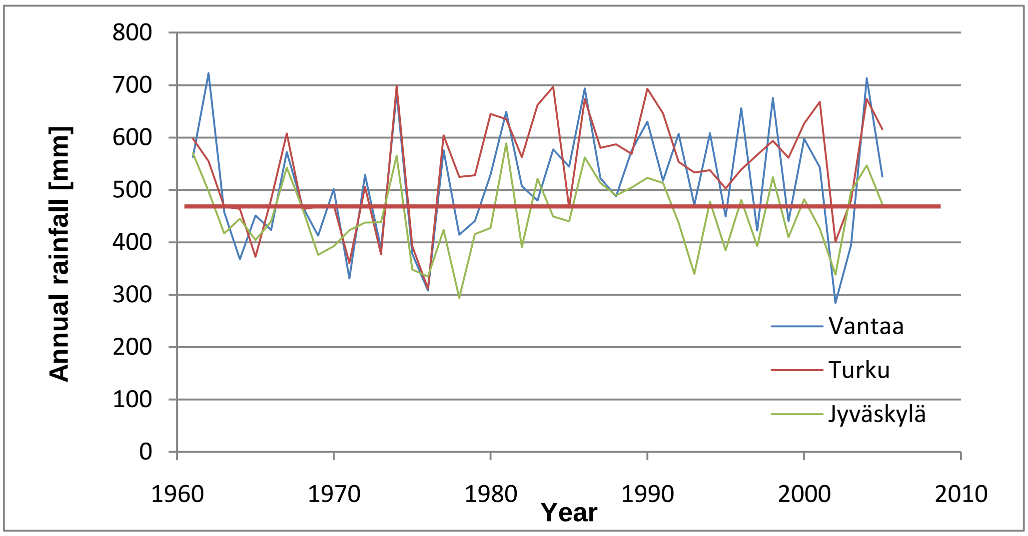


Fig. 5.14 *Annual amount of rain and sleet in Jyväskylä (inland), Vantaa (southern coastal area) and Turku (southern coast) between 1961 and 2005. The horizontal line depicts the annual rainfall that will contribute to a faster corrosion rate.*

In the southern coastal area (Turku) and southern Finland (Vantaa) the amount of annual rainfall is higher compared to inland (Jyväskylä). In the southern coastal area and southern Finland annual rainfall has also exceeded the critical amount of 480 mm per year most years, especially since the 1980's. In the 1980's the carbonation of concrete propagated to reinforcement lying near the surface in most facade and balcony panel types manufacture in the 1970's. Thus, the higher amounts of rain and sleet could be a reasonable explanation for the wider corrosion damage in the southern coastal area than inland.

The weather observations of FMI also indicate that the amount of rain and sleet is the highest in autumn when the relative humidity of outdoor air is typically high. These two factors keep concrete structures wet for long periods, which keeps the corrosion rate at a high level.

Influence of prevailing wind directions

The annual amount of rain and sleet does not fall uniformly across all facades. The distribution depends on the height of the building and the prevailing wind directions during rain and sleet. Prevailing wind directions and wind speeds largely determine on the distribution of rain and sleet on a building. Most of the liquid precipitation comes with southerly to westerly winds in all parts of Finland. Rain events with wind from other directions have been rare, see Fig. 5.14 and Appendix 7.

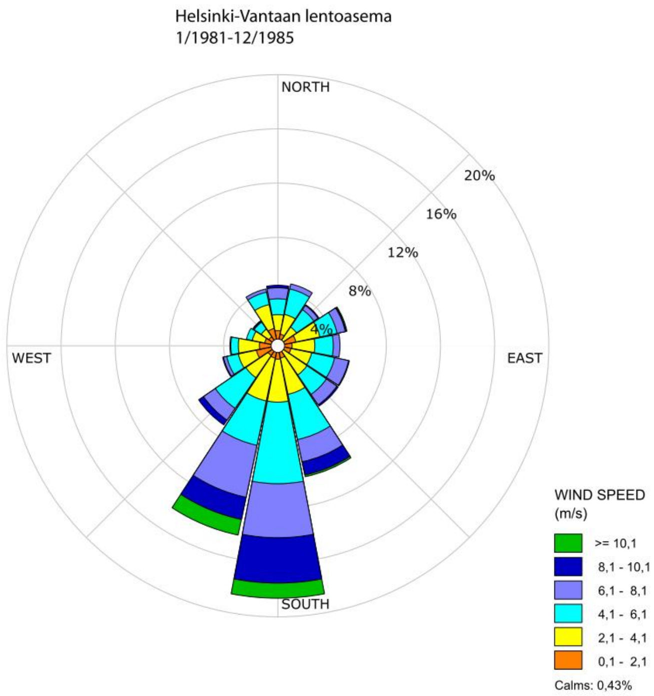


Fig. 5.14 Example of prevailing wind directions during liquid precipitation at Helsinki airport from Jan. 1981 to Dec. 1985.

The amount of rain and sleet falling on facades and balconies depends on the drop-size distribution of rain, the velocity of falling drops, wind speed, building height, protection offered by the environment, structures guiding wind, etc. According to FMI, the average velocity of falling rain drops is 4-8 m/s, which means that an average of 40-60% of the rainfall hits facades. During high winds, which occur more frequently in southern Finland, a larger portion of the rainwater hits the vertical surfaces of facades.

According to Jerling and Schechninger (1983), the upper parts and corners of facades receive more rainfall than lower and central parts. A practical example of the effect of wind direction on the distribution of rainfall on a facade can be seen in Fig. 5.15: The upper section of the end of the precast concrete building is already wet while the lower part is still dry, and no rain is falling on the facade with windows.

Fig. 5.15 End of a concrete building at the beginning of a rain event. The upper sections receive more rain than the lower ones.

Hardly any corrosion damage can be detected on the northern or eastern facades despite the reinforcement being embedded in carbonated concrete as on the southern and western facades. The high outdoor relative humidity in Finland during winter is not high enough to increase the humidity of carbonated concrete to the extent that corrosion initiates or propagates relatively fast, which is evidenced e.g. by the relatively slow corrosion of the soffits of concrete balcony slabs and the minor corrosion of the reinforcements of facades that are protected from rain and sleet or receive very little of them. Thus, a high corrosion rate is not possible without rain and sleet. In Finland, the prevailing wind directions during rainfall are southerly to westerly. Snowfall, which usually is accompanied by northerly winds, cannot be absorbed in the pore structure of concrete. That is a reasonable explanation for more corrosion damage occurring on southern and western facades than northern and eastern ones.

5.3 Disintegration of concrete

This chapter looks at the frost resistance properties, degradation observations, occurrence of frost damage and climatic factors affecting disintegration related to concrete facades and balconies.

5.3.1 Frost resistance of concrete

Generally, condition investigations have examined the frost resistance of concrete in outdoor conditions by protective pore tests. The tests have often been complemented by some thin-section analyses aimed mainly at determining the existence and degree of frost damage as well as the degree of filling of the concrete's pore system. $p_r \geq 0.20$ is considered a pore ratio that provides protection against frost (Finnish concrete code 1980). Correspondingly, $p_r < 0.10$ is a pore ratio that makes concrete non-frost-resistant in Finnish outdoor conditions.

Facades

The protective pore ratios of concrete have been compiled in Table 5.11 while the protective pore ratio distributions are shown in Figure 5.17 and air-entrainment determined on the basis of thin sections in Figure 5.18. The tables show that the frost resistance of concrete differs considerably between various surface types of facades.

Table 5.11 Protective pore ratios by facade surface types measured from samples.

	Protective pore ratio, p_r				No.
	Av.	**Std. dev.**	**Min.**	**Max.**	
Exposed aggregate	0.11	0.07	0.00	0.40	573
Brushed painted	0.17	0.09	0.00	0.62	883
Clinker-clad	0.11	0.08	0.00	0.34	188
Painted form-finish	0.13	0.10	0.00	0.88	238
Unpainted form-finish	0.12	0.09	0.00	0.49	109
Brick panel-clad	0.17	0.11	0.00	0.47	243
Brushed unpainted	0.16	0.05	0.08	0.27	24
White concrete	0.20	0.11	0.00	0.40	40
Floated unpainted	0.12	0.07	0.05	0.23	12

The protective air-entrainment of all facade surface types has been highly successful in some cases and failed completely in others. Wide variation may exist between individual buildings. The protective pore ratios of concrete vary even between samples drilled from different panels of the same building. Variation is typically quite small when the concrete used for the building is of low frost resistance with a protective pore ratio $p_r < 0.10$. In buildings where an attempt has been made to ensure the frost resistance of concrete by air-entrainment, there are larger deviations between samples from the same building. That is most probably due the fact that people have been experimenting with the use of air-entrainment agents in concrete mixing, which has resulted in varied quality. Use of air-entrainment agents in concrete mixing was not systematic before the 1980's.

The porosity protection of Finnish concrete facades can be considered quite inadequate on the whole. The worst it is in the case of exposed aggregate, clinker-clad and unpainted form-finish facades. The protective pore ratio, p_r, of about half of these facade surface types is under 0.10, i.e. they have received no effective porosity protection. Air-entrainment has been most successful with painted brushed and brick panel-clad and white concrete facades. They exceed the concrete code requirement $p_r \geq 0.20$ by 32%, 41% and 54%, respectively, see Fig. 5.17.

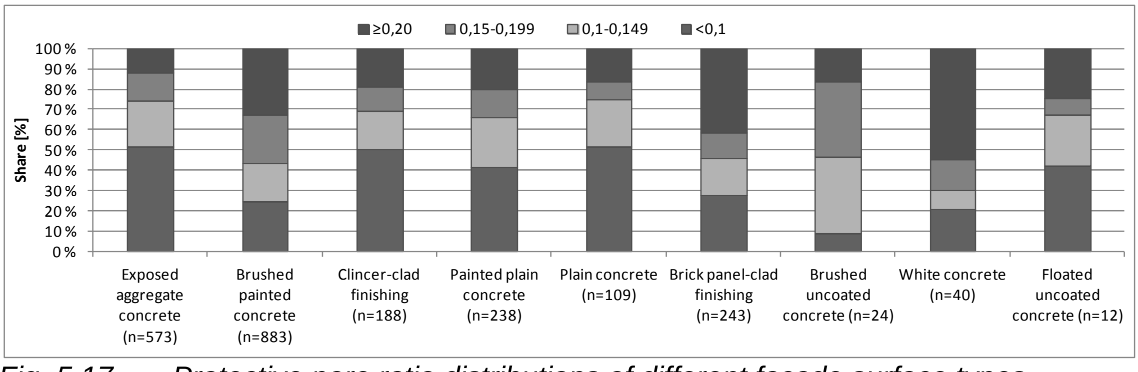


Fig. 5.17 Protective pore ratio distributions of different facade surface types.

Thin-section analyses indicate that the concrete of facades is even less frost resistant than can be assumed based on protective pore tests, see Fig 5.18. Thin-section analyses determine the frost resistance of concrete on the basis of the spacing of protective pores, which in the case of frost resistant concrete must be less than 0.25 mm (Koskiahde 2004).

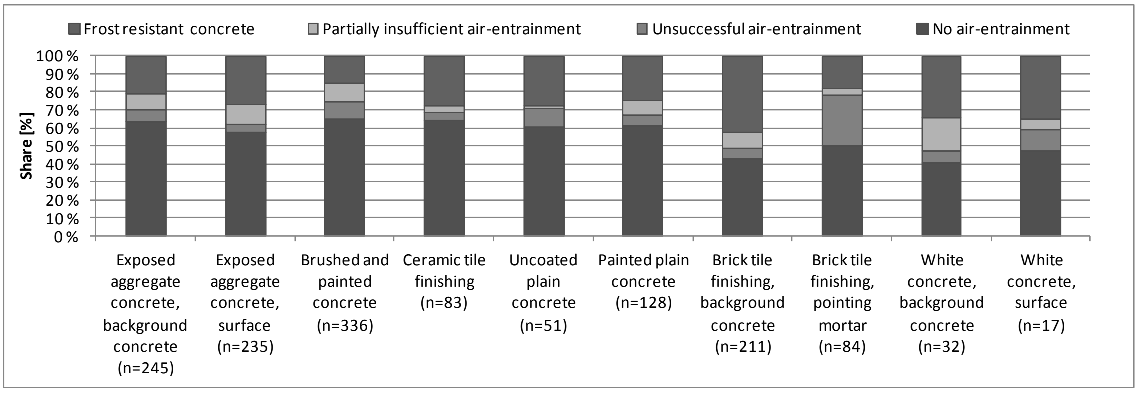


Fig. 5.18 Distribution of air-entrainment in different facade surface types according to thin section analyses.

The number of conducted thin-section analyses is about half of that of protective pore analyses. Both a protective-pore-ratio test and thin-section analysis has been made of only about 10 per cent of concrete samples per surface type. Examination of these small samples reveals a corresponding difference in frost resistance distributions as in the above Figures 5.17 and 5.18. According to both the protective-pore-ratio test and the thin-section analysis, the shares of concrete considered frost resistant are approximately the same by both methods. However, the so-called gray zone between high and low frost resistance is clearly narrower based on thin-section analyses than could have been assumed based on the protective-pore-ratio test.

No regional differences are noticeable in the protective pore ratios or frost resistance based on thin-section analyses of concretes used for facades, which proves that concrete technology has been equally developed or undeveloped in different parts of the country.

Balconies

The protective pore ratios of concrete samples drilled from the side panels, slabs and parapet panels of balconies are compiled in Table 5.12 while the protective pore ratio distributions of different panel types are shown in Figure 5.19, and the air-entrainment of concrete based on thin sections in Figure 5.20.

Table 5.12 Measured protective pore ratios of concrete samples drilled from balcony panels by panel types.

	Protective pore ratio, p_r				**No.**
	Av.	**Std. dev.**	**Min.**	**Max.**	
Side panel	0.09	0.07	0.00	0.45	721
Slab	0.10	0.07	0.00	0.42	711
Parapet	0.12	0.08	0.00	0.52	487

The protective air-entrainment of all balcony panel types has been highly successful in some cases and failed completely in others. Wide variation may occur between individual buildings. The protective pore ratios of balcony panels vary even between samples from different panels of the same building. The variation in protective pore ratios is similar to that of facade panels and for the same reason.

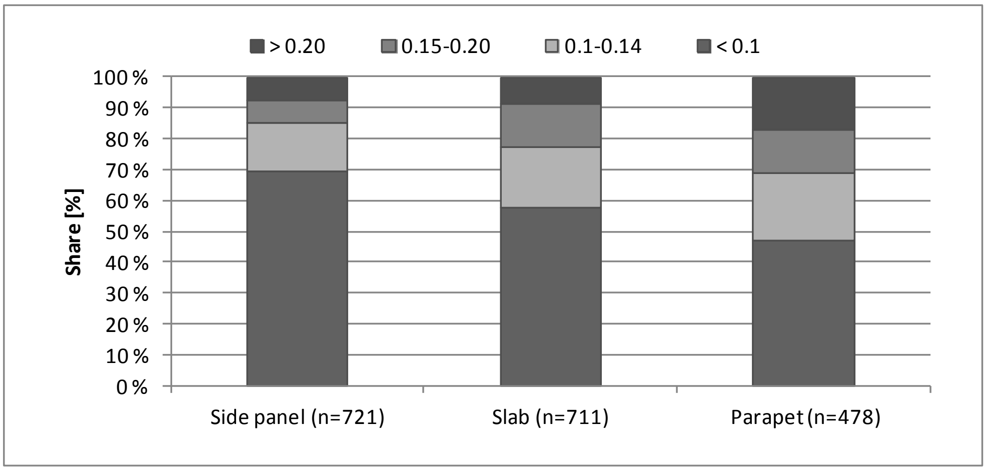


Fig. 5.19 Distribution of protective pore ratios in different balcony panel types.

The situation with respect to frost resistance is the worst with balcony side panels, of which 69% have a protective pore ratio, p_r, under 0.10. That is the case with 57% of the slabs and 47% of the parapets, see Fig. 5.19. Thus, the protective pore ratios of balcony side panels and slabs are generally clearly worse than those of any facade surface type.

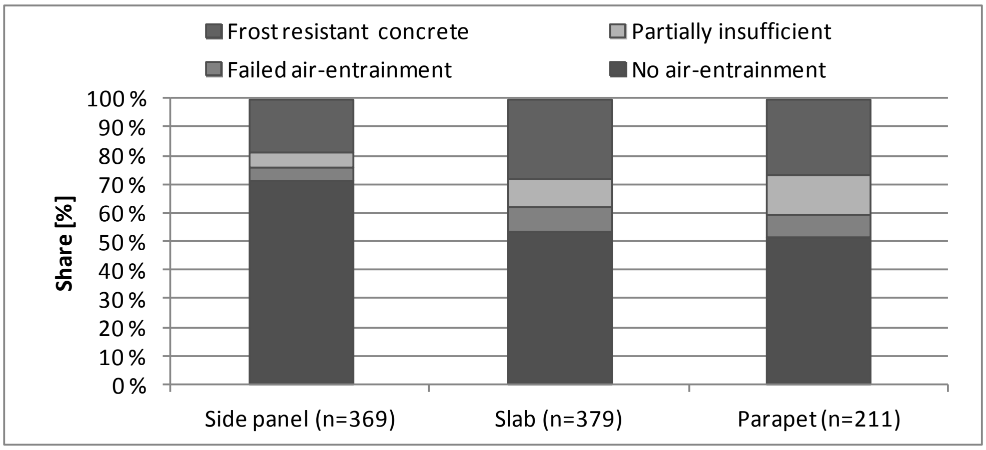


Fig. 5.20 Distribution of protective air-entrainment in different balcony element types according to thin-section analyses.

According to thin-section analyses, the share of concrete of poor frost resistance in balcony panels is about the same as indicated by protective-pore-ratio tests, see Fig 5.20. On the other hand, the share of highly frost resistant concrete in all balcony panels is clearly larger than can be assumed based on the protective pore test. Thin-section analyses have focused mainly on different concrete samples than protective-pore-ratio tests. No regional differences have been detected in the protective pore ratios or frost resistance determined by thin-section analysis of concrete used for balcony panels.

The poor frost resistance of side panels can be regarded as a significant factor limiting the service life of the entire concrete panel balcony stock since they are bearing structures, which cannot be replaced without tearing down the entire balcony structure. The frost resistance of slabs is also poor to a large extent. The top of the slab has, however, often been coated with quite dense paint which, while unbroken, effectively prevents carbonation of the concrete on top as well as rainwater from penetrating into the concrete. That has been previously determined on the basis of the carbonation depths of the tops and soffits of balcony slabs and the quite slight corrosion damage to the soffit, see Chapter 5.2. Thus, the poor frost resistance of balcony slabs cannot be considered as big a risk to the bearing capacity and end of service life of the balcony as that of side panels.

Influence of design standards on frost resistance

Attempts have been made to improve the frost resistance of concrete by means of concrete technology and design standards. Important years in this respect were 1976, when guidelines for air-entrainment of concrete structures exposed to outdoor conditions (Durability of concrete 1976) were given, and 1980 when testing of air-entrainment guidelines were incorporated in the official concrete code governing concrete construction (Finnish concrete code 1980), see Chapter 1.2.3. There is proof that air-entrainment agents have been used before the publication of the design standards more or less randomly without always achieving the desired end result.

Based on the protective pore ratios of the research data, the use of air-entrainment with balcony panels began in 1976 and increased slowly thereafter. Only after 1981 did air-entrainment start to become a standard feature in precast concrete production, which started to increase considerably the frost resistance of the building stock completed subsequently, see Fig. 5.21.

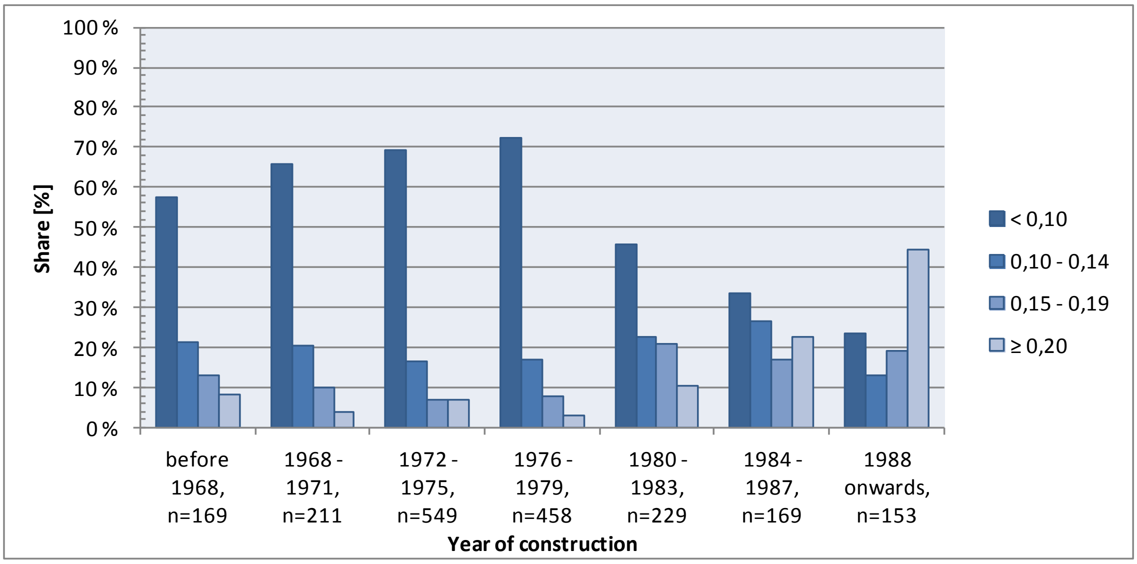


Fig. 5.21 Distribution of protective pore ratios of all balcony panels by four-year periods. Number of samples, n, is 1 938.

Use of air-entrainment began a little earlier with concrete facades than balcony panels around the mid-1970's. Increasing the frost resistance of different facade types appears to have been very challenging since the protective pore ratios of concrete started improving substantially only in 1984, see Fig. 5.22. Concrete facades are not a uniform group in the sense that balconies are since facade types have changed along with architectural trends over the decades. Yet, the graph shows clearly the general trend in the development of concrete facades' frost resistance.

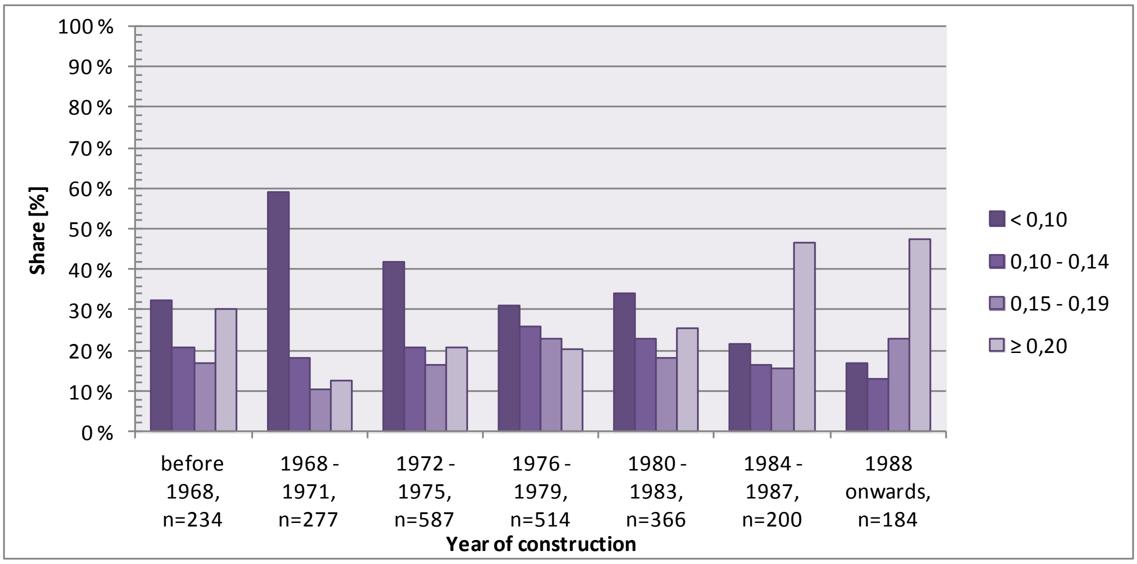


Fig. 5.22 Distribution of protective pore ratios of all facade panels by four-year periods. Number of samples, n, is 2 362.

More than half of the present precast building stock was built in 1960-1979 (Statistics Finland 2010), a period when the frost resistance of concrete clearly was inadequate. Hardly any attention had been paid to the durability of concrete structures under climatic stresses in design standards prior to 1976. The inadequate frost resistance of this large precast building stock can thus be considered a significant risk for the value of the country's national wealth.

Concrete sector guidelines and national design standards and requirements played a crucial role in the improvement of frost resistance of concrete in the 1980's. Especially the systematic use of air-entraining agents and wider introduction of various test methods have essentially improved the frost resistance of the concrete used for facade and balcony panels.

Despite systematic air-entraining, the frost resistance of the concrete of 24% of the balconies and 17% of the facades built in 1988 and since is poor. The most probable reasons for that are the general increase in concrete strength at the end of the 1980's and early 1990's, the move toward finer cements, the general strive to lower the water-cement ratio, and the introduction of chemicals that improve the workability of concrete. These changes that influenced concrete technology were intended to increase the service life of concrete structures. The changes in concrete technology were quite fast and the combined effects of all variables were not taken into consideration. On the other hand, higher concrete strength indirectly improves concrete's frost resistance meaning that unsuccessful air-entrainment does not necessarily lead to a short service life.

5.3.2 Frost damage

Visible frost damage

Visual inspection of concrete structures can reveal only far advanced frost damage and its extent. Visible frost damage of facades and balconies has been recorded in condition investigation reports under building-specific data, which means that these data can only be compared suggestively to data on frost damage derived from individual samples drilled from structures.

Facades. Visible frost damage in concrete facades was reported in the case of a total of 811 buildings. Visually observable degradation at the time of the condition investigation had occurred in all facade types in 43% of all buildings. Most frost damage had been local (35%); extensive far advanced damage had been suffered by 7.3% of the buildings. Altogether 57% of the examined facades showed no visually observable frost damage at the time of the condition investigation.

There are significant differences in visible frost damage occurrence between different facade surface types, see Fig. 5.23. The most local and extensive damage occurs in exposed aggregate facades, of which 46% had sustained local and 16% extensive damage. No layers retarding the absorption of rainwater exist on the surface of exposed aggregate, like a coat of paint or brick panels or clinker tiles, which allows rainwater easy access into the pore system of the concrete. Clinker-clad and painted form-finish facades also showed more than average local and extensive frost damage.

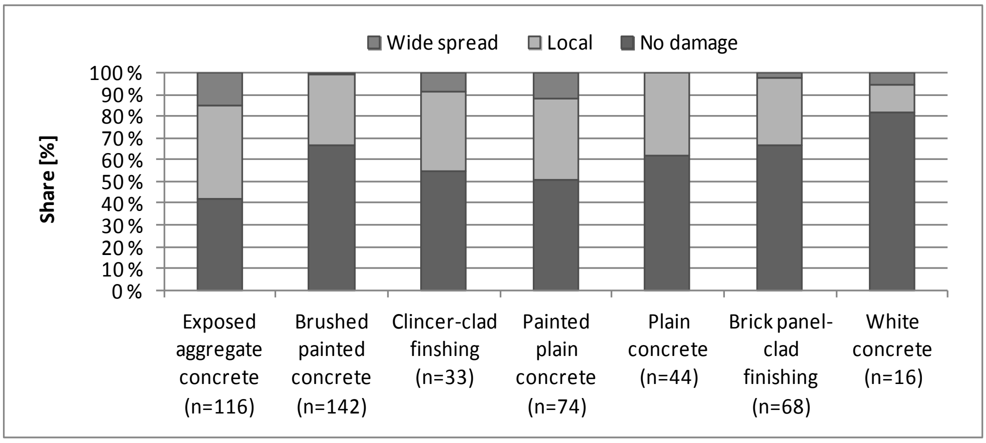


Fig. 5.23 Distribution of visually observed frost damage in different facade types.

There is clear correlation between visually observable frost damage of facade surface types and their frost resistance, see Figs. 5.17, 5.18 and 5.23. Those facade surface types with a smaller proportion of inadequate frost resistance naturally also showed less visually observable frost damage.

Local frost damage typically occurs in the upper corners of a building and at the edges of panels. Far advanced frost damage increases the volume of concrete, which typically causes the edges of damaged panels to curl outward from the facade surface. The deformation due to concrete degradation is permanent. In extensive far advanced concrete degradation, the entire panel may have swollen and curved strongly, often compressing the elastic joints between panels together. Thus, swelling has varied in different layers of the concrete, see Fig. 5.24. Generally 5-10% more visible frost damage occurs at higher storeys than lower ones. Especially the share of extensive damage increases at higher storeys compared to lower storeys. An exception is exposed aggregate facades, where extensive frost damage often occurs quite uniformly across the entire facade height.

Fig. 5.24 On the left, local frost damage at the edge of an exposed aggregate concrete panel. On the right, far advanced and wide spread frost damage to exposed aggregate concrete panels.

Wide local variation occurs in visible frost damage between different facade orientations. All facades suffer frost damage, but extensive damage occurs most in the case of all facade types on facades facing south to west.

The occurrence of visible frost damage is typically 10 percentage points higher in southern coastal areas than inland. A zone about 10 km wide along the sea coast has the most frost damage. An exception is brick panel-clad facades, which have suffered 40% more visible frost damage in the southern coastal area than inland. Precipitation in the form of water and sleet is higher in the coastal area than inland, see Fig. 5.14, and therefore it is natural that frost damage of concrete is more severe there due to stronger moisture stress. White concrete facades occur practically only in the southern coastal area, which means that they cannot be included in the regional investigation.

Brushed painted facades are also an exception since 11 percentage points more visible frost damage occurs in them inland than in the southern coastal area. Yet, all extensive damage has occurred on the coast. All in all, brushed painted panels suffer nearly as much frost damage as brick panel-clad facades, see Fig. 5.23. The frost resistance properties of these facade types are also very close to each other, see Figs. 5.17 and 5.18. However, brick panel cladding is highly capillary and can quickly absorb water into its pore system during rain, from where it migrates to the backing concrete allowing the initiation and propagation of frost damage under suitable conditions. An unbroken coat of paint on a brushed painted panel may reduce the wetting of concrete. Buildings located in the southern coastal area show more local and extensive flaking of painted surfaces than those located inland. One explanatory factor is local weather conditions and the protection offered by the immediate surroundings of the buildings since it generally rains more in coastal areas than inland.

No correlation between visible frost damage and capillarity of concrete was found with any facade type. The thickness of thermal insulation and the heat flow through it did not have a significant influence on the occurrence of frost damage in any facade type either. The thermal transmittance of even the oldest sandwich panels is at least 0.63

W/m^{2}K, see Table 5.8, which means that the heat flow through the external wall, and its drying effect, are relatively small. The durability properties of a facade panel, such as frost resistance of the concrete, and prevailing stress conditions are much more determining factors in the realisation of frost damage.

Balconies. Visible frost damage was recorded in condition investigation reports from a total of 335 buildings. Visible frost damage was found in 27% of all balcony element types at the time of investigation. Most damage had been local (22%), extensive far advanced degradation was detected in 5.6% of the buildings. In 73% of the investigated balconies no degradation could be observed visibly. Significant differences existed in occurrence of visible degradation between balcony panel types, see Fig. 5.25. The most local and extensive frost damage was found in side panels, of which 27% had suffered local and 8.6% extensive damage. Of slabs and parapet panels 25% and 22%, respectively, showed frost damage. Side panels received rain and sleet stress from as many as three directions due to winds, which means that the stress on them may be considerably heavier than on the slab and parapet.

Compared to the generally quite deficient frost resistance of balcony panels, the share of wide spread and far advanced visible frost damage is remarkably small, see Figs. 5.19, 5.20 and 5.25. Frost damage occurred mostly in side panels, which also have the poorest frost resistance, and the least in parapet panels, which had been produced in a way that gave them the best frost resistance of any balcony panel. Thus, it is logical that visible damage exists. The surfaces of balcony panels are typically painted, which means that an unbroken and dense coat of paint may have reduced the wetting of concrete and consequently the occurrence of frost damage.

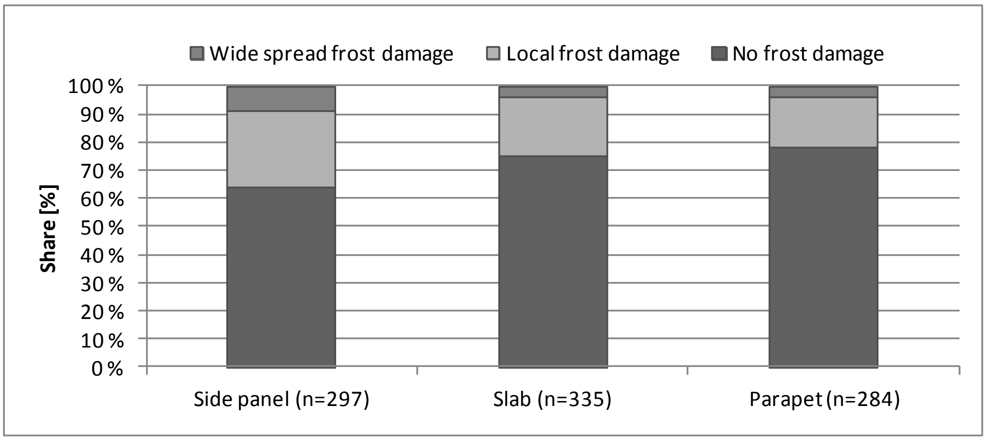


Fig. 5.25 Distribution of visually observed frost damage in different balcony elements.

Local frost damage typically occurs at the upper sections of balconies and, especially, at the front edges of side panels that receive the most rain and sleet. Far advanced frost damage degrades hydrated cement and destroys the bond with the aggregate resulting in a large reduction in concrete strength. Extensive far advanced frost damage lowers the bearing capacity of slender side panels and, especially, the bearing columns of balconies.

Fig. 5.26 *On the left, local frost damage in the front edge of a balcony side panel. On the right, far advanced and extensive frost damage in balcony column. Both are load-bearing structures.*

Dwelling-specific balconies are typically located along one facade of a building; the opposite facade may have so-called airing balconies accessed from the stairwell. The dwelling-specific balconies most often face south-east to west, which means that visible damage cannot be investigated according to different orientations. Slightly more visually observable degradation occurred in the outermost stacked balconies than in those in the middle.

Drainage of balconies had been implemented by so-called spout pipes through the parapet (63% of buildings), a gap between the parapet and slab (23% of buildings) or by a drain pipe at the corner of the slab (14% of buildings), see Fig. 5.12. Drainage or rainwater through pipes penetrating the parapet and gaps between parapet and slab can be considered uncontrolled since in both cases the water draining away wets the structures underneath increasing their moisture stress and consequently the risk of far advanced frost damage. Clearly the most visible frost damage appears in balcony panels when drainage is by a gap between parapet and slab, see Fig. 5.27. Drainage through the parapet generally wets the parapet and side panels underneath, which is also clearly reflected in amounts of frost damage.

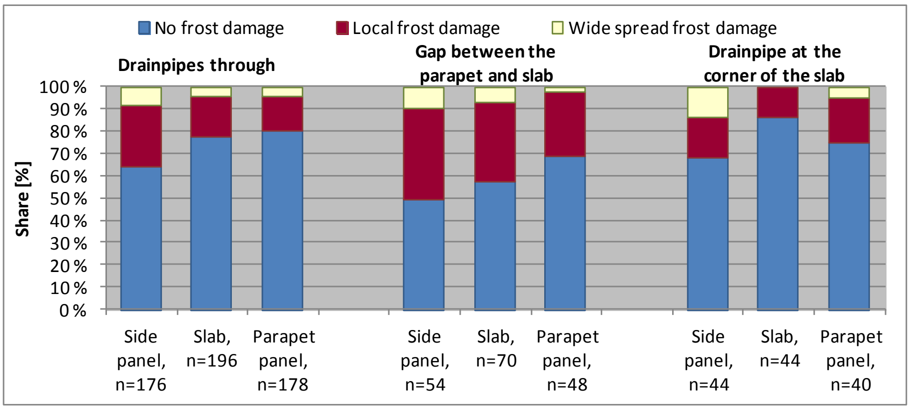


Fig. 5.27 *Influence of balcony water drainage system on visually observable frost damage in different balcony panels.*

A water drainage system based on a pipe installed through the balcony slab to ground causes extra moisture stress on surrounding structures only in exceptional flooding situations when the system freezes or gets otherwise blocked. In the most recent building stock balcony drainage is implemented by a drain pipe, whose frost resistance has been found to be better, on average, than earlier.

Frost damage according to laboratory analyses

The frost damage of concrete samples drilled from a structure has been analysed either by a thin-section analysis or a tensile strength test in a laboratory. A thin-section analysis can also detect incipient cracking indicating frost damage. Frost attack of concrete breaks the internal structure of concrete whereby the tensile strength of concrete weakens considerably faster than its compressive strength.

Facades. The distributions of cracking indicating frost damage revealed by thin-section analyses of concrete samples from facades are presented in Figure 5.28 and the tensile strength test distributions of concrete in Figure 5.29.

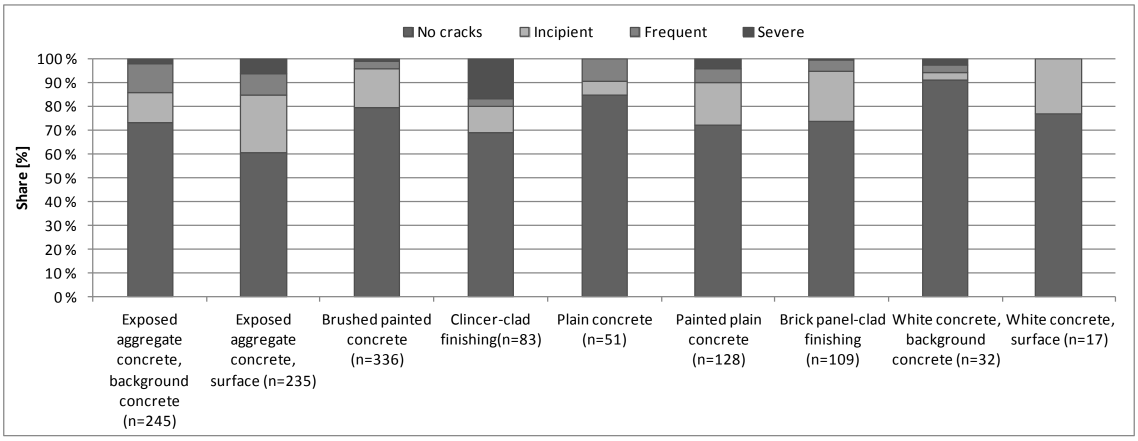


Fig. 5.28 Distribution of frost damage indicating cracking in concrete of different facade types according to thin section analyses.

The cracking indicating frost damage revealed by thin-section analysis correlates well with the visible frost damage and frost resistance of corresponding facade surface types. In the case of precast sandwich structures, exposed aggregate and white concrete, backing and surface concrete have been analysed separately. The distributions show that frost damage is more general in surface concrete than backing concrete. That is, frost damage is usually caused by the freezing of the rain or sleet received by the facade. The rain or sleet wets the outer surface of the outer layer the most, and since the temperature of the outer surface changes quicker, it freezes sooner. The condensation due to the diffusion of indoor air moisture contains only a marginal amount of water compared to the rain and sleet stress on the outer surface.

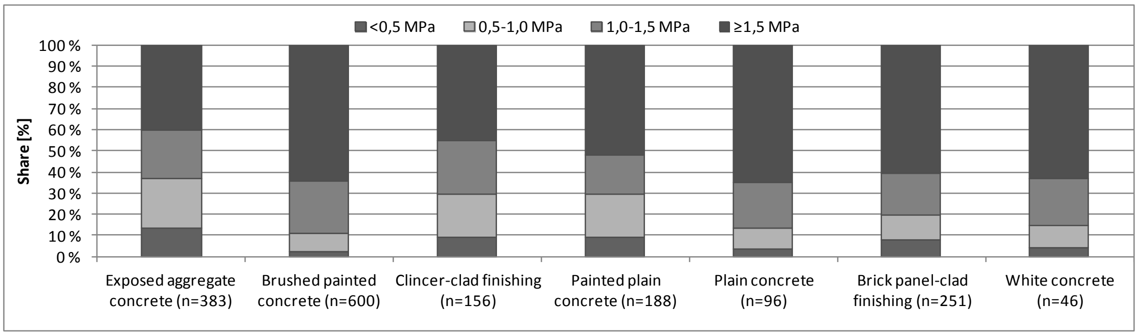


Fig. 5.29 Tensile strength distributions of concrete of different facade surface types.

Tensile strength distributions of concrete of different facade surface types also correlate well with the visible frost damage of corresponding facade surface types, the cracking indicating frost damage revealed by thin-section analysis, and frost resistance of concrete. It has not been possible to run both a thin-section analysis and a tensile strength test on the same sample since both break up the drilled core cylinder. Thus, the correlation between the results of thin-section analyses and tensile strength tests can be considered very good.

Balconies. The distributions of cracking indicating frost damage in concrete samples from balcony elements are shown in Figure 5.30 and the distributions of tensile strength test results in Figure 5.31.

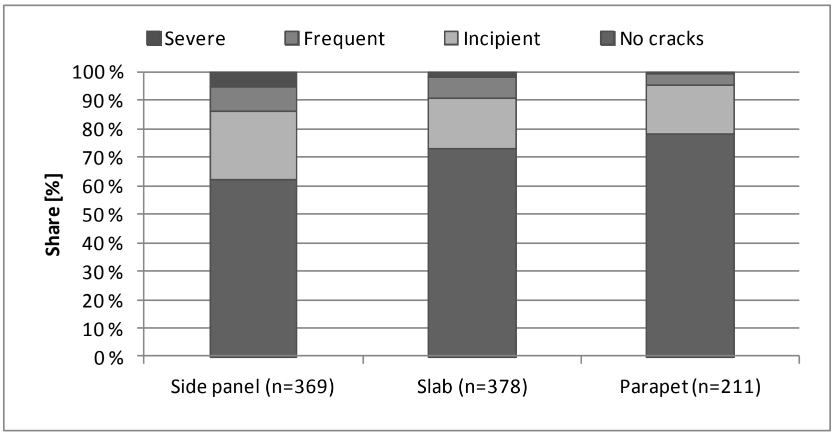


Fig. 5.30 Distribution of frost damage indicating cracking in concrete of different precast balcony panels according to thin-section analyses.

According to thin-section analyses, slightly more extensive and general frost damage occurs in balcony panels than indicated by visual investigations of buildings. Despite the inadequate frost resistance of the concrete used in balcony panels, quite little damage has occurred. On the whole, the cracking indicating frost resistance detected in thin-section analyses correlates well with the visually observed frost damage to balcony panels and their frost resistance.

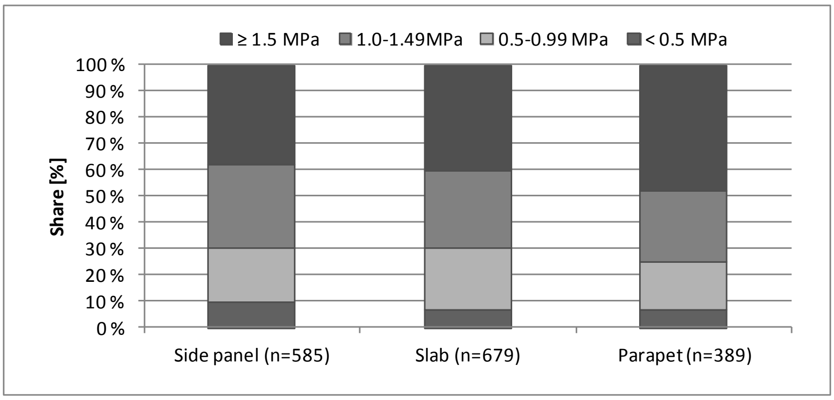


Fig. 5.31 Distribution of tensile strength of concrete in different precast balcony panels.

Extremely poor (< 0.5 MPa) and extremely good (≥ 1.5 MPa) concrete tensile test value distributions of different balcony panels correlate well with the visible frost damage, the cracking indicating frost damage revealed by thin-section analysis, and the concrete frost resistance of corresponding panels. The range between these two tensile strength values is quite wide, which is why the tensile strength test does not describe the frost damage situation accurately enough. On the other hand, it should be remembered that thin-section analyses and tensile strength tests of concrete must be made using different samples.

According to the Condition investigation manual for concrete facade panels (2002), tensile strength test values of concrete between 0.5-1.5 MPa should be interpreted as indicating “some degree” of degradation of concrete. Tensile strength tests on concrete samples taken from both facade and balcony panels indicate that the lower limit of 1.5 MPa of the tensile strength test, which determines whether concrete is considered solid and non-frost-damaged, should not be considered absolute, but it should be possible to reduce it to 1.2 MPa. That improves the correlation with results of thin-section analyses significantly.

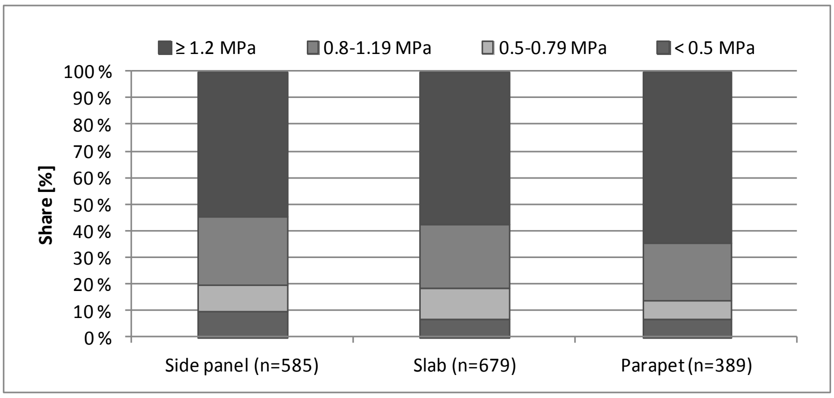


Fig. 5.32 Tensile strength distribution of concrete in different precast balcony panels where the limit of undamaged concrete has been reduced to 1.2 MPa.

Annual amount of rain and sleet during the winter season

Frost damage does not occur in dry concrete. The pore system of the concrete must be filled with water at least to the critical degree of saturation for it to happen (Fagerlund 1977). The rain and sleet falling in winter contribute to the frost damage of concrete facades and balconies since they can be absorbed into the pore system by capillary action, but drying is remarkably slow due to the structure and low temperature and high humidity of outdoor air. Slow drying allows water from autumn and winter rains and sleet to accumulate in the pore system of the concrete.

The annual amounts of rain and sleet during the winter period from September to April are presented in Figure 5.33, which shows that the amounts are as a rule higher on the southern coast (Turku) and in southern Finland (Vantaa) than inland (Jyväskylä).

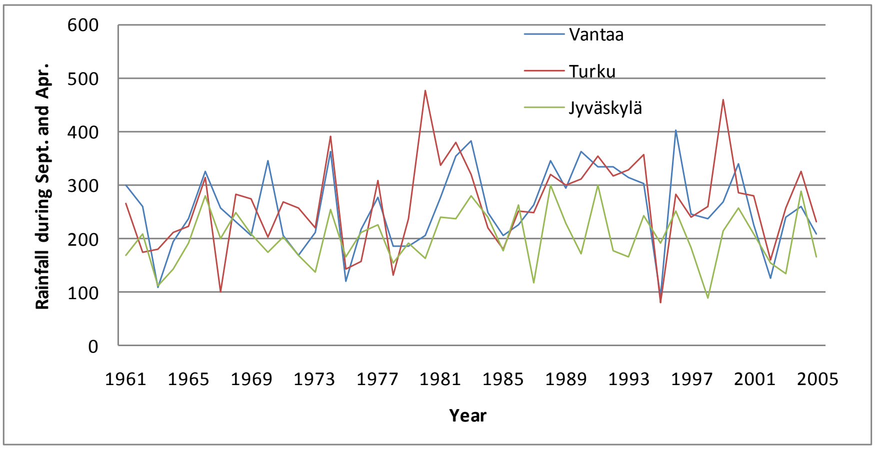


Fig. 5.33 Annual amount of rain and sleet from September to April in Vantaa (in southern coastal area), Turku (on the coast) and Jyväskylä (inland) during 1961 and 2005.

Influence of prevailing wind directions during winter season

Rain and sleet loads do not affect uniformly all facades of a building, but depend first and foremost on the wind directions prevailing during the precipitation. Wind directions prevailing during autumn and winter precipitation have been monitored from the beginning of September to the end of April in five-year periods. During winter precipitation, the winds on the southern and south-western coast (Helsinki and Turku) blow mainly from the east to southeast sector, and inland (Jyväskylä, Oulu and Rovaniemi) from the southeast to southwest sector, see Fig. 5.34 and Appendix 8. The distribution of prevailing wind directions during the rain and sleet periods is quite different at each survey location compared to prevailing wind directions in general.

Most wind speeds during rain and sleet recorded in Helsinki have been 6.1-10.1 m/s and 4.1-6.1 m/s. The wind speed distribution of Rovaniemi is highly similar to that in Helsinki, but in Rovaniemi wind speeds over 10.1 m/s during rain and sleet are rare. Most wind speeds recorded in Turku and Oulu fall in the 4.1-6.1 m/s and 2.1-4.1 m/s ranges. Jyväskylä has the lowest wind speeds during rain and sleet at 2.1-4.1 m/s and 4.1-6.1 m/s. According to FMI (2010), the average falling velocity of rain drops is 6 m/s.

The prevailing wind directions during rain and sleet are a quite clear reason for the lower incidence of frost damage on the north to east facades of buildings than on ones facing south to west. Due to stronger winds, about 60% of the rain and sleet load in the coastal area hits the facades; the corresponding share inland is about 40%. Combined with the higher amount of precipitation in coastal areas, see Fig. 5.33, facades are subjected to a considerably higher moisture stress there than inland, which clearly causes more frost damage in coastal areas. Winds are stronger at higher reaches of buildings than close to ground level, which makes it natural that the upper sections of high buildings receive more rain and sleet stress than lower buildings, and the lower sections of buildings in general.

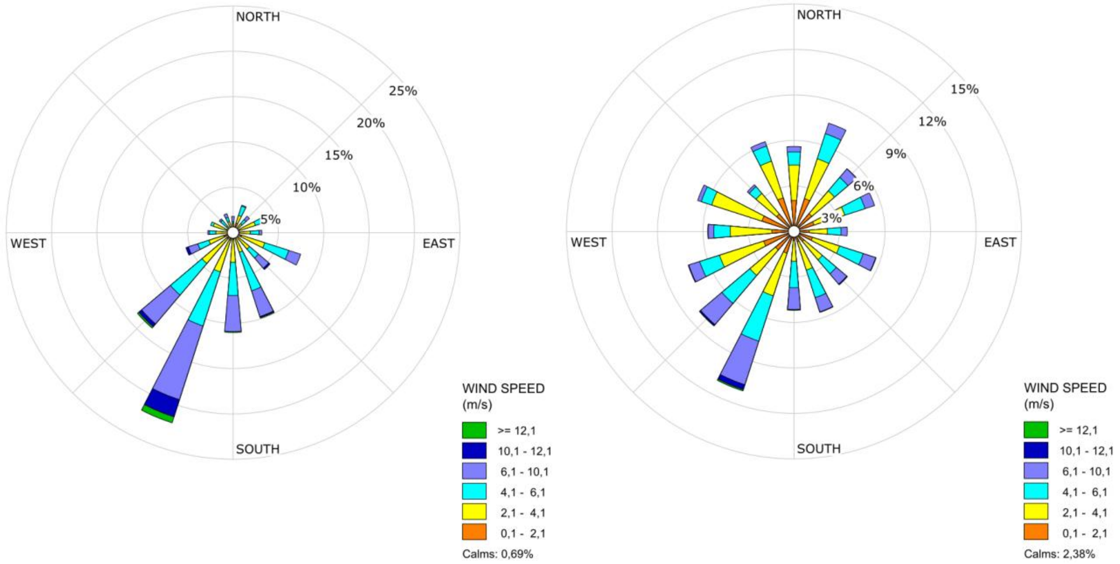


Fig. 5.34 Example of prevailing wind directions in winter during rain and sleet on the left, and year-round including also snow fall and dry weather on the right, Helsinki Airport from Sept. 1975 to Apr. 1980.

Prevailing wind directions and speeds allow examining the stress load on different facades on a general level. Naturally, the shape of the terrain and factors shielding buildings, such as closeness to a forest and nearby buildings, affect the microclimate of the building site and wind directions and force during rain and sleet, as does an open environment. Therefore, local variations e.g. in the occurrence of frost damage may deviate from the general trends presented above.

Snowfalls typically come from quite different directions than rain and sleet across Finland. The direction and amount of snowfall is inconsequential for the frost damage of concrete facades and balcony structures since snow cannot get absorbed into the pore system of concrete, but falls off facades onto the ground.

Number of annual freeze-thaw cycles

The numbers of annual freeze-thaw cycles since 1960 by survey locations are presented in Appendix 9 and a summary of them in Table 5.13. Cracking indicating frost damage detected in thin-section analyses is compared to the freeze-thaw cycles experienced by the buildings in Figure 5.35.

The observations indicate that the annual variation in freeze-thaw cycles is remarkably large in all measuring locations. A significant observation from the viewpoint of frost damage of concrete structures is that slightly more freeze-thaw cycles occur inland (Jyväskylä) than in southern Finland (Helsinki-Vantaa) and on the coast (Helsinki, Kaisaniemi). The distance between Kaisaniemi and Helsinki-Vantaa is about 20 km. A larger change in the number of freeze-thaw cycles takes place over that distance than between Helsinki-Vantaa and Jyväskylä.

Outdoor concrete structures do not suffer frost damage if they are dry enough (Fagerlund 1977), and according to Litvan's theory, all the water in the pore system of concrete does not freeze immediately after temperature falls below zero degrees centigrade (Pigeon and Pleau 1995).

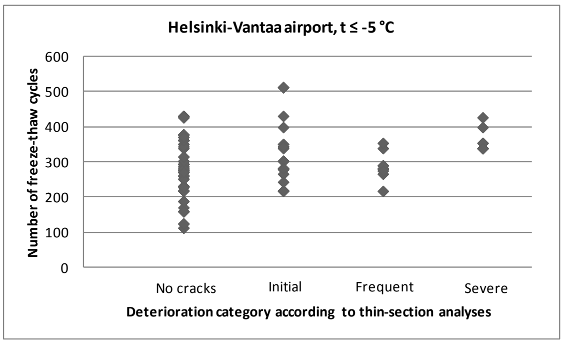
Table 5.13 Average annual number of freeze-thaw cycles at different measuring locations according to freezing temperature and pre-freezing conditions.

Measuring station	**Conditions**	**Temp. [°C]**	**No. of freeze-thaw cycles**			
			Av.	**Std. dev.**	**Min.**	**Max.**
Helsinki, Kaisaniemi	all	< 0	76.4	17.9	26	122
	rainfall ≤ 3 days before frost	≤ -2	22.3	5.1	11	34
		≤ -5	14.0	3.9	6	23
		≤ -10	5.9	2.7	1	13
Helsinki-Vantaa Airport	all	< 0	89.8	20.6	34	150
	rainfall ≤ 3 days before frost	≤ -2	22.0	5.7	8	33
		≤ -5	13.6	4.3	4	26
		≤ -10	6.1	2.4	2	12
Jyväskylä Airport	all	< 0	91.5	19.8	45	132
	rainfall ≤ 3 days before frost	≤ -2	23.5	5.0	15	33
		≤ -5	16.4	4.5	8	25
		≤ -10	8.2	3.5	2	15

The numbers of freeze-thaw cycles at different measuring locations have also been observed in conditions of rain and sleet for a maximum of three days before freezing. It has also been expected that temperature drops to -2 °C, -5 °C or -10 °C. The conditions have been chosen so that the actual frost attack effect under the prevailing conditions is clearly stronger than when examining the total number of annual freeze-thaw cycles. Yet, the chosen conditions are not based on a detailed analysis of the conditions under which the entire pore system of the concrete would fill with water.

Surveys based on roughly defined conditions such as these show that the number of annual freeze-thaw cycles decreases dramatically compared to all freeze-thaw cycles. Differences between various measuring locations also even out while the order remains when using this type of surveys.

The cracking indicating frost damage revealed by thin-section analyses of concrete samples has been compared to the number of freeze-thaw cycles undergone by the same samples. The survey targeted the outer surface of the exposed aggregate panel in an effort to eliminate the influence of a paint coat and other finishing products on the absorption of rainwater into the pore system of the concrete. Due to the small number of the buildings right on the coast, it was possible to compare the southern coastal area and inland.



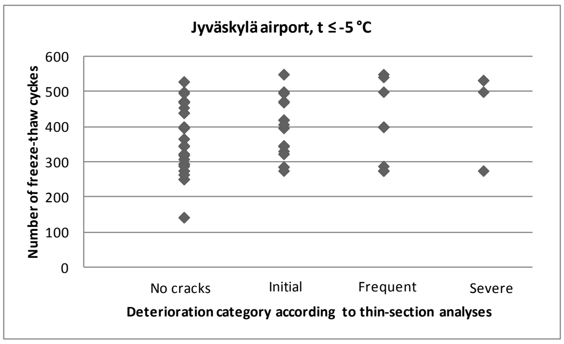


Fig. 5.35 Number of freeze-thaw cycles in different deterioration categories of exposed aggregate concrete facades according to thin-section analyses.

Concrete samples shown to contain much or general cracking indicating frost damage by thin-section analysis had typically been subjected to a slightly larger number of freeze-thaw cycles than samples showing only incipient cracking indicating frost damage. Samples that showed no, or only incipient, cracking indicating frost damage in thin-section analysis dominated since incipient frost damage cannot be observed visually on the outer surface of a structure. It has not been necessary to run as many thin-section analyses of structures where far advanced frost damage already is visible as in the case of visibly undamaged structures when selecting a repair method.

The starting point of the condition investigation of a structure is the determination of its damage situation and what repair measures are needed. Condition investigations have targeted buildings of a quite wide age-range, which means that the time of the initiation of damage due to frost damage is not known accurately. All that is known is whether damage existed at the time of the condition investigation. Consequently, it is only possible to estimate the range of the numbers of different freeze-thaw cycles in relation to level of damage.

Cracking indicating frost damage is the result of an average of 388 freeze-thaw cycles ($t \leq -5$ °C) inland and 307 cycles in the southern coastal area. If the freezing temperature criterion is $t \leq -10$ °C, incipient frost damage occurs on average after 189 and 140 freeze-thaw cycles, respectively. On the southern coastal area this translates into about 22 years and inland into about 24 years. General frost damage revealed by thin sections begins to occur in exposed aggregate facades in the southern coastal area on average after 330 freeze-thaw cycles ($t \leq -5$ °C) and after 416 cycles inland.

Due to the condition investigations made on buildings of different ages for various reasons, and especially the variation in the material properties and the true stress level of concrete, deviation is large, which makes it more interesting to focus on the smallest free-thaw cycle numbers at which frost damage is revealed by thin-section analyses. Incipient frost damage has been detected in the southern coastal area after 210 and general damage after 207 freeze-thaw cycles ($t \leq -5$ °C). The respective numbers of freeze-thaw cycles inland are 270 and 277. These numbers of freeze-thaw cycles, that led to various levels of damage based on thin-section analysis, reveal that the same level of damage normally requires more freeze-thaw cycles inland than on the southern coastal area.

The amount of annual rain and sleet on the coastal area is clearly larger than inland, see Fig. 5.33. Prevailing wind speeds during rain and sleet on the coastal area are also clearly higher in autumn/winter in the coastal area than inland. The amount of rain and sleet received by facades and wind direction are thus more crucial for the occurrence of frost damage than the mere number of freeze-thaw cycles since the actual rain/sleet stress received by facades on the coastal area is clearly higher than inland, where, again, the number of freeze-thaw cycles, also those leading to frost damage, is larger than on the coastal area.

5.3.3 Secondary void filling

The formation of ettringite or portlandite has not led to degradation of concrete based on any condition investigations on concrete facades and balconies. Thin-section analyses have shown that crystallisation of ettringite and portlandite occurs on the surface of concrete pores in all facade and balcony panel types, see Figs. 5.36 and 5.37.

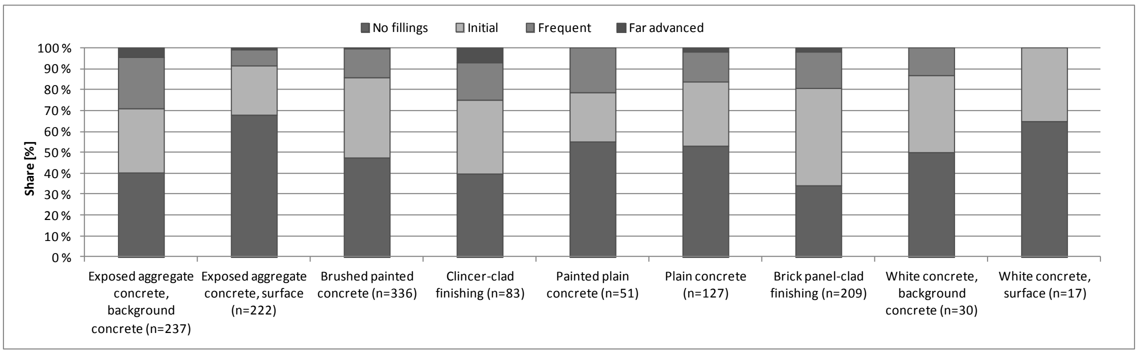


Fig. 5.36 Degree of pore filling of different facade types.

Incipient filling of the pore system is substantial in all facade panel types at 25% or more. Frequent circular and a high degree of filling occurs in different facade types to the same extent that frost damage has been observed visually and frost cracking has been revealed by thin sections. In the case of precast sandwich panels, exposed aggregate and white concrete, the examination of the degree of pore filling has been done separately for backing concrete and surface concrete. The distributions show that the degree of filling of the pore system is on the whole more prevalent in backing concrete than surface concrete in contrast to the occurrence of frost damage in said panel types.

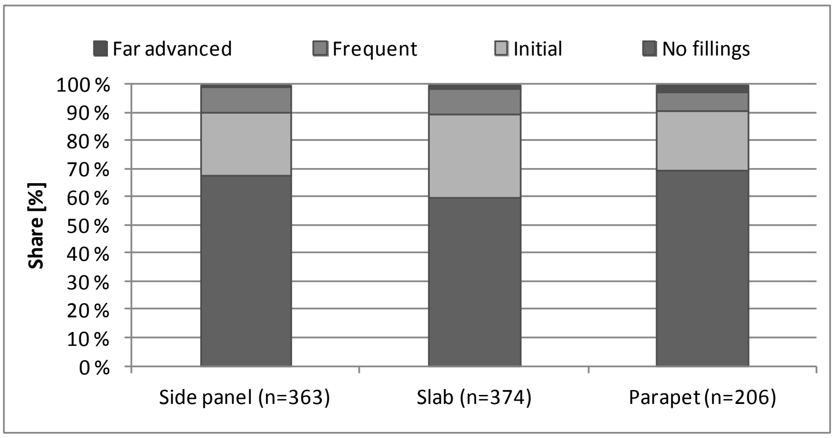


Fig. 5.37 Degree of pore filling of different balcony panel types.

Based on thin-section analyses, the degree of filling of pores and cracking indicating frost damage of side panel correlate well. On the other hand, a clearly larger portion of slabs and parapet panels show some degree of filling than cracking indicating frost damage. The shares of far advanced degree of filling are yet very small in all balcony panel types.

Although the formation of ettringite and portlandite in the pore system of concrete has not as such caused degradation of concrete, the substance that accumulates in the pore system does reduce pore volume. The pore system is filled by a smaller amount of water due to the crystallisation products in the pores making it easier for frost damage to take place.

5.3.4 Alkali-aggregate reaction

Degradation due to alkali-silica reaction (ASR) has been detected in ten bridges in Finland over the last 10 years, when samples have been drilled from beneath the surface layers of a structure. Also there the damage from ASR has been quite small compared to frost damage of concrete and chloride corrosion of steels in concrete (Pyy and Holt 2010).

The facade and balcony condition investigation reports compiled for this study contain thin-section analyses of a total of 2 435 samples. None of the reports tells of observations related to alkali-aggregate reaction (AAR). The aggregate of the concretes has consisted primarily of natural gravel, crushed aggregate has only been used in isolated cases, and then the buildings have been completed in 1986 or later.

According to Pyy and Holt (2010), in the case of samples drilled close to the outer surface of a structure, from a depth of 50-60 mm, the crack-filled gel typical of ASR caused by wetting and drying of concrete, may have flowed out onto the concrete, and the empty cracks may have been thought to have been caused by frost damage. Although the outer layers of facade panels are typically quite thin, the claim that the gel has flowed out with moisture is not convincing. The washing out of the gel would require quite strong rain stress that does not occur, especially on a northern facade. The side panels and slabs of balconies are clearly thicker than the outer layer of the facade, yet the thin-sections (a total of 973) removed from balcony structures have not revealed signs of AAR. Neither do the condition investigation reports contain any verbal or pictorial indications of the gel flowing onto the facade surface.

The development of AAR is considerably slow in Finnish outdoor conditions (Pyy and Holt 2010). All in all, it may be stated that AAR has not resulted in the need to repair precast concrete buildings, at least until now.

5.4 Other deterioration and malfunctioning of facades and balconies

5.4.1 Safety risks

Both far advanced corrosion of reinforcements and frost damage of concrete may have a detrimental impact on the bearing capacity and fastening reliability of a concrete structure. Although a considerable share of the reinforcement steels of quite a few buildings had been corroded already at the time of the condition investigation, the reports made no mention of inadequate bearing capacity of reinforcements due to corrosion. The corrosion risk of the fastenings of sandwich panels made of conventional steel was recognised in the reports, but the problem was not considered acute and repair measures were recommended over a 5-10 year period.

Neither was far advanced extensive frost damage of concrete seen as causing immediate loss of bearing capacity or fastening reliability of panels. It was recommended that repair measures be implemented, on average, over the same 5-10 year period, but on condition that the development of deterioration is monitored at 1-3 year intervals depending on the case.

The condition investigation reports reveal that quite considerable deterioration of structures has been allowed without considering that it would radically shorten the technical service life of the examined structure. The condition investigation reports regarded pieces of concrete coming off a facade or balcony panel as a result of corrosion of reinforcements or frost damage as the most significant safety risks.

5.4.2 Degradation of coatings

Facades and balconies have been painted with both impermeable and permeable paints. The type of paint applied to structures has not generally been determined in connection with condition investigations; at least it has not been recorded in the reports. Thus, the impact of a coat of paint on the wetting of concrete cannot be studied definitively based on this material, only suggestively.

Clearly more extensive and far advanced frost damage has been detected on painted facades and side and parapet panels of balconies when the paint coating has been damaged widely, see Fig. 5.38. This allows concluding that the used paints have mainly been types that reduce the absorption of rainwater. Visual inspection shows that an unbroken coat of paint has protected the concrete structure from frost damage.

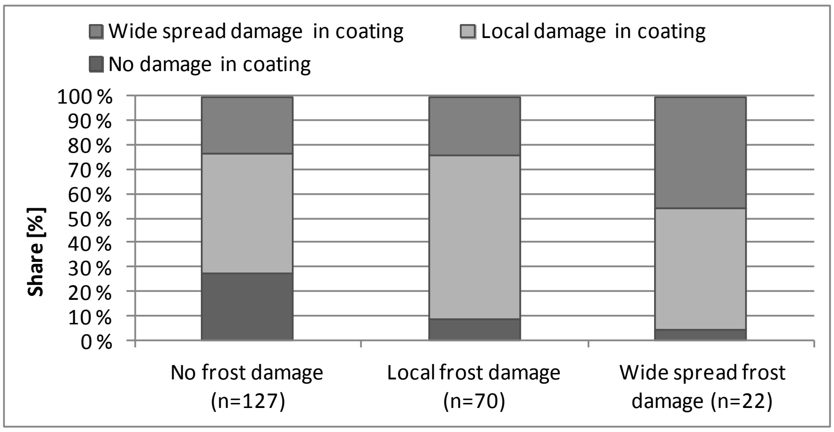


Fig. 5.38 Distribution of degrees of degradation of coatings due to frost damage in balcony side panels.

As earlier stated, the majority of balcony structures do not suffer from any frost damage. Thus, an unbroken coat of paint is not the most crucial factor for the occurrence of frost damage. For instance, the share of an unbroken coat of paint is clearly the largest (28%) in balcony side panels where no frost damage has been detected. Yet, they suffer from extensive (24%) and local flaking (49%) of the painted surface in the investigated buildings. Considering the extremely large share of poorly frost-resistant concrete in balcony side panels, extensive and far advanced frost damage should thus occur much more widely in side panels. This leads to the conclusion that the actual rain and sleet stress received by a structure and the functioning of details are more crucial for the occurrence of frost damage in concrete of low frost resistance than used facade paints.

5.4.3 Other deterioration mechanisms

The elastic joints between panels were quite often damaged at the time of investigation or detached from the edges of panels. Completely undamaged joints were found only at 21-40% of the buildings depending on facade surface type. Rainwater can penetrate deeper inside a panel through the damaged joints between panels resulting in higher local moisture stress at the edges of panels. The condition of elastic joints was not found to be clearly linked to visible corrosion or frost damage of panels.

Besides the condition of joints, individual observations about defects in the moisture behaviour of the facade and balconies were recorded in condition investigation reports. The most common defects, according to the reports, have been the quite small slopes and projections of exterior window sills; sometimes the slopes have also been in the wrong direction. Eaves flashings have also been found generally deficient without specifying the defects.

Other degradation mechanisms affecting concrete facades, such as debonding of brick panels and clinker tiles, or deformations of structures and cracking due to other causes than corrosion of reinforcements or frost damage, have been recorded quite rarely in condition investigation reports. Observations about the condition of the mortar joints of balconies are also occasional. It is not possible to conduct a universal survey of individual degradation mechanisms and their effects.

6 CONCLUSIONS

The general objective of this research effort was to study the factors that have actually had an impact on the service life, occurrence and progress of different degradation mechanisms in existing concrete facades and balconies.

The research was based on condition investigation data from existing concrete buildings and measured weather data. The research material consisted of a database of material properties and observations about the deterioration of existing Finnish concrete facade panels and balconies covering the period between 1960 and 1996, and weather observations since 1961 by the Finnish Meteorological Institute (FMI).

6.1 On structural properties

Facades. The average thickness of the outer layer of sandwich panels is 55-70 mm depending on the facade surface type, which is close to the design values of both bearing and non-bearing panels. In theory, it is only possible to install the designed reinforcement with its splices and lifting lugs in an outer layer 65 mm thick to meet the 20 mm cover depth requirement. A 25 mm cover depth requires, considering the installation tolerances, an 85 mm outer layer. From the viewpoint of the durability of a concrete structure, it has thus been impossible even theoretically, to provide steels the 25 mm cover depth set in requirements.

However, local under 40 mm and over 80 mm outer layer thicknesses occur in all facade surface types. In the case of under 40 mm outer layers, it is impossible to place the reinforcement at a proper depth from the viewpoint of corrosion protection. Inevitably the reinforcement is either too close to the outer surface in contradiction of the concrete cover requirement, or too close to the thermal insulation, which may endanger bonding. Also, outer layer reinforcements do not perform optimally in excessively thick outer layers. When reinforcements lie too deep, they cannot prevent cracking due to shrinking of concrete. Variation in the thickness of the outer layer can be considered a work defect since none of the investigated buildings had only remarkably thin or thick outer layers. Workers walking on top of thermal insulations and careless spreading of concrete in the form are the most likely causes of the non-uniform thicknesses of outer layers.

The outer layers of sandwich panels are most often supported by stainless steel trusses, and their fastening reliability has generally been good according to condition investigation reports. Defects in fastening reliability due to far advanced frost damage have been detected only in isolated cases. Thus, strong curvature of thin-shell panels is not necessarily a sign of extensive and far advanced frost damage, especially in clinker- and brick panel-clad buildings, where curvature is typically caused by shrinkage differences between cladding and backing concrete.

In eight individual buildings completed in the 1960’s and 1970’s, the truss material had been mild steel, and in five buildings the outer layer had been attached by some other means. By way of generalization, one may say that there has been wider variation in the type of attachment of the outer layer in buildings erected in the 1960’s than in the so-called BES period buildings.

Balconies. The dwelling-specific balconies of precast multi-storey residential buildings, that were among the subjects of this research, are stacked balconies supported on their own foundations and consisting of side panels, floor slabs and panels. Only a small number of cantilever balconies are included in the material. The thicknesses of balcony structures have been decided on the basis of carrying capacity design. The thicknesses of side panels and slabs have typically not prevented achieving the required protective concrete layers on reinforcements. The average thickness of parapet panels is 87 mm, which allows the required concrete cover on both surfaces. On the other hand, it is not possible to provide a sufficient concrete cover on the reinforcements of the 80 mm thick parapets, 37 of which are included in the research material.

Defects in fastening reliability have not occurred in balconies supported on their own foundations except in a few isolated instances. They have involved extensive far advanced frost damage. Damage to support structures has been more common with cantilever balconies. There it has mostly resulted from the corrosion of unprotected tension reinforcement at slit insulations in walls. Balconies supported by railway rails or other structural steels have generally had sufficient capacity also for more advanced corrosion.

The drainage of balconies has been implemented with spout pipes penetrating the parapet, through a gap between the parapet and the slab, or by a drain pipe at the corner of the slab. An uncontrollable balcony drainage system results in higher local moisture stress on balcony panel of lower stories. Corrosion and frost damage of the reinforcements of all balcony panel types are clearly more common when drainage is implemented through a gap between the slab and the parapet. It conveys most water onto the side panels underneath, and especially onto the soffit of the slab.

6.2 On durability properties

Compaction and capillary porosity of concrete. There is wide deviation in the degree of compaction of the concrete samples from the same building's facade surface types and balcony panels. On the whole, the compaction of concrete has been moderately successful; compaction had failed totally only in a few isolated instances.

The capillary porosity of the concrete used for facades and balconies is quite high, which indicates a fairly high water-cement ratio of concrete. Capillarity affects greatly the wetting of concrete in outdoor conditions: the more capillary the concrete, the quicker and the more water enters its pore system during rainfall. In porous concrete carbonation propagates faster than in dense concrete. On the other hand, no correlation between the degree of compaction of concrete and its capillary porosity was found.

Cover depths of reinforcement. Reinforcement cover depths can be measured most reliably from concrete samples since the measurements of field investigations generally do not eliminate the effect of the surface structure of the panel or the finishing products. If the effect of surface roughness or, for instance, the thickness of a clinker tile is deducted from the reading of the cover depth meter, the reinforcement cover depth distributions measured from samples and those determined by the cover depth meter are quite consistent.

Reinforcement cover depths do not meet in all respects the minimum requirements set for them in design and durability guidelines of various periods. Yet, it can be stated that on the whole the shares of small, under 5 mm and under 10 mm, cover depths are quite small in all facade surface types – less than 3.6%. In clinker-clad panels the share of under 10 mm cover depths is 6.2%, which is significant from the viewpoint of the economy of cementitious patch repairs.

On the other hand, in the case of balcony elements, the shares of small reinforcement cover depths are remarkably large especially on inner and outer surfaces of structures with a view to cementitious patch repairs. The situation is bad as far as durability properties are concerned, especially in the case of parapet panels, where the occurrence of under 10 mm cover depths in inner surfaces is 7.7%.

Presence of chlorides. The critical chloride content of concrete necessary for the initiation of reinforcement corrosion is exceeded in only a few individual cases – in buildings mostly completed before 1976. Use of chlorides as a hydration accelerator has been considered primarily a problem of on-site prefabrication plants of the 1960's. Therefore, it was surprising to discover high chloride contents also in nine precast panels of buildings completed in the 1970's, and especially in four buildings completed in the 1980's, where high chloride contents from the viewpoint of corrosion initiation were found in balcony side panels and slabs. It was impossible for chlorides to have penetrated into the concrete of any of the buildings after their completion. They must have entered it in connection with the manufacture of the precast panels either intentionally or accidentally.

Although 3.8% of the samples drilled from balconies (1.4% of facade samples) exceed the critical chloride content, corrosion of reinforcements initiated by chlorides is yet extremely rare. High chloride contents are not regionally concentrated, which means that – despite its rareness – it is justified to determine the chloride content of concrete in connection with a condition investigation since the effects of chloride corrosion may be severe with respect to the safe use of structures.

Frost resistance of concrete. Both thin-section analyses and protective pore tests have shown that the frost resistance of the concrete used for facades and balconies is quite inadequate in general. Exposed aggregate, clinker-clad and unpainted form-finish facades and balcony side panels have the lowest frost resistance.

The low frost resistance of balcony side panels can be considered a significant service-life limiting factor for the entire precast balcony stock since the side panels are bearing structures, which cannot be replaced without tearing apart the entire balcony structure. The frost resistance of slabs is also poor to a large extent. However, the top of the slab has often been coated with quite dense paint, which as long as it remains unbroken, effectively prevents carbonation of the top surface and penetration of rainwater into the concrete. Consequently, the low frost resistance of balcony slabs is not as big a risk as that of side panels for the bearing capacity of a balcony or the ending of its service life.

6.3 On guidelines for durability of concrete

The protective pore ratios of the research material indicate that the use of air-entrainment in the manufacture of balcony panels began in 1976 and increased gradually. Only after 1981 did it start becoming an established feature of balcony panel production, which improved considerably the frost resistance of the building stock completed since. Air-entrainment of concrete facades began somewhat earlier than with balcony elements around the mid-1970's. Making different facade types frost resistant has most likely been a challenging undertaking since the protective pore ratios of concrete started improving significantly only after 1984.

The durability guidelines published by the concrete sector and the national design standards and requirements played a crucial role in the improvement of the frost resistance of concrete in the 1980's. Especially the systematic use of air-entrainment agents and the wider introduction of various testing methods have essentially increased the frost resistance of the concrete used in facade and balcony elements. Despite the development of concrete technology, the frost resistance of the precast panels produced in 1988 and thereafter still suffer from major deficiencies: 24% of balconies and 17% of facades have poor frost resistance. On the other hand, the mere raising of the strength of concrete improves its frost resistance, which means that unsuccessful air-entrainment does not necessarily lead to a short service life.

Corresponding development in realised reinforcement cover depths has not occurred although the cover depth requirement of the guidelines has increased considerably since the 1980's.

6.4 On actual deterioration

Despite the generally quite poor durability properties of concrete facades and balconies, relatively little far advanced and extensive corrosion and frost damage occurs in concrete structures.

Corrosion of reinforcement. Much visible corrosion damage has been observed in both facades and balconies in connection with condition investigations. Visible corrosion damage is typically local, extensive corrosion damage was detected in 5.7% of the facades and 15% of the balconies of the target buildings. Corrosion of reinforcements is the most common reason for the repair need of structures in the case of all balcony elements and facade surface types, except for exposed aggregate.

Corrosion damage has been caused almost solely by the carbonation of concrete. As concrete carbonises, corrosion first initiates in reinforcements closest to the outer surface. The most typical local corrosion damage in facade panels has occurred at the ends of reinforcement mesh wires at splices near the outer surface and the edge bars at the sides of window openings. Corrosion damage in balcony elements occurs most typically at the edge bars of side panels and the steels of balcony parapets.

The concrete carbonation coefficient is clearly linked to porosity of concrete. As a result of lower carbon dioxide diffusion resistance and a wider continuous capillary pore system, carbonation of concrete progresses fastest in concretes with the largest share of capillary pores, and slowest in those where the share of capillary pores is smallest.

On the other hand, different orientations have not been found to affect facade carbonation rate. The varying rain and sleet stress received by different facades would thus not appear to have a significant impact on carbon dioxide diffusion resistance – other properties of concrete, primarily porosity and amount of carbonating material, are more determining factors.

The carbonation coefficients of the inner surface of sandwich panels are quite small with all panel types compared to the carbonation of the facade surface. Hence, it may be concluded that hardly any air circulates across the inner surface of panels despite the ventilation pipes in the elastic joints of panels and ventilation channels. The stringers of the trusses of outer layers are thus also well protected against corrosion.

There is great variation in the carbonation rate of concrete between various samples from the same building and between different facade types. With the exception of white concrete and clinker-clad facades, the average carbonation of concrete has typically reached a depth of 5 mm in less than ten years and a depth of 10 mm in about 20-30 years. The present average carbonation depth of a precast multi-storey residential building completed in 1970 is 11-20 mm depending of the facade surface type. In white concrete and clinker-clad surfaces carbonation has proceeded on average only to a depth of about 7 mm. Thus, it has been possible for the steels of reinforcements closest to the surface in buildings completed before 1980 to corrode for 20 years or longer.

A minor difference was detected in the average carbonation rates between the outer and inner surfaces of balcony side and parapet panels: the carbonation coefficients of the more weather-protected inner surfaces were slightly larger than those of the outer surfaces. The soffits of balcony slabs are completely protected from rain and sleet meaning that their carbonation coefficients are the largest. On the other hand, corrosion of reinforcements progresses very slowly in structures protected from rain and sleet. The average carbonation rate of the concrete of all balcony elements in balconies dating from the 1980's is only slightly lower than that of the balconies of the 1960's and 1970's. The carbonation rate of balconies built in the 1990's is clearly lower than that of those built in earlier decades.

Frost damage in concrete. In light of the quite inadequate air-entraining of concrete, the visually observable frost damage to both facades and balconies is remarkably light. Visually observable frost damage was detected in 43% of all facade types and only 27% of balconies of all targets. Frost damage has been local for the most part. Very little frost damage has occurred especially in balcony side panels considering the inadequate frost resistance properties.

Local frost damage to facade panels typically occurs in the upper corners of a building and at the edges of panels. In southern coastal areas facades generally suffer slightly more frost damage than inland, and the damage is clearly more common on facades facing south-east to west than on other facades. Typical local frost damage to balconies occurs in the upper sections, especially at the front edges of side panels. Visually observable frost damage clearly correlates with frost resistance. In the case of facade surface types or balcony panels with the smallest shares of substandard frost resistance, less visible frost damage naturally also occurs.

There are considerable differences in the occurrence of frost damage between various facade surface types. The most local and extensive frost damage occurs in exposed aggregate facades, of which 46% had suffered local and 16% extensive damage. There are no layers retarding the absorption of rainwater on an exposed aggregate surface, such as a coat of paint or brick panels or clinker tiles. Therefore, rainwater can easily be absorbed into the pore system of the concrete. Clinker-clad and painted form-finish facades also suffer more than average local and extensive frost damage. In balconies, frost damage occurs the most in side panels, whose frost resistance is the lowest, and the least in parapet panels, which have been provided the best frost resistance during manufacture. The occurrence of visually observable damage is thus logical. The surfaces of balcony elements are typically painted, which means that the unbroken and dense coating may have limited the wetting of concrete and thus also the occurrence of frost damage.

The thickness of the thermal insulation of sandwich panels, and thus the heat flow through the facade, has no major impact on the occurrence of frost damage in any facade surface type. The thermal transmittance of even the oldest sandwich panels is at least 0.63 W/m^2K meaning that the heat flow through the external wall is relatively small. The durability properties of a facade panel, such as the frost resistance of its concrete, and prevailing stress conditions, are much more determining factors in the realisation of the frost damage of concrete.

Other degradation mechanisms. The initial degree of pore filling was quite high with all facade and balcony panel types at 25% or more. Frequent circular and a high degree of filling occurs in various facade to the same extent as visible frost damage and frost cracking revealed by thin-section analysis. Formation of ettringite and portlandite in the pore system has not as such caused weathering of concrete, but the water content of the pore system reduces the pore volume of concrete. The pore system is filled by a smaller amount of water due to the crystallisation products therein, which allows frost damage to take place more easily.

None of the results of the thin-section analyses compiled for this research from condition investigation reports on facades and balconies indicate observations related to alkali-aggregate reaction (AAR). The aggregate of concrete has consisted primarily of natural gravel, crushed aggregate has been used only in isolated instances, and then the buildings have been completed in 1986 or later. According to literature, the development of AAR is remarkably slow in Finnish outdoor conditions. All in all, it can be stated that, at least so far, AAR has not caused any need to repair precast concrete buildings.

6.5 On Finnish outdoor climate

Corrosion of steel embedded in carbonated concrete requires a relatively high moisture content of concrete and a pore system of low frost resistance – filled almost completely by capillary action before the frost attack – in order that frost damage may start. The rain and sleet stress received by facades and balconies has thus a crucial impact on the development of damage and the damage rate.

Annual rainfall. Both annual and winter-time precipitation measurements show that more rain and sleet is received in the coastal area than inland. Thus, the higher rainfall could be a reasonable explanation for more corrosion damage and frost damage in the coastal area than inland. It has also been discovered that rainfall is the highest in autumn when the relative humidity of outdoor air is typically also high. These factors together keep concrete structures wet for long periods. As a result, the corrosion rate of steels remains high for long, and water collects deep inside the pore system of concrete before the beginning of the frost period.

The rain and sleet falling during the winter contribute to frost attack of concrete since they can be absorbed by capillary action into the pore system of concrete, but drying is remarkably slow due to the low temperature and high humidity of outdoor air. The slow drying process allows water to accumulate in the pore system during autumn and winter precipitation.

Prevailing wind directions. Rain and sleet do not fall evenly on all facades of a building, but are largely driven about by prevailing winds. During the autumn and winter rain and sleet, the prevailing winds on the southern and south-western coast blow from between east and south-west, and inland from between south-east and south-west. Compared to year-round wind directions, the distribution of wind directions during the survey period in question is quite different. Wind direction and speed during snowfall have no impact on the damage to concrete structures since snow cannot be absorbed into the pore system of concrete. During summer rains, the winds blow most often from between east and south-west, yet the southerly winds dominate.

Thus, prevailing winds during rain and sleet are a quite clear reason for the fact that less corrosion damage and frost damage of concrete occurs on facades facing north to east than on those facing south to west.

Due to the stronger winds prevailing in the coastal area during rain/sleet, a larger portion of it hits the facades than inland. With the contribution of the higher precipitation of the coastal area, the moisture stress received by facades is considerably higher in the coastal area than inland, which also has a clear causal relationship with the more prevalent corrosion of reinforcements and frost damage of concrete in the coastal area. Since winds are stronger at the level of the higher sections of buildings, it is natural that the upper parts of high buildings receive more precipitation stress than lower buildings, and the lower parts or buildings in general.

Freeze-thaw cycles. A significant observation with respect to frost damage of concrete structures is that slightly more freeze-thaw cycles occur inland (Jyväskylä) than in southern Finland (Helsinki-Vantaa) and on the immediate coast (Helsinki, Kaisaniemi). The number of freeze-thaw cycles varies more over the about 20 km distance between Kaisaniemi and Helsinki-Vantaa than between Helsinki-Vantaa and Jyväskylä.

The number of annual freeze-thaw cycles falls dramatically compared to all natural freeze-thaw cycles when the requirement is that temperature drops to -2 °C, -5 °C or -10 °C and rain or sleet is received for a maximum of three days before freezing. This type of assessment also evens out the differences between various measuring locations while maintaining their order.

Incipient frost damage is detected inland by thin-section analysis after an average of 388 freeze-thaw cycles (rain/sleet and $t \leq -5$ °C) while the corresponding figure in the southern coastal area is 307. Time-wise this means about 22 years in the southern coastal area and about 24 years inland. General frost damage begins to appear in the exposed aggregate facades of the southern coastal area on average after 330 freeze-thaw cycles (rain/sleet and $t \leq -5$ °C) and inland after 416 cycles.

Due to the condition investigations conducted on buildings of various ages for different reasons, and especially the variation in the material properties and actual stress level of concrete, the deviation is large, which means that these average numbers of freeze-thaw cycles having led to damage should be considered suggestive. Yet, it can be stated that a smaller number of freeze-thaw cycles leads to the same degree of frost damage in the coastal area than inland.

Annual precipitation in the form of rain and sleet is clearly larger in the coastal area than inland. Moreover, the speeds of the winds prevailing in the autumn and winter seasons during that precipitation are clearly higher there than inland. The amount of precipitation received by facades and wind speeds have thus a more crucial role in the occurrence of frost damage of concrete than the mere number of freeze-thaw cycles since the actual precipitation stress on the facades in the southern coastal region is clearly higher than inland, where, again, the number of freeze-thaw cycles, also those leading to frost damage, is larger than in the coastal area.

6.6 Utilisation of the results

The database containing the durability properties of the different precast panel types of the existing precast concrete building stock and the actual damage suffered by it constitutes an excellent starting point for the development of service-life models for concrete structures and assessment of the impact of climate change on the technical repair need of the existing precast building stock.

Service-life models for existing concrete buildings. The presently used life-cycle models for concrete structures are based on theoretical degradation models of concrete structures and laboratory analyses. They have been developed for to estimating the future service life of new concrete structures. They are often unsuitable for estimating the service life of existing building stock since reliable initial values and assumptions for the models are not available. Moreover, the existing building stock also typically suffers from some types of damage that service-life models do not take into account.

The starting point of current service-life models is that a concrete structure is manufactured in accordance with valid design and manufacturing specifications. Only installation tolerances are considered. As shown by this research effort, the existing precast concrete building stock has a lot more deficiencies with respect to reinforcement cover depths and frost resistance of concrete than even earlier guidelines and installation tolerances permitted.

This research has produced an abundance of new information about phenomena that damage structures under actual natural conditions covering a long period. Based on that information and actual measured meteorological observations, it is possible to develop more accurate service-life models based on actual measurements for existing precast concrete buildings erected in the 1960’s to 1980’s.

The results of this research also include parameters that can improve the accurary of existing service-life models. Reliable parameters are needed especially in the investigation of propagation an variation of damage and factors influencing them, such as climate, structure and material properties.

Influence of climate change on existing precast concrete building stock. No broader studies or surveys of the adaptation of the existing precast concrete building stock to future climate have been made so far. Yet, it is a matter of great economic significance since the service-life goal of buildings designed today is 50-100 years, and the service life of older building stock is also being extended by renovation.

Several models on future climate change have been built and evaluated comprehensively e.g. in the ACCLIM Climate Change Survey of the Finnish Meteorological Institute (Jylhä et al. 2009). In addition, the Institute has developed its own model for forecasting future weather. Such climate models are used to make forecasts about changes in windiness, rainfall and numbers of freeze-thaw cycles. The RH and temperature of air are other significant variables in the assessment of drying conditions.

These meteorological data will allow evaluating the amount of rain and sleet falling on structures and changes in drying capabilities compared to present climatic conditions. In the case of porous materials, the number of freeze-thaw cycles undergone by a filled pore system is critical for frost damage while RH of air and the humidity of a structure increase the corrosion rate of steels and allow mould growth.

Estimation of technical repair need. This research provided a clear and reliable picture of the durability properties of the existing precast multi-storey building stock by panel and surface types. That information allows estimating the technical repair need of the Finnish precast multi-storey building stock.

Reliable estimation of the building stock’s repair need requires thorough understanding of the stock by panel and surface types. The data available from Statistics Finland is, however, quite raw, which means that further research in the area is needed.

6.7 Need of further research

Porosity and water-cement ratio of concrete. The durability properties of concrete have been dealt with quite comprehensively and from many angles in this research. According to literature, the water-cement ratio of concrete and the amount of carbonating material, in practice cement, are quite crucial factors.

The carbonation rate of concrete varies considerably even between samples drilled from the same building independent of the construction year as previously shown. The realised water-cement ratio of concrete can be assessed by thin-section analysis based on so-called comparison samples. The thin-section analyses compiled for this research did not, however, explore the actual water-cement ratio of concrete. To do that, comparison samples should be prepared based on given water-cement ratios to allow comparison with existing thin sections. That would provide more detailed understanding of the impact of the capillary porosity of concrete on its carbonation rate.

Correlation between freeze-thaw cycles and actual amount of rain and sleet on facades. This research has clearly shown that the prevailing wind directions during rain and sleet and their amount are crucial factors contributing to the frost damage of concrete. Further research is needed on the actual amount of rain and sleet received by facades and being absorbed into the pore system of concrete as well as the connection between actual amounts of rain and sleet and freeze-thaw cycles.

Extension of database. This research deals quite comprehensively with the durability properties and degradation of precast concrete multi-storey buildings erected between the end of the 1960’s and the end of the 1980’s on the basis of compiled condition investigation reports. The durability of concrete structures in outdoor conditions has been considered in the design standards for concrete structures since 1989. The database accumulated for this research contains quite few buildings completed in the 1990’s and should therefore be complemented with condition investigation information on more recent precast multi-storey buildings.

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APPENDICES

Appendix 1 List of buildings subjected to condition investigation, 4 pages.

Appendix 2 Visual evaluation of degree of compaction from surface of concrete, 1 page.

Appendix 3 Distribution of cover depths of reinforcement according to field measurements in different facade surface types and balcony elements, 7 pages.

Appendix 4 Distribution of carbonation coefficient of concrete in different facade surface types and balcony elements, 9 pages.

Appendix 5 Carbonation coefficient relative to the capillary porosity of concrete in different facade surface types and balcony elements, 5 pages.

Appendix 6 Annual precipitations without snowfall during 1961 and 2005, 1 page.

Appendix 7 Wind directions and wind speeds during annual rain and sleet amount during 1981 and 1985, 3 pages.

Appendix 8 Wind directions and wind speed in winter during rain and sleet and at all times including snow fall and dry weather during Sept.1975 and Apr. 1980, 3 pages.

Appendix 9 Annual freeze-thaw cycles during Sept. 1961 and Apr. 2006, 3 pages.

Appendix 1 List of buildings subjected to condition assessment

Name of building	Location
As. Oy Ahvenispolku	Tampere
As. Oy Haarakatu 1	Tampere
As. Oy Hangon Pitkäkatu 31	Hanko
As. Oy Harjupää	Hollola
As. Oy Haukanhaka	Tampere
As. Oy Heinätori 7	Tampere
As. Oy Helkavuori	Valkeakoski
As. Oy Hiiralankaari 23	Espoo
As. Oy Hollolan Salpakankaantie 8-14	Hollola
As. Oy Honkametso	Tampere
As. Oy Honkateeri	Tampere
As. Oy Hovioikeudenkatu 4	Turku
As. Oy Hämeenlinnan Viipurinpuisto	Hämeenlinna
As. Oy Ispoinen ©	Turku
As. Oy Jaatsinkatu 3	Vammala
As. Oy Kallioväylä	Oulu
As. Oy Karkkilan Miilutie 6	Karkkila
As. Oy Kastanjametsä	Turku
As. Oy Kaviokatu 2-4	Lahti
As. Oy Keihäsrinne	Vantaa
As. Oy Kerttulinkatu 10	Turku
As. Oy Kimnaasipolku	Helsinki
As. Oy Koivukerttu	Tampere
As. Oy Kokkokalliontie 1	Helsinki
As. Oy Korson Näätäkuja	Vantaa
As. Oy Kuohunkorkea	Kangasala
As. Oy Kämnerinkuja 1	Helsinki
As. Oy Käpylänkara	Kouvola
As. Oy Kärkikaksikko	Tampere
As. Oy Loukonlehmus	Pirkkala
As. Oy Länsitorni	Oulu
As. Oy Maijanpolku	Espoo
As. Oy Malminkallio	Tampere
As. Oy Marinraitti	Tampere
As. Oy Metsämarja	Oulu
As. Oy Naapurilähiö	Tampere
As. Oy Näsipuisto	Tampere
As. Oy Opinkallio	Tampere
As. Oy Palokuja 4	Helsinki
As. Oy Pirkanmaija	Pirkkala
As. Oy Porvarintie 8-10	Helsinki
As. Oy Porvarintie 8-10	Helsinki
As. Oy Puistokatu 12	Turku
As. Oy Puistotalo	Vammala
As. Oy Puistovärttinä	Tampere
As. Oy Päivinraitti	Tampere
As. Oy Rajakettu	Oulu
As. Oy Riimukatu 4-6	Lahti
As. Oy Riimukatu 4-6	Lahti
As. Oy Ruotulan tornit (2)	Tampere
As. Oy Ruotulan tornit (8)	Tampere
As. Oy Salon Miilunpohja	Salo
As. Oy Salpakankaantie 8-14	Hollola
As. Oy Sarasipi	Tampere
As. Oy Satopäivi	Tampere
As. Oy Satopääsky	Tampere
As. Oy Satovarpunen	Tampere

Name of building	Location
As. Oy Siltakoivu	Helsinki
As. Oy Sirkanhovi	Turku
As. Oy Taavintorppa	Espoo
As. Oy Tarjanteenkallio	Tampere
As. Oy Timonpuisto	Lahti
As. Oy Valjaskatu 2	Lahti
As. Oy Valjaskatu 2	Lahti
As. Oy Valtakuja	Tampere
As. Oy Valtaraitti	Tampere
As. Oy Vaskitie 1	Oulu
As. Oy Vuolukiventie 2 ja 4	Vantaa
ASO Asunnot Oy	Espoo
ASO Asunnot Oy	Vantaa
ASO Asunnot Palokorvenkatu 7	Kerava
ASO Asunnot Vasamakatu 10	Lahti
Espoonkruunu Oy Ankkuripoiju	Espoo
Espoonkruunu Oy Avaruuskatu 1	Espoo
Espoonkruunu Oy Iivisniemenkatu 1 (Itähovi)	Espoo
Espoonkruunu Oy Iivisniemenkatu 1 (Itäkontu)	Espoo
Espoonkruunu Oy Jänkäkoira	Espoo
Espoonkruunu Oy Kuitinkatu 2	Espoo
Espoonkruunu Oy Kyyhkysmäki 13	Espoo
Espoonkruunu Oy Kyyhkysmäki 5	Espoo
Espoonkruunu Oy Nupukivenrinne	Espoo
Espoonkruunu Oy Postipuuntie 12	Espoo
Espoonkruunu Oy Sepetlahdentie 11	Espoo
Espoonkruunu Oy Soukankuja 10-12	Espoo
Espoonlahti	Espoo
Esppoonkruunu Oy Maapallonkatu 3	Espoo
Hakkuri	Oulu
Hannuniitun koulu	Turku
Herralahden koulu	Pori
Herttuan koulu, Antinkatu 15 A	Pori
HOAS Juhana Herttuan tie 3	Helsinki
HOAS Kylänevantie 16	Helsinki
Hätilän liikekeskus	Hämeenlinna
Jupiterinkatu 4	Riihimäki
Kalervo	Oulu
Kantakaupungin kiinteistöt Oy Palkkatilankatu 1-3	Helsinki
Kantakaupungin kiinteistöt Oy Palkkatilankatu 1-3	Helsinki
Kauppakeskus Kompassi	Hollola
Kaupungin vuora-asun. Marsinkatu 6	Riihimäki
Kokkolan terveyskeskus	Kokkola
Konsulttitalo Oy	Espoo
Koy Ankkuripoiju	Espoo
Koy Asemakatu 38-40	Kuopio
Koy Asiakkaankatu 3	Helsinki
Koy Auringonrata	Espoo
Koy Eestinmaja	Espoo
Koy Hakku	Espoo
Koy Hansahovi	Espoo
Koy Hervannan Puistokallio	Tampere
Koy Hervannan Turva	Tampere
Koy Huhtimonlinna	Riihimäki
Koy Jakomäenkiinteistöt, Talo 11b	Helsinki
Koy Jakomäenkiinteistöt, Talo 15b	Helsinki
Koy Jakomäenkiinteistöt, Talo 20a	Helsinki

Name of building	Location
Koy Jakomäenkiinteistöt, Talo 6a	Helsinki
Koy Jakomäenkiinteistöt, Talo 6b	Helsinki
Koy Jakomäenkiinteistöt, Talo 6e	Helsinki
Koy Jakomäenkiinteistöt, Talo 6g	Helsinki
Koy Jakomäenkiinteistöt, Talo 6i	Helsinki
Koy Jakomäenkiinteistöt, Talo 8a	Helsinki
Koy Jakomäenkiinteistöt, Talo 9b	Helsinki
Koy Juvanmänty Annalankatu 5	Tampere
Koy Juvanmänty Annalankatu 8	Tampere
Koy Kalamiehenkallio	Espoo
Koy Kalamiehenlaakso	Espoo
Koy Kannintie	Vammala
Koy Kantakylän-salpa	Helsinki
Koy Kapunkoti	Espoo
Koy Kartanonisäntä II	Järvenpää
Koy Kartanonmaja	Järvenpää
Koy Kartanonpirtti	Järvenpää
Koy Kartanontupa	Järvenpää
Koy Katrinkeskus	Espoo
Koy Katumantie 28	Hämeenlinna
Koy Katuportti	Espoo
Koy Kaulushaikara	Espoo
Koy Kehänvarsi	Espoo
Koy Kerinhonka	Lahti
Koy Kipparinsolmu	Espoo
Koy Kirkonkello	Espoo
Koy Kirstintie	Espoo
Koy Koskelomppi	Oulu
Koy Kukinkuja 8	Vantaa
Koy Kukkaromäki	Espoo
Koy Kylänaukio	Espoo
Koy Kylänraitti	Espoo
Koy Leipurinkoti	Espoo
Koy Leipurinkoti	Espoo
Koy Leppäkeihäs	Espoo
Koy Leppätupa	Espoo
Koy Lintukorpi	Espoo
Koy Lounaismaininki	Espoo
Koy Lounaistuuli	Espoo
Koy Lukkarinkoti	Espoo
Koy Maininkihovi	Espoo
Koy Malminhaka	Tampere
Koy Merenkulkija	Espoo
Koy Merenkulkija	Espoo
Koy Merimatti	Espoo
Koy Merisilta	Espoo
Koy Merkuriuksen-Salpa	Riihimäki
Koy Mäkkylä K9	Espoo
Koy Nekalantie 81	Tampere
Koy Nihtiristi	Espoo
Koy Peijaksentie 31	Vantaa
Koy Puolarkoti	Espoo
Koy Pyyntitie 4	Espoo
Koy Rastaanpesä	Espoo
Koy Rautakiskonkuja 1	Espoo
Koy Rinkeliniitty	Espoo
Koy Riukhänt	Rauma

Name of building	Location
Koy Salamapolku	Helsinki
Koy Seitsenlinna	Espoo
Koy Siperianportti	Espoo
Koy Sirkkavuori	Lempäälä
Koy Sorakuja	Vantaa
Koy Soukanrinne 4	Espoo
Koy Soukantupa	Espoo
Koy Soukka 37	Espoo
Koy Taavintupa	Espoo
Koy Tiistiläntörmä 3	Espoo
Koy Timpurinkoti	Espoo
Koy Tähtitaivas	Espoo
Koy Tähystäjänkoto	Espoo
Koy Vaakamestarinpolku 2	Helsinki
Koy Vallikontu	Espoo
Koy Vanulanpolku 2	Salo
Koy Vanulanpolku 4	Salo
Koy Vanulanpolku 6	Salo
Koy Vasama	Espoo
Koy Venetie 16	Joensuu
Koy Veturitie 1	Vantaa
Koy Viherrinne	Espoo
Koy Viliniementie	Espoo
Koy Välitalontie 71	Helsinki
KVT Lintulampi (A)	Tampere
KVT Lintulampi (B)	Tampere
KVT Lintulampi (D)	Tampere
LEL Mikropolku 4	Tampere
Malmin kiinteistöt Oy Pukinkuja 2	Helsinki
Merkuriuksenkatu 9	Riihimäki
Opiskelijankatu 21-23 (1-3)	Tampere
Opiskelijankatu 21-23 (4)	Tampere
Peltolammin koulu	Tampere
Peltolammin koulu	Tampere
Pispan palvelukeskus	Tampere
Porin lentoasema, matkustaja-asemarakennus	Pori
Sammon kiinteistö Hämeenkatu 20	Tampere
Sato Junailijankuja 10	Helsinki
Sato Kauppakartanonkatu 17	Helsinki
Sato Lupajantie 2	Helsinki
Sato Maistraatinkatu 7	Helsinki
Sato Matruusinkatu 2	Helsinki
Sato Opastinsilta 1	Helsinki
Sato Palkkatilankatu 9	Helsinki
Sato Saarenvainionkatu 9	Tampere
Simpsintie 7	Oulu
Suomalais-Venäläinen koulu, Kaarelankuja 2	Helsinki
Tampereen Kaupungin vuokratalot, Kalkun Viertotie 11	Tampere
Tampereen Uintikeskus	Tampere
Tampereen Uintikeskus	Tampere
Tampereen Vuokratalosäätiö, Orivedenkatu 13	Tampere
Tampereen Yliopisto, Hoitotieteen laitos	Tampere
Tampereen Yliopisto, Lääketieteen laitos	Tampere
Toas Wäinölä III	Tampere
VAV Apajakuja 7	Vantaa
VAV Esikkotie 7	Vantaa
VAV Helmikuja 1	Vantaa

Name of building	Location
VAV Huddingenpolku 2	Vantaa
VAV Kirvisenkuja 1	Vantaa
VAV Leikkitie 1	Vantaa
VAV Leiritie 6	Vantaa
VAV Linnustajankuja 1	Vantaa
VAV Marsinkuja 4	Vantaa
VAV Pähkinärinteentie 22	Vantaa
VAV Pähkinärinteentie 24	Vantaa
VAV Pähkinärinteentie 26	Vantaa
VAV Raikukuja 14	Vantaa
VAV Ruukuntekijäntie 10	Vantaa
VAV Talvikkirinne 4	Vantaa
VAV Terhotie 6	Vantaa
VAV Valtuustokatu 1	Vantaa
VAV Varpusenkuja 1	Vantaa
VVO 124 Oppegårdinpolku 1	Tuusula
VVO 141 Jampankaari 7	Järvenpää
VVO 150 Koskustie 4	Helsinki
VVO 161	Järvenpää
VVO 161	Järvenpää
VVO 168 Ketokivenkaari 6	Helsinki
VVO 175 Hartaantie 12	Oulu
VVO 181 Jampankaari 8	Järvenpää
VVO 185 Hartaantie 14	Oulu
VVO 201 Kuntotie 8	Klaukkala
VVO 24 Kaarnapolku 2	Järvenpää
VVO 308 Kuparisepänkatu 3	Kerava
VVO 5014 Humalatarhantie 1-5	Porvoo
VVO 636 Hakaniemenranta 30	Helsinki
VVO Aittakarinkatu 16-18	Rauma
VVO Aittapellonkatu 11	Lahti
VVO Aittapellonkatu 5	Lahti
VVO Ampuhaukantie 4	Oulu
VVO Brennerinpolku 4	Vaasa
VVO Eerikinkallio 4	Kirkkonummi
VVO Eerikinkallio 6	Kirkkonummi
VVO Hakakatu 3	Raisio
VVO Hallituskatu 19	Oulu
VVO Haltianpolku 10	Järvenpää
VVO Havukoskenkatu 14	Vantaa
VVO Honkaharjuntie 14	Jyväskylä
VVO Hopearinne 3	Kirkkonummi
VVO Ieskatu 11	Turku
VVO Ihantolantie 5	Nurmijärvi
VVO Joukahaisentie 7	Porvoo
VVO Joukahaisentie 9	Porvoo
VVO Joupinmäenrinne 6	Espoo
VVO Joupinmäensyrjä 4	Espoo
VVO Joupinrinne 4	Espoo
VVO Jukolantie 7	Kouvola
VVO Juurakkotie 47	Rovaniemi
VVO Jälkimaininki 4	Espoo
VVO Kaakonpyrstö 1	Jyväskylä
VVO Kaakonpyrstö 5	Jyväskylä
VVO Kaartinkatu 6	Lappeenranta
VVO Kaartinkatu 8	Lappeenranta
VVO Kannistonkaarre 4	Kerava
VVO Kannistonkaarre 7	Kerava
VVO Kappalaisenkatu 5	Mikkeli
VVO Karhitie 3	Hämeenlinna

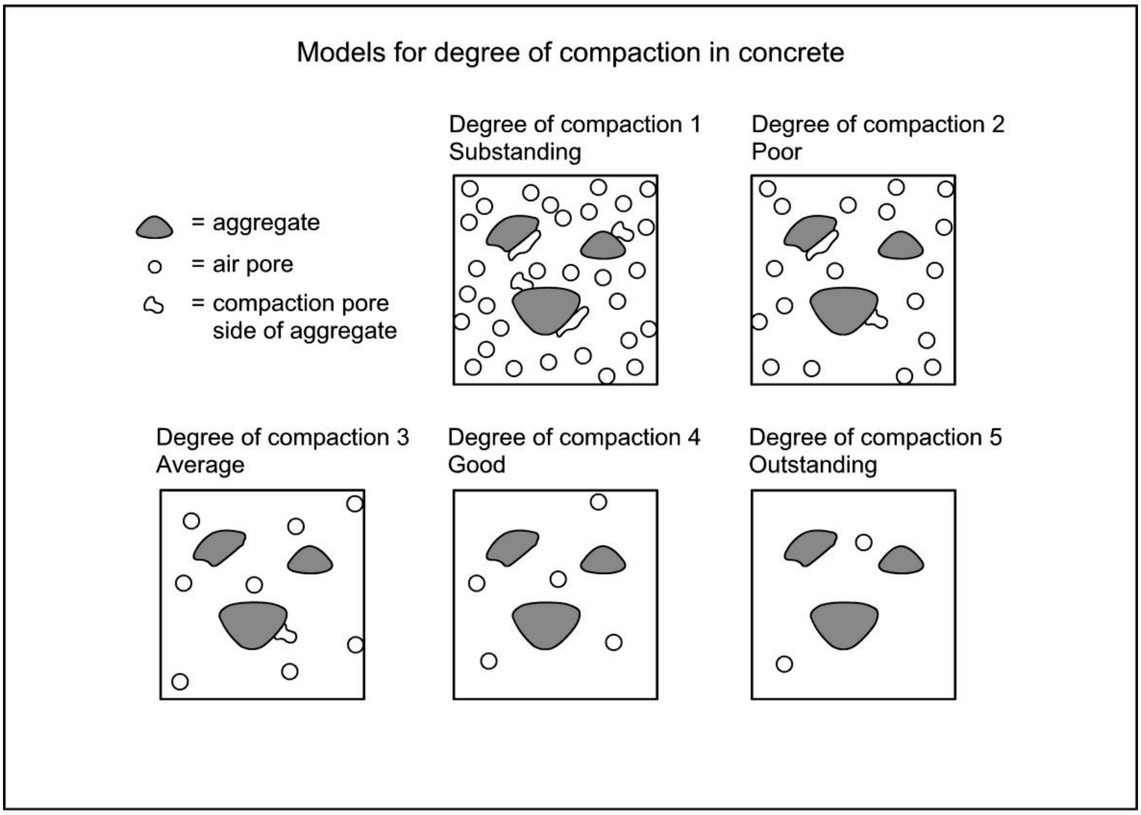
Name of building	Location
VVO Karhuntie 18	Hämeenlinna
VVO Karpalokuja 2	Jyväskylä
VVO Karpalokuja 3	Jyväskylä
VVO Karpalopolku 8	Rauma
VVO Katariina Saksilaisenkatu 11	Helsinki
VVO Katumantie 23	Hämeenlinna
VVO Kauppakartanonkuja 3	Helsinki
VVO Kauppakatu 8	Iisalmi
VVO Kavallintie 1	Kauniainen
VVO Keijunpolku 6 (1)	Kotka
VVO Keijunpolku 6 (2)	Kotka
VVO Keijunpolku 6 (3)	Kotka
VVO Kemiankatu 13	Tampere
VVO Kirstinharju 23	Espoo
VVO Kirstintie 17	Espoo
VVO Kirstintie 17	Espoo
VVO Kiulukuja 6	Vantaa
VVO Kivenlahdenkatu 5	Espoo
VVO Koivukatu 5	Riihimäki
VVO Kolikkotie 2	Jyväskylä
VVO Kolupolku 6	Helsinki
VVO Konsulinkatu 4	Heinola
VVO Konttikatu 3	Kaarina
VVO Koukkutie 33	Kerava
VVO Koulutie 6	Hollola
VVO KOY Suntionkolmoset	Turku
VVO Kukkumäentie 20	Jyväskylä
VVO Kukkumäentie 22	Jyväskylä
VVO Kultasepänkatu 3	Kerava
VVO Kuntalantie 7	Kuusankoski
VVO Kurjenkellonkuja 2	Helsinki
VVO Käsityöläisentie 4-10	Helsinki
VVO Kääpäkatu 8	Kerava
VVO Laiturikuja 1	Jyväskylän mlk.
VVO Lauklähteenkatu 7	Turku
VVO Lemminkäisenkatu 17	Turku
VVO Lentokonetehtaantie 5	Tampere
VVO Lepolantie 4	Kerava
VVO Liisankuja 2	Espoo
VVO Lindalintie 4	Kirkkonummi
VVO Lumikero 3	Vantaa
VVO Luoteisväylä 18-20	Helsinki
VVO Lupajantie 4-6	Helsinki
VVO Maakotkantie 8	Vantaa
VVO Maitotie 5	Kuopio
VVO Maitotie 5	Kuopio
VVO Mannerheimintie 168	Helsinki
VVO Marttilankatu 3 ja 5	Kerava
VVO Mesenaatinkuja 4	Helsinki
VVO Metsäkaari 16	Hyvinkää
VVO Nekalankulma 5A-F	Tampere
VVO Neulaskuja 5	Kerava
VVO Opiskelijankatu 20	Tampere
VVO Opiskelijankatu 28	Tampere
VVO Opiskelijankatu 36	Tampere
VVO Opiskelijankatu 8	Tampere
VVO Orelinkatu 49-51	Nokia
VVO Paalupolku 3	Vantaa
VVO Paavo Kolin katu 9	Tampere
VVO Pasuunakuja 1	Helsinki

Name of building	Location
VVO Paturintie 15-21	Pori
VVO Pehtoorinkatu 3	Heinola
VVO Peipontie 6-8	Porvoo
VVO Peltovainionkatu 4	Tampere
VVO Peräpelto 6	Jyväskylä
VVO Porvoonkatu 3	Kerava
VVO Punkkerikatu 8	Lappeenranta
VVO Pupuhuhdantie 12	Jyväskylä
VVO Pupuhuhdantie 6	Jyväskylä
VVO Rantakylänkatu 17	Joensuu
VVO Rappulanharju	Salo
VVO Ratsutilantie 1	Pirkkala
VVO Rientolankatu 9	Tampere
VVO Ristikedonkatu 33	Salo
VVO Risupadontie 6	Helsinki
VVO Rouskunkatu 11	Imatra
VVO Ruiskukkatie 3	Oulu
VVO Ruoritie 5	Kuopio
VVO Ruununmaankatu 9	Kotka
VVO Rypysuontie 49 (1, 2)	Kuopio
VVO Rypysuontie 49 (3)	Kuopio
VVO Rypysuontie 51	Kuopio
VVO Rypysuontie 66	Kuopio
VVO Rälssintie 7	Helsinki
VVO Saarijärventie 22	Kuopio
VVO Saarijärventie 22	Kuopio
VVO Saarnimäenkuja 4	Espoo
VVO Satamakatu 7	Iisalmi
VVO Sauvatie 4-6	Vantaa
VVO Seljapolku 9	Vantaa
VVO Seppäläntie 13	Nurmijärvi
VVO Sillanpäänpolku 8	Salo
VVO Siltakatu 7 (ABC)	Jyväskylän mlk
VVO Siltakatu 7 (DEF)	Jyväskylän mlk
VVO Simpsintie 2,4,6 ja 6A	Oulu
VVO Soukanahde 4	Espoo

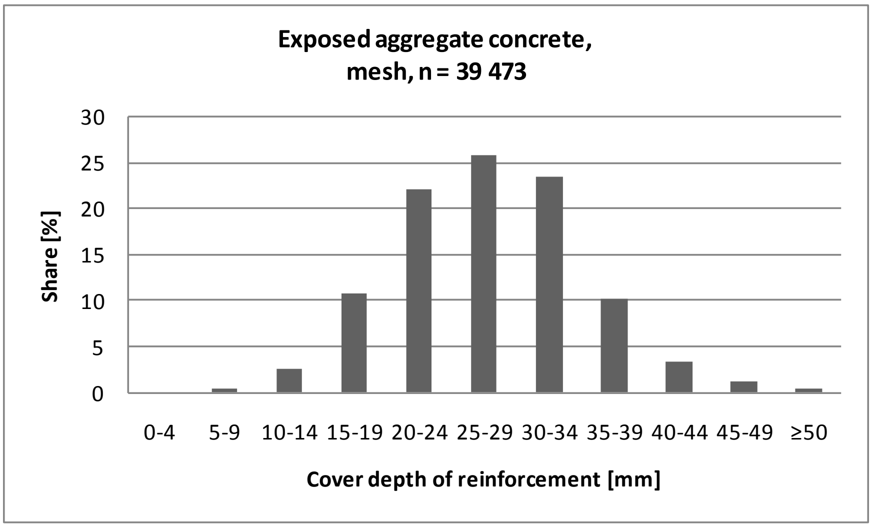
Name of building	Location
VVO Stoltinkatu 6	Turku
VVO Sähkökatu 3	Tampere
VVO Taiteentekijäntie 7	Helsinki
VVO Taivaskero 1	Vantaa
VVO Taivaskero 6	Vantaa
VVO Takilatie 4	Kuopio
VVO Tanhuantie 1	Helsinki
VVO Tapparanpirtti	Lahti
VVO Tasaajankatu 1	Kotka
VVO Tellervontie 3	Oulu
VVO Tellervontie 5	Oulu
VVO Tiistilänkuja 3	Espoo
VVO Tikkamäentie 2	Joensuu
VVO Tikkamäentie 4	Joensuu
VVO Tinatie 1	Helsinki
VVO Tolsanpolku 6	Kirkkonummi
VVO Tornimäki	Vantaa
VVO Tornipolku 15 & 17	Porvoo
VVO Tornipolku 2	Porvoo
VVO Tornipolku 4	Porvoo
VVO Touvitie 15	Kuopio
VVO Trumpettikuja 2	Helsinki
VVO Tykkikuja 3	Vantaa
VVO Tyllilänkatu 3	Hämeenlinna
VVO Tyllilänkatu 5	Hämeenlinna
VVO Vaaksatie 1	Pori
VVO Vaasankatu 17	Helsinki
VVO Vanha Yrttimaantie 22	Helsinki
VVO Varistonkuja 2	Vantaa
VVO Veteraanitie 7	Mäntsälä
VVO Vihantatie 3	Vantaa
VVO Viilarinkatu 2-4	Vaasa
VVO Vilppulantie 29	Helsinki
VVO Yliopistonkatu 21	Turku
VVO/KOY Dahlpark	Kirkkonummi
VVO/KOY Lindalinlaakso	Kirkkonummi

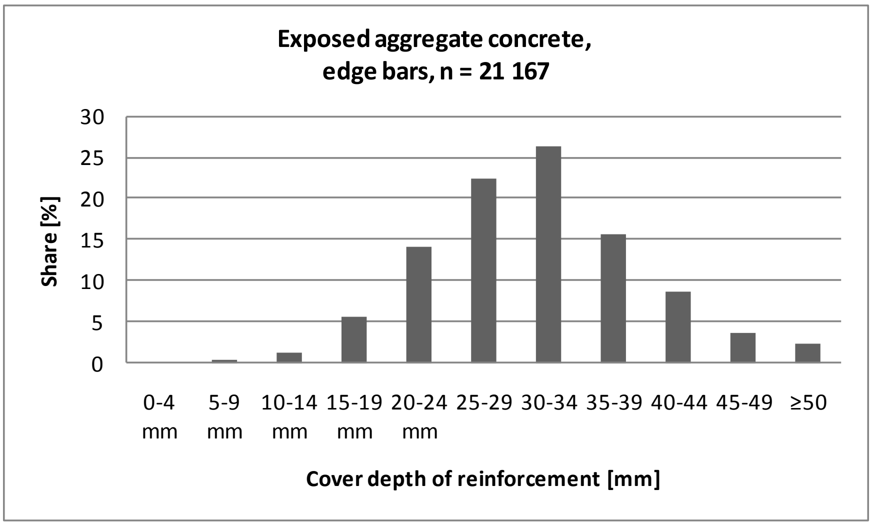
Appendix 2 Visual evaluation of degree of compaction from surface of concrete

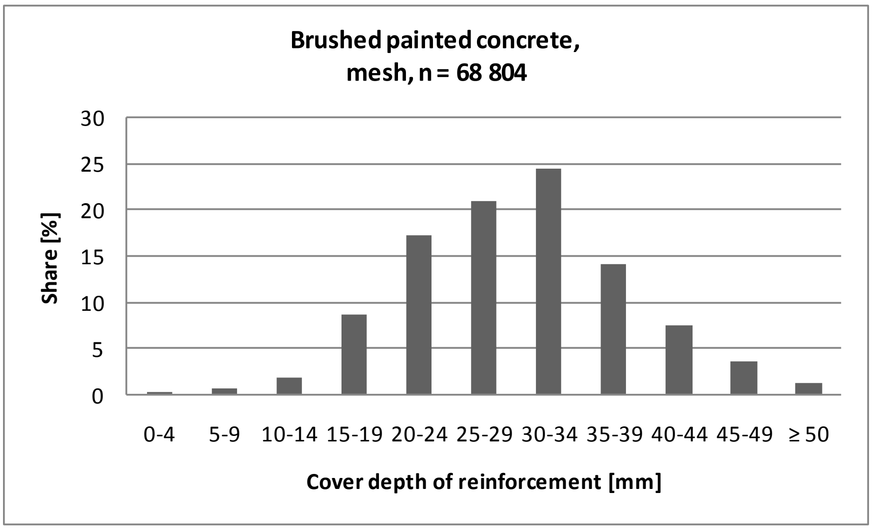
Criteria for determining degree of compaction of concrete		
Degree of comp.	**Observations**	**Share**
1	air pores pores by aggregate and reinforcement	> 15% of area > 5% of area
2	air pores pores by aggregate and reinforcement	8-15% of area 1-5% of area
3	air pores pores by aggregate and reinforcement	5-8% of area 0-1% of area
4	air pores no pores by aggregate and reinforcement	2-5% of area -
5	air pores no pores by aggregate and reinforcement	< 2% of area -
Visual observations of surface of sample without instruments		

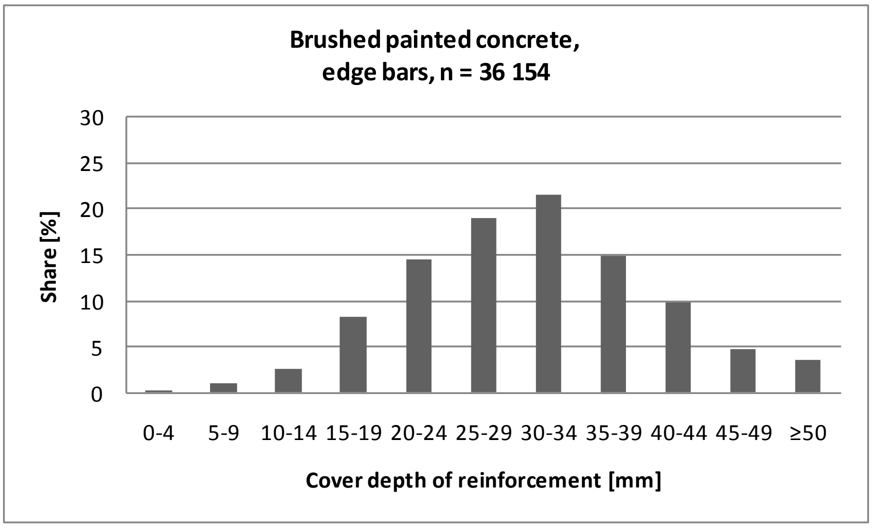


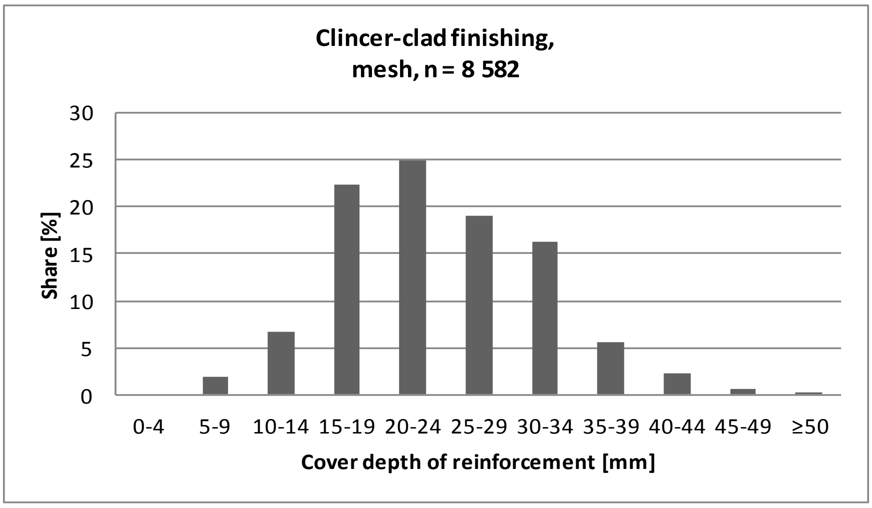
Appendix 3 Distribution of cover depths of reinforcement according to field measurements in different facade surface types and balcony elements

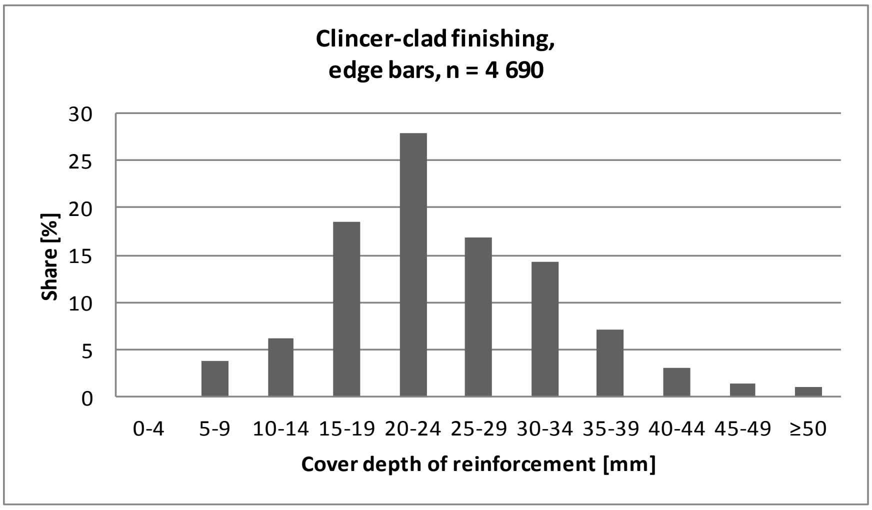


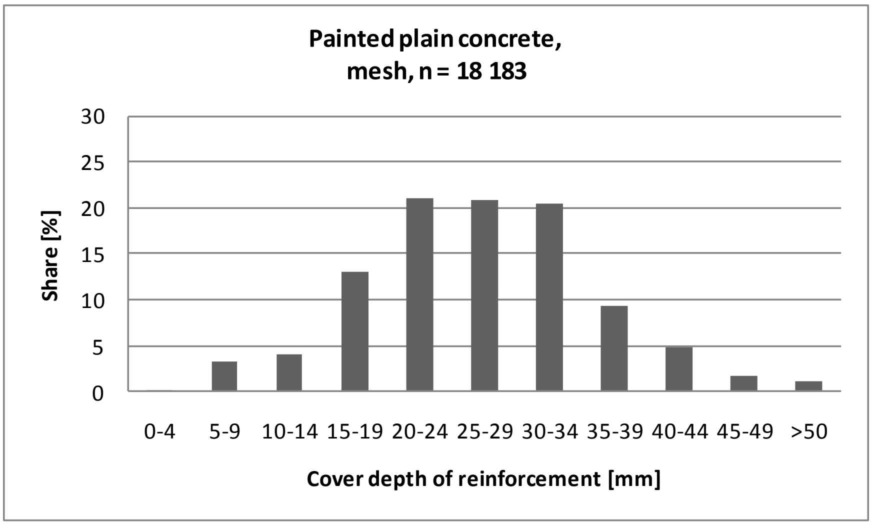


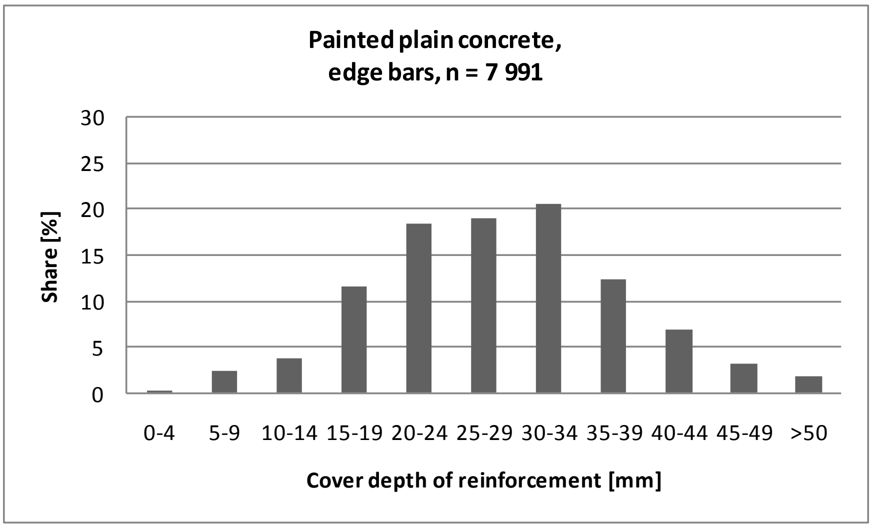


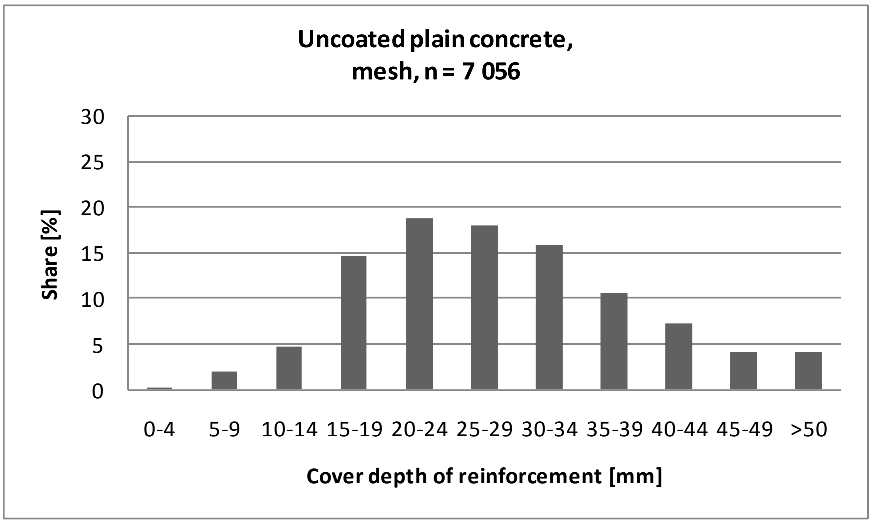

Brushed painted concrete,
edge bars, n = 36 154
30
25
20
15
10
5
0
Share [%]
0-4
5-9
10-14
15-19
20-24
25-29
30-34
35-39
40-44
45-49
≥50
Cover depth of reinforcement [mm]

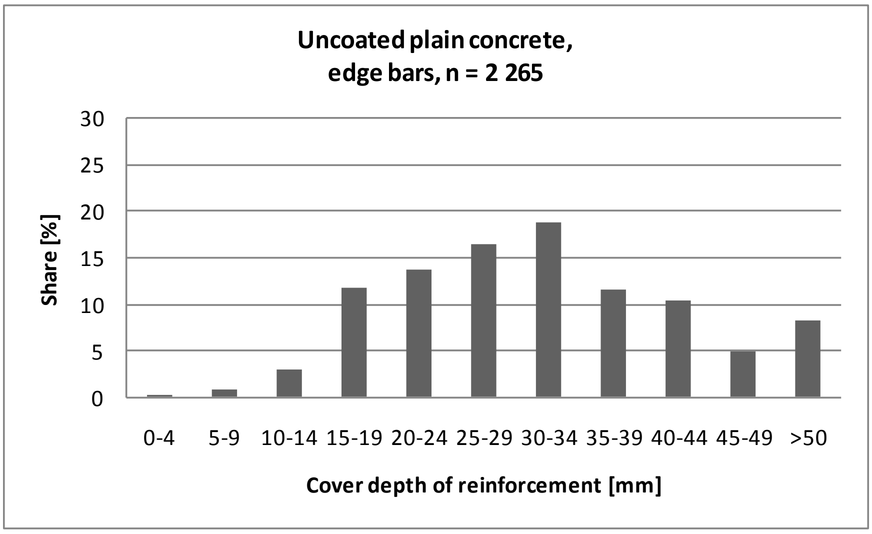

Clincer-clad finishing,
mesh, n = 8 582
30
25
20
15
10
5
0
Share [%]
0-4
5-9
10-14
15-19
20-24
25-29
30-34
35-39
40-44
45-49
≥50
Cover depth of reinforcement [mm]

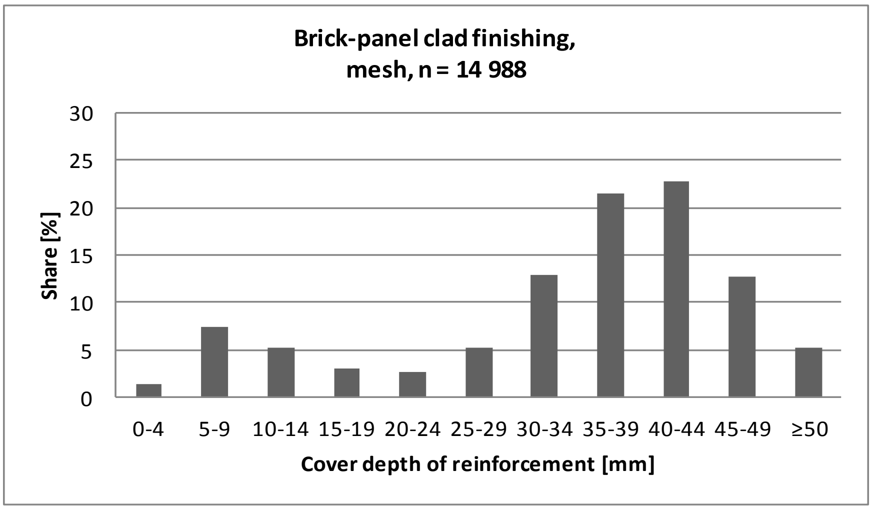

Clincer-clad finishing,
edge bars, n = 4 690
30
25
20
15
10
5
0
Share [%]
0-4
5-9
10-14
15-19
20-24
25-29
30-34
35-39
40-44
45-49
≥50
Cover depth of reinforcement [mm]

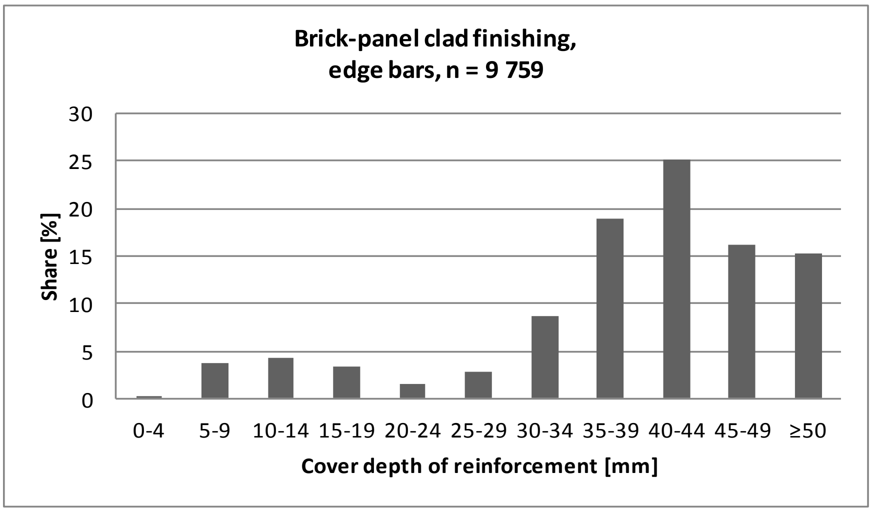

Painted plain concrete,
mesh, n = 18 183
30
25
20
15
10
5
0
Share [%]
0-4
5-9
10-14
15-19
20-24
25-29
30-34
35-39
40-44
45-49
>50
Cover depth of reinforcement [mm]

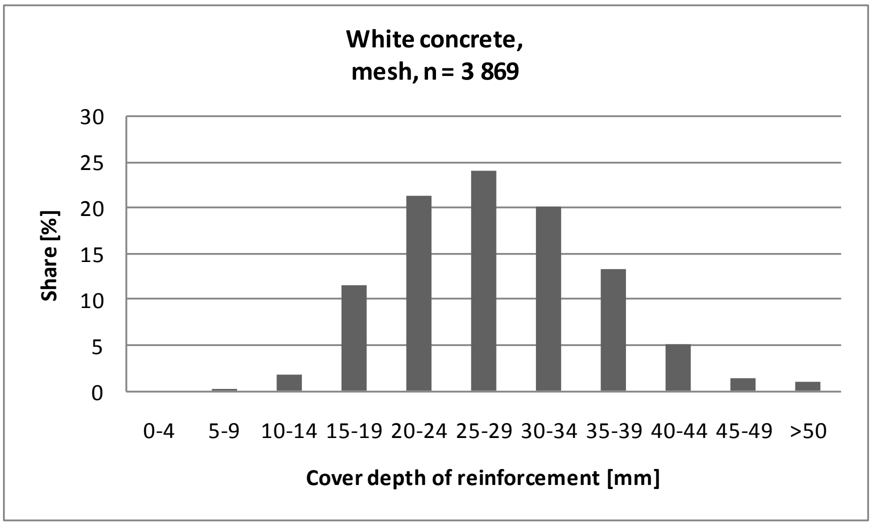

Painted plain concrete,
edge bars, n = 7 991
30
25
20
15
10
5
0
Share [%]
0-4
5-9
10-14
15-19
20-24
25-29
30-34
35-39
40-44
45-49
>50
Cover depth of reinforcement [mm]

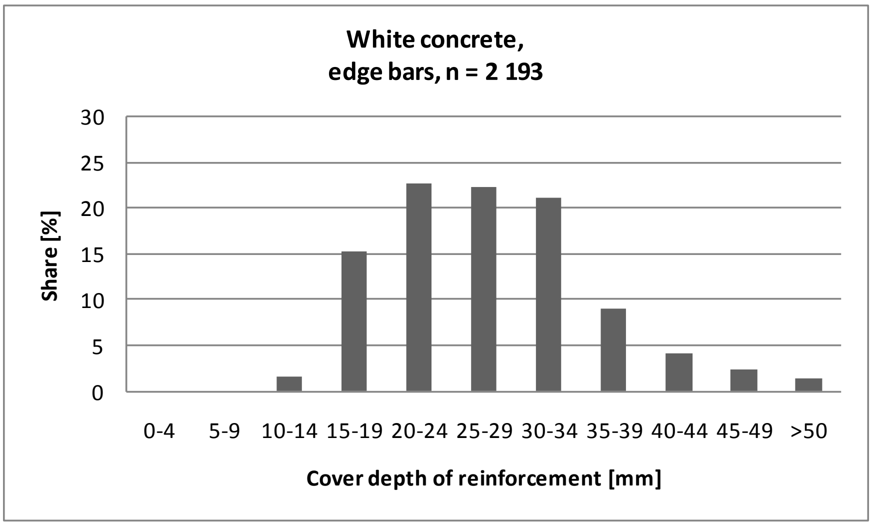

Uncoated plain concrete,
mesh, n = 7 056
30
25
20
15
10
5
0
Share [%]
0-4
5-9
10-14
15-19
20-24
25-29
30-34
35-39
40-44
45-49
>50
Cover depth of reinforcement [mm]

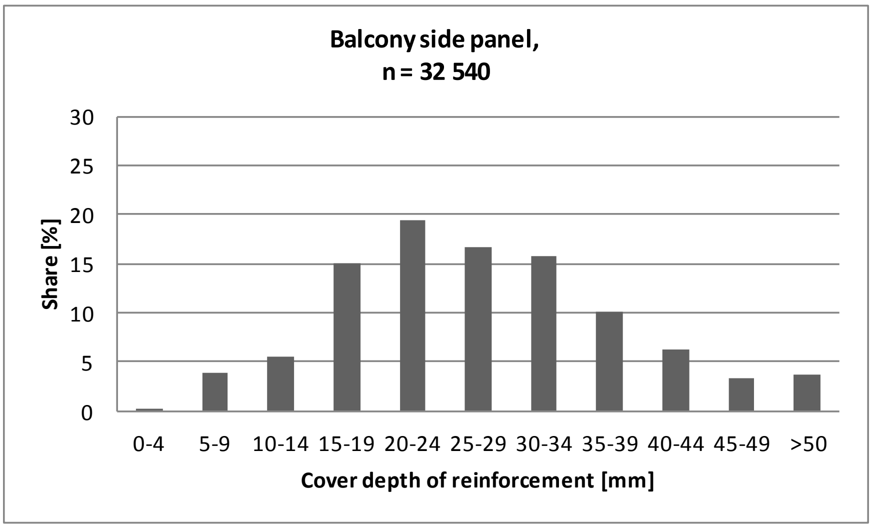

Uncoated plain concrete,
edge bars, n = 2 265
30
25
20
15
10
5
0
Share [%]
0-4
5-9
10-14
15-19
20-24
25-29
30-34
35-39
40-44
45-49
>50
Cover depth of reinforcement [mm]

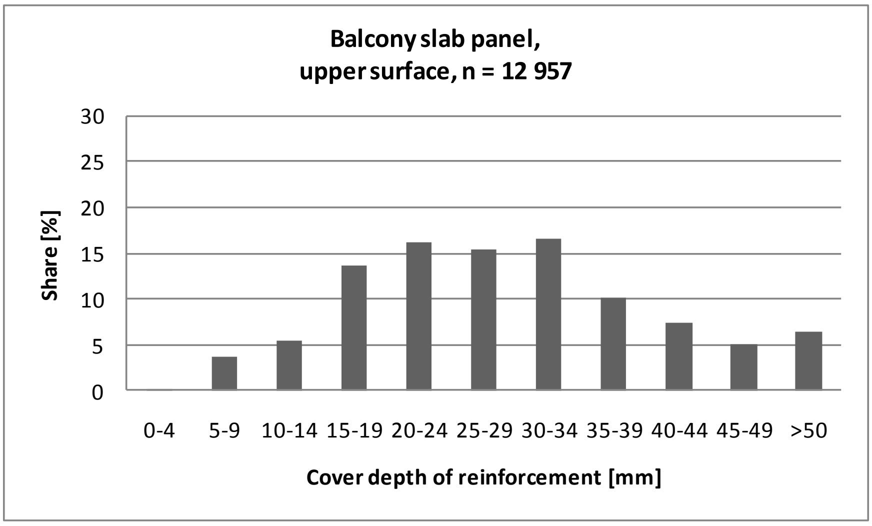

Brick-panel clad finishing,
mesh, n = 14 988
30
25
20
15
10
5
0
Share [%]
0-4
5-9
10-14
15-19
20-24
25-29
30-34
35-39
40-44
45-49
≥50
Cover depth of reinforcement [mm]


Brick-panel clad finishing,
edge bars, n = 9 759
30
25
20
15
10
5
0
Share [%]
0-4
5-9
10-14
15-19
20-24
25-29
30-34
35-39
40-44
45-49
≥50
Cover depth of reinforcement [mm]

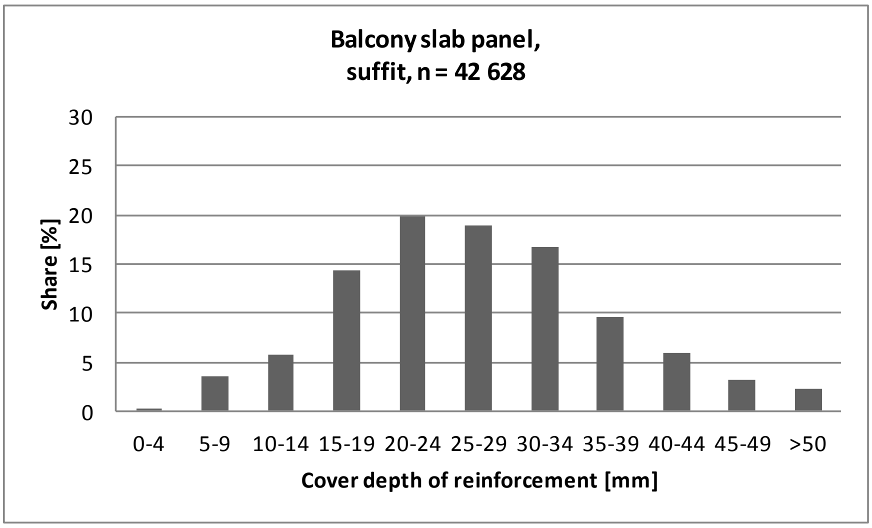

White concrete,
mesh, n = 3 869
30
25
20
15
10
5
0
Share [%]
0-4
5-9
10-14
15-19
20-24
25-29
30-34
35-39
40-44
45-49
>50
Cover depth of reinforcement [mm]


White concrete,
edge bars, n = 2 193
30
25
20
15
10
5
0
Share [%]
0-4
5-9
10-14
15-19
20-24
25-29
30-34
35-39
40-44
45-49
>50
Cover depth of reinforcement [mm]

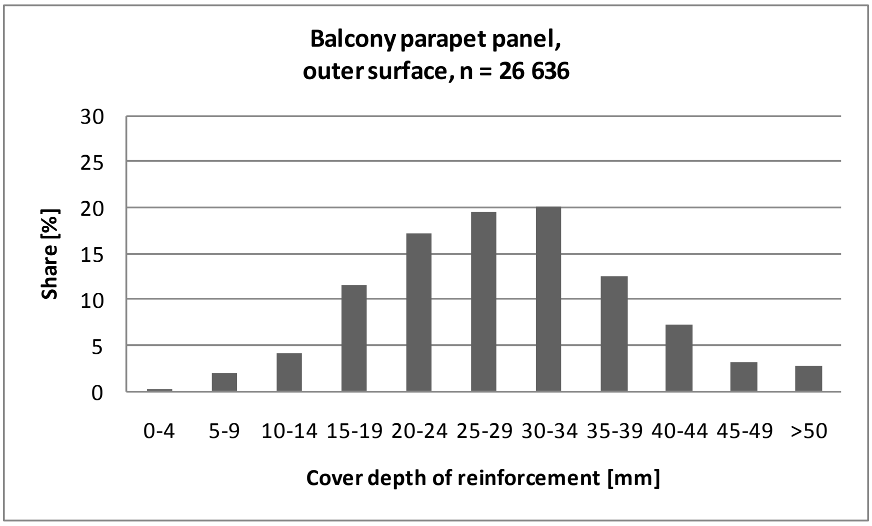

Balcony side panel,
n = 32 540
30
25
20
15
10
5
0
Share [%]
0-4
5-9
10-14
15-19
20-24
25-29
30-34
35-39
40-44
45-49
>50
Cover depth of reinforcement [mm]

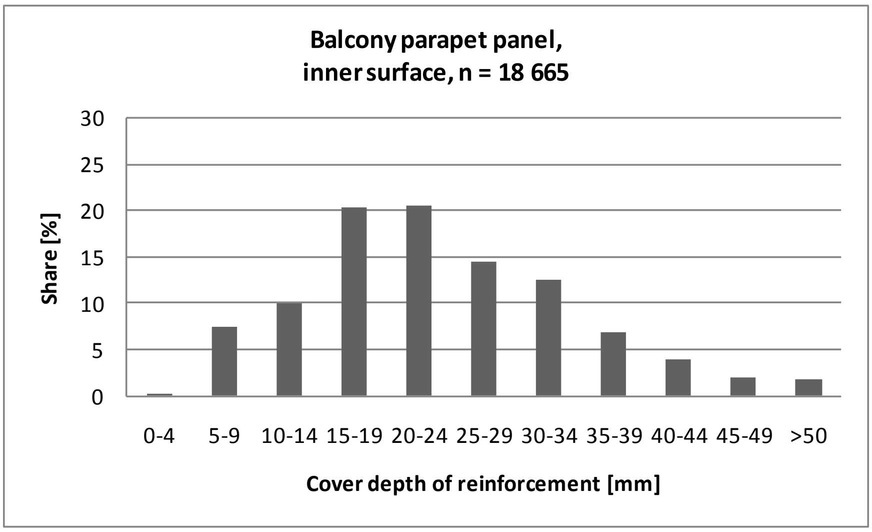


Balcony slab panel,
upper surface, n = 12 957
30
25
20
15
10
5
0
Share [%]
0-4
5-9
10-14
15-19
20-24
25-29
30-34
35-39
40-44
45-49
>50
Cover depth of reinforcement [mm]

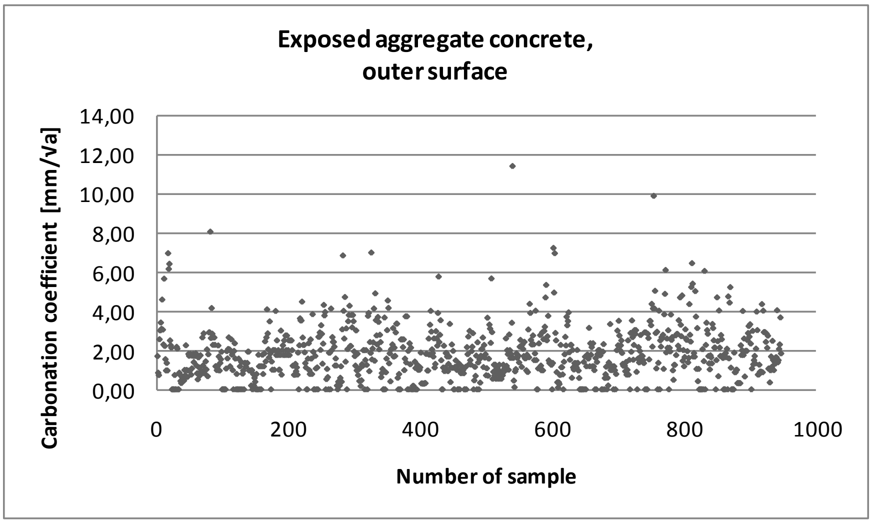


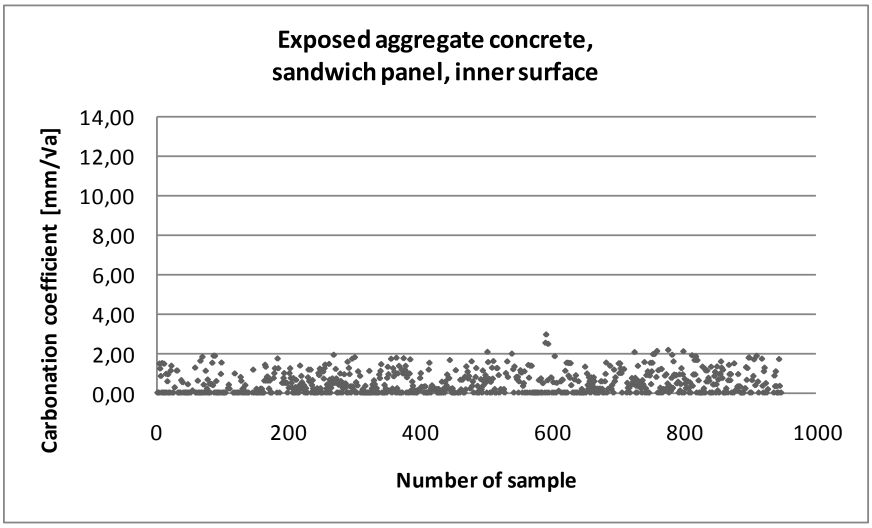
Balcony slab panel,
suffit, n = 42 628
30
25
20
15
10
5
0
Share [%]
0-4
5-9
10-14
15-19
20-24
25-29
30-34
35-39
40-44
45-49
>50
Cover depth of reinforcement [mm]

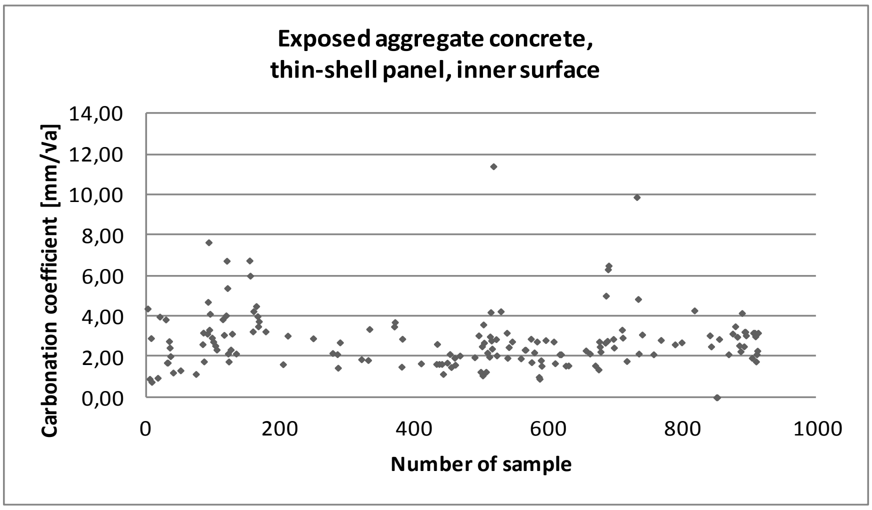

Balcony parapet panel,
outer surface, n = 26 636
Share [%]
30
25
20
15
10
5
0
0-4
5-9
10-14
15-19
20-24
25-29
30-34
35-39
40-44
45-49
>50
Cover depth of reinforcement [mm]

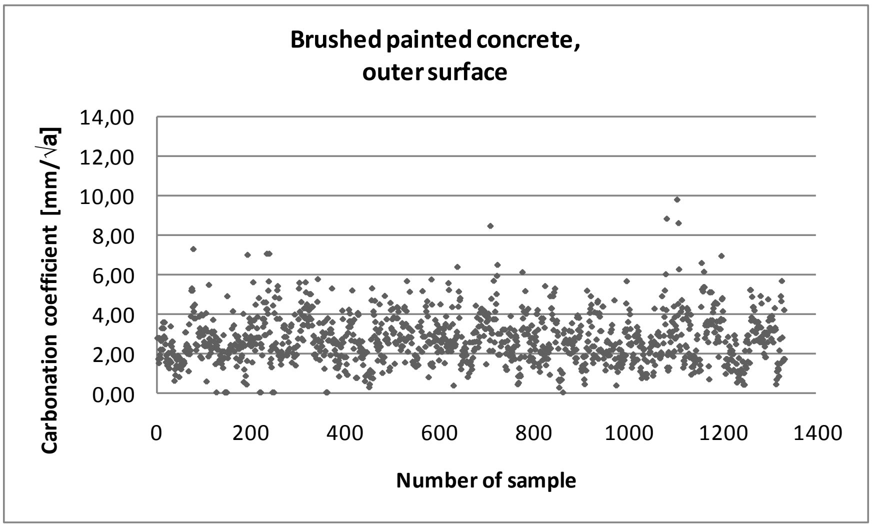

Balcony parapet panel,
inner surface, n = 18 665
Share [%]
30
25
20
15
10
5
0
0-4
5-9
10-14
15-19
20-24
25-29
30-34
35-39
40-44
45-49
>50
Cover depth of reinforcement [mm]

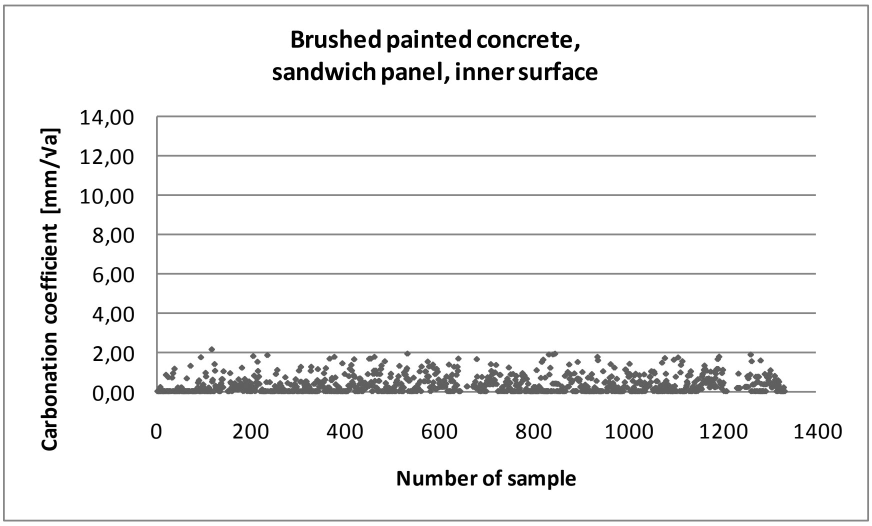
Appendix 4 Distribution of carbonation coefficient of concrete in different facade surface types and balcony elements

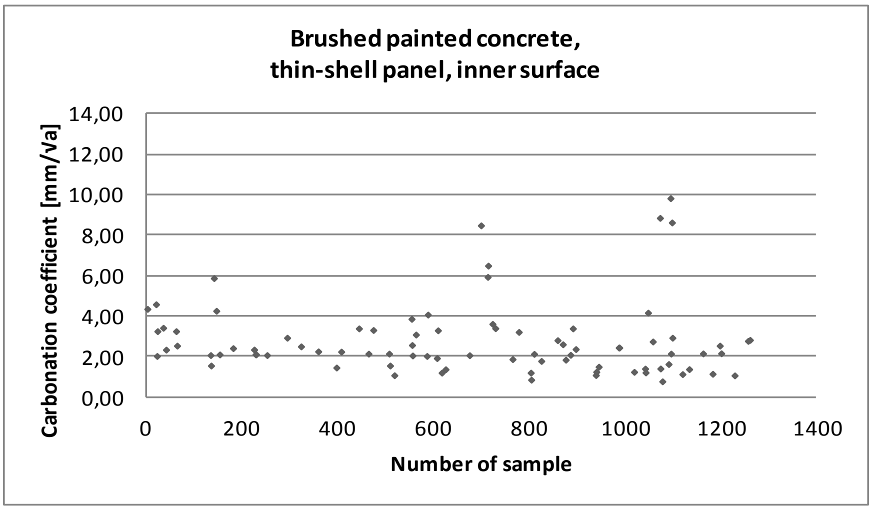


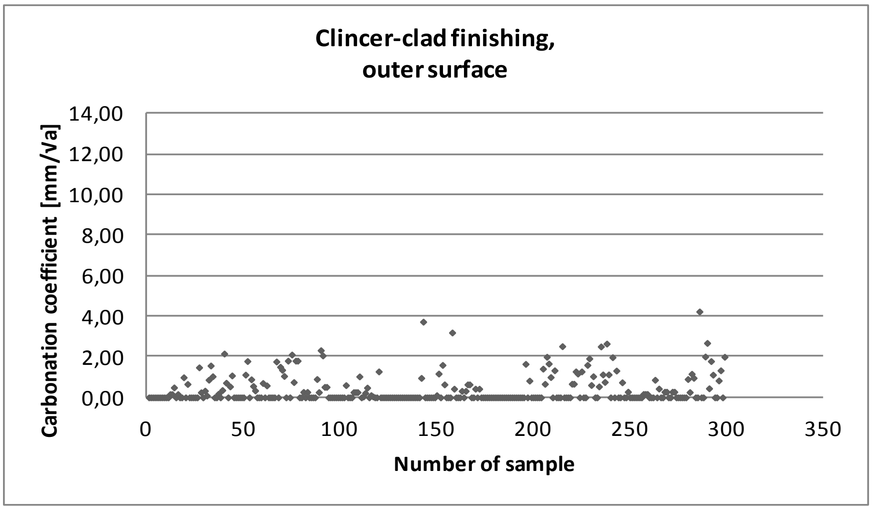


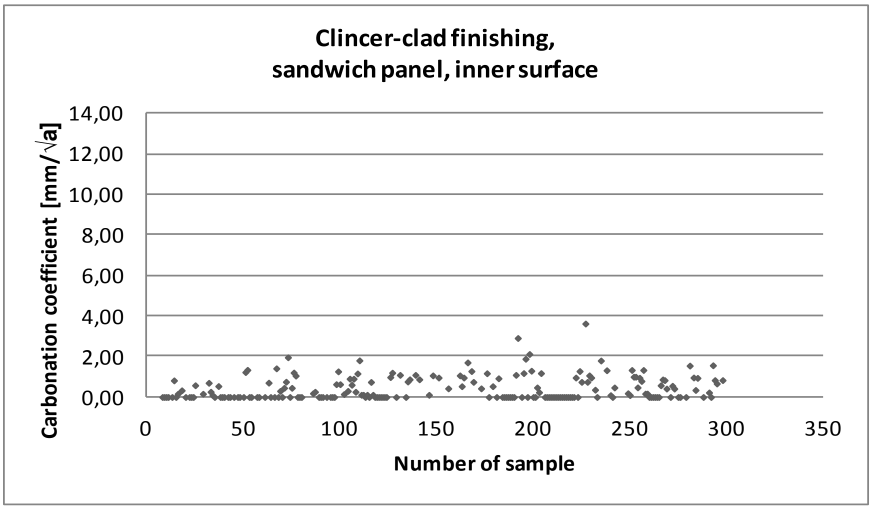


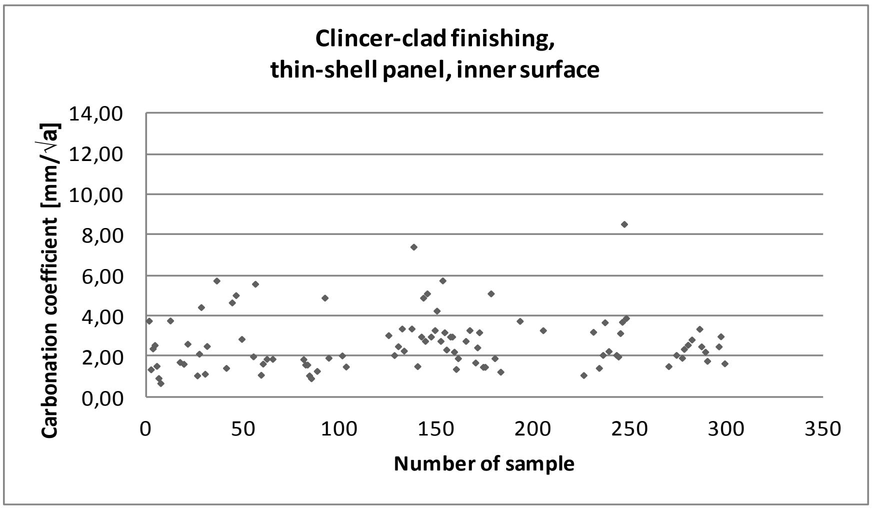

Brushed painted concrete,
outer surface
Carbonation coefficient [mm/√a]
14,00
12,00
10,00
8,00
6,00
4,00
2,00
0,00
0
200
400
600
800
1000
1200
1400
Number of sample

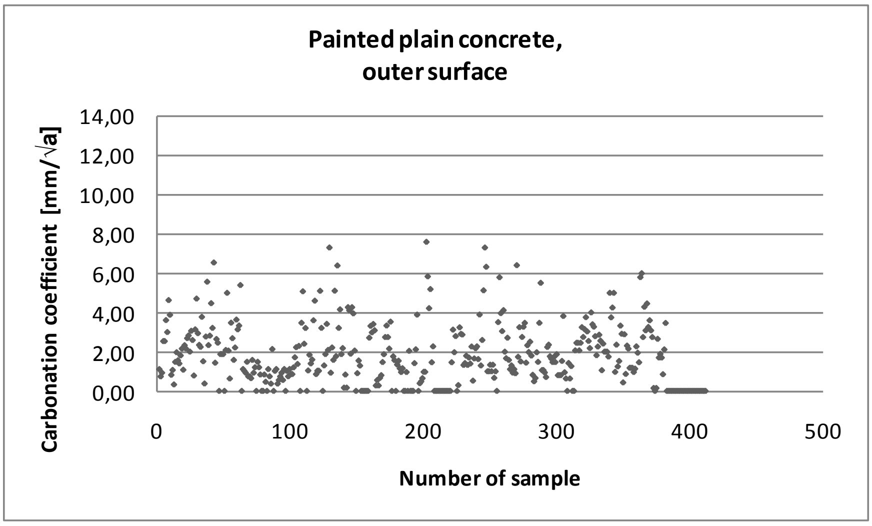

Brushed painted concrete,
sandwich panel, inner surface
Carbonation coefficient [mm/√a]
14,00
12,00
10,00
8,00
6,00
4,00
2,00
0,00
0
200
400
600
800
1000
1200
1400
Number of sample

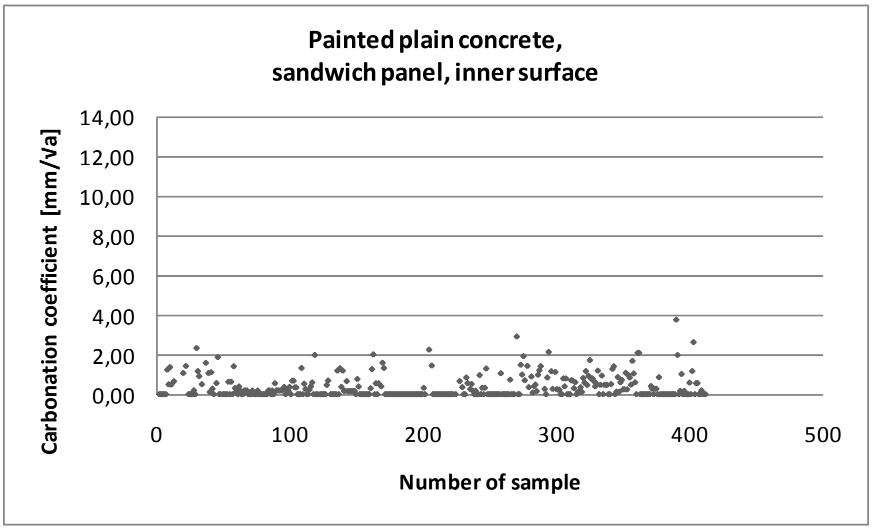

Brushed painted concrete,
thin-shell panel, inner surface
Carbonation coefficient [mm/√a]
14,00
12,00
10,00
8,00
6,00
4,00
2,00
0,00
0
200
400
600
800
1000
1200
1400
Number of sample

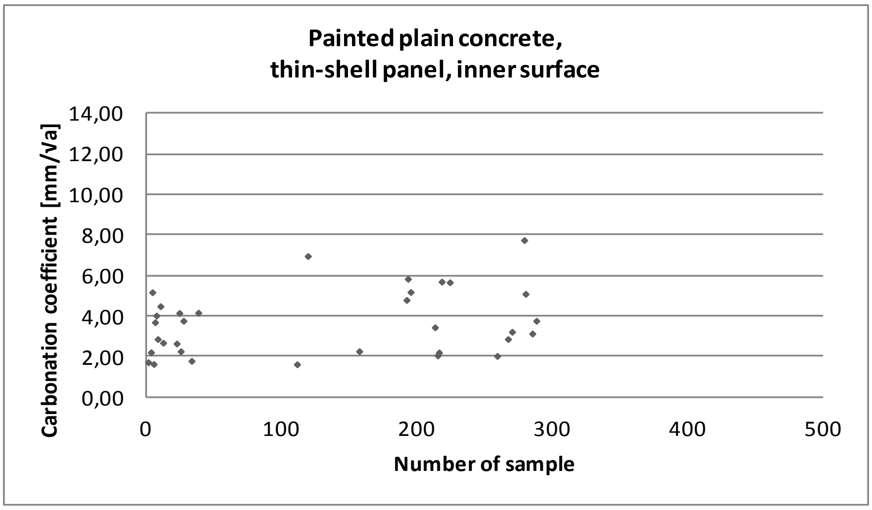

Clincer-clad finishing,
outer surface
Carbonation coefficient [mm/√a]
14,00
12,00
10,00
8,00
6,00
4,00
2,00
0,00
0
50
100
150
200
250
300
350
Number of sample

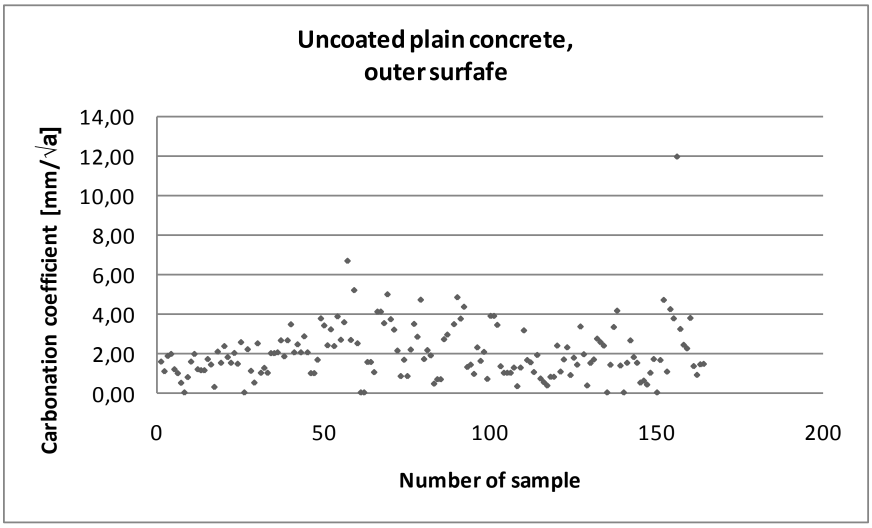

Clincer-clad finishing,
sandwich panel, inner surface
Carbonation coefficient [mm/√a]
14,00
12,00
10,00
8,00
6,00
4,00
2,00
0,00
0
50
100
150
200
250
300
350
Number of sample

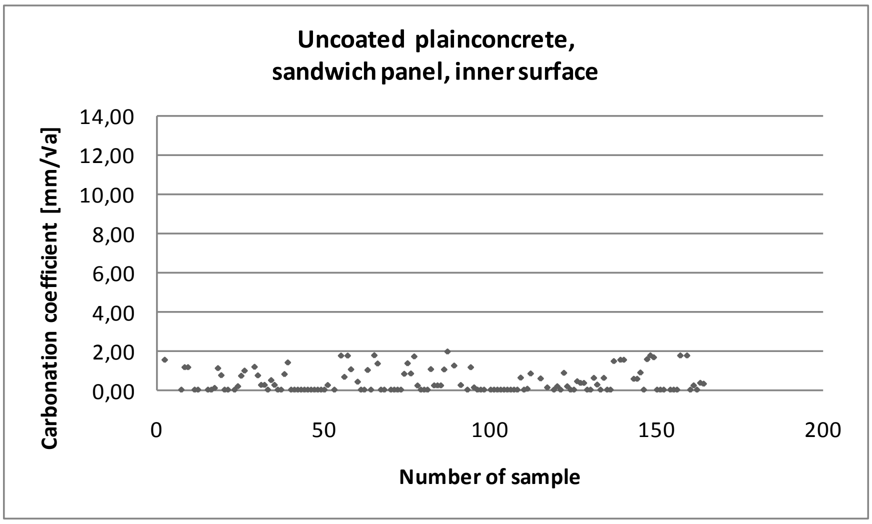

Clincer-clad finishing,
thin-shell panel, inner surface
Carbonation coefficient [mm/√a]
14,00
12,00
10,00
8,00
6,00
4,00
2,00
0,00
0
50
100
150
200
250
300
350
Number of sample

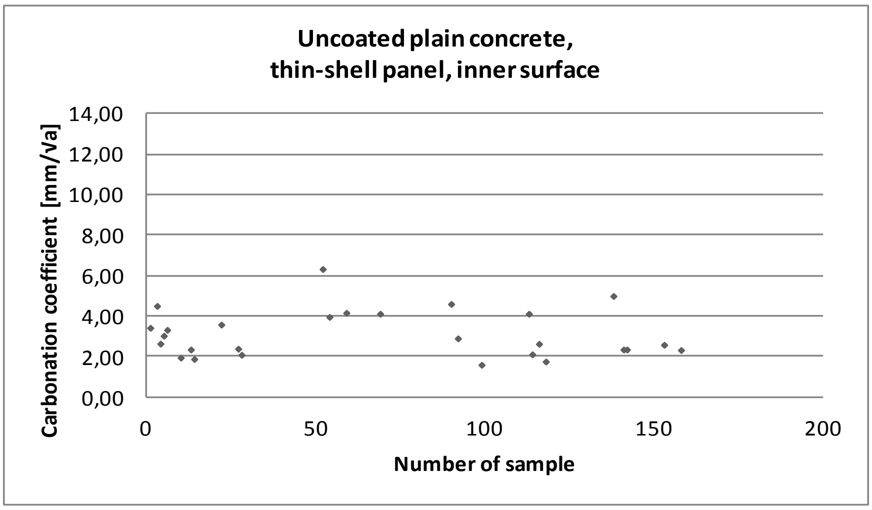

Painted plain concrete,
outer surface
Carbonation coefficient [mm/√a]
14,00
12,00
10,00
8,00
6,00
4,00
2,00
0,00
0
100
200
300
400
500
Number of sample

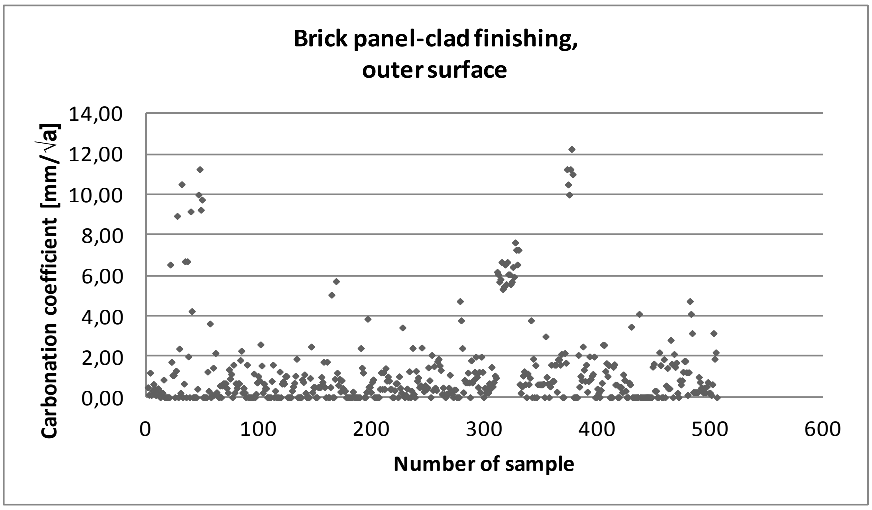

Painted plain concrete,
sandwich panel, inner surface
Carbonation coefficient [mm/√a]
14,00
12,00
10,00
8,00
6,00
4,00
2,00
0,00
0
100
200
300
400
500
Number of sample

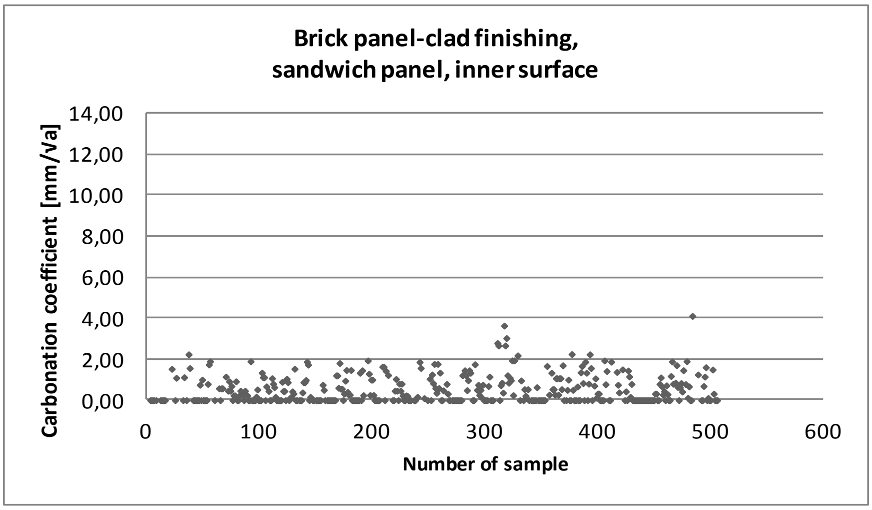

Painted plain concrete,
thin-shell panel, inner surface
Carbonation coefficient [mm/√a]
14,00
12,00
10,00
8,00
6,00
4,00
2,00
0,00
0
100
200
300
400
500
Number of sample

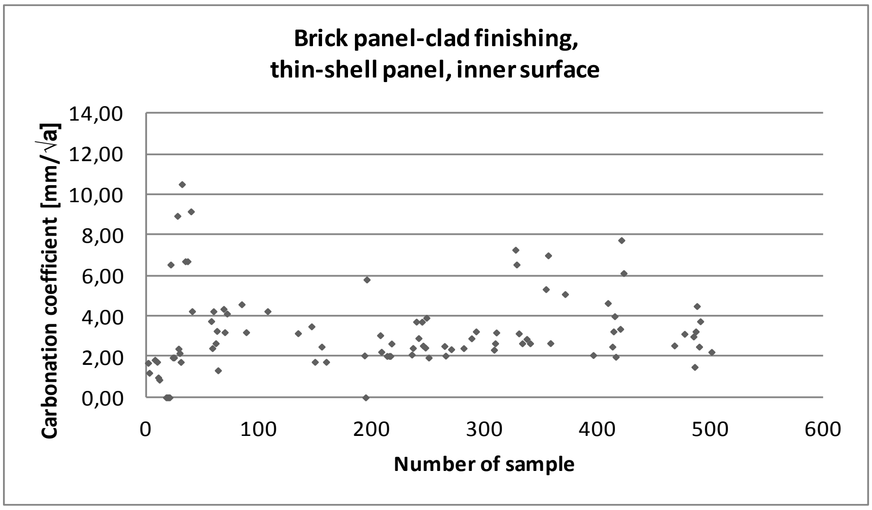

Uncoated plain concrete,
outer surfafe
Carbonation coefficient [mm/√a]
14,00
12,00
10,00
8,00
6,00
4,00
2,00
0,00
0
50
100
150
200
Number of sample

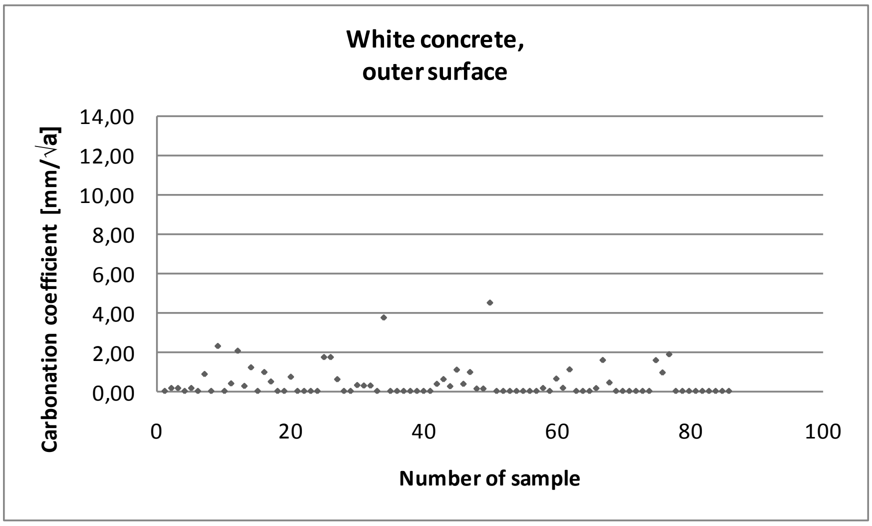

Uncoated plainconcrete,
sandwich panel, inner surface
Carbonation coefficient [mm/√a]
14,00
12,00
10,00
8,00
6,00
4,00
2,00
0,00
0
50
100
150
200
Number of sample

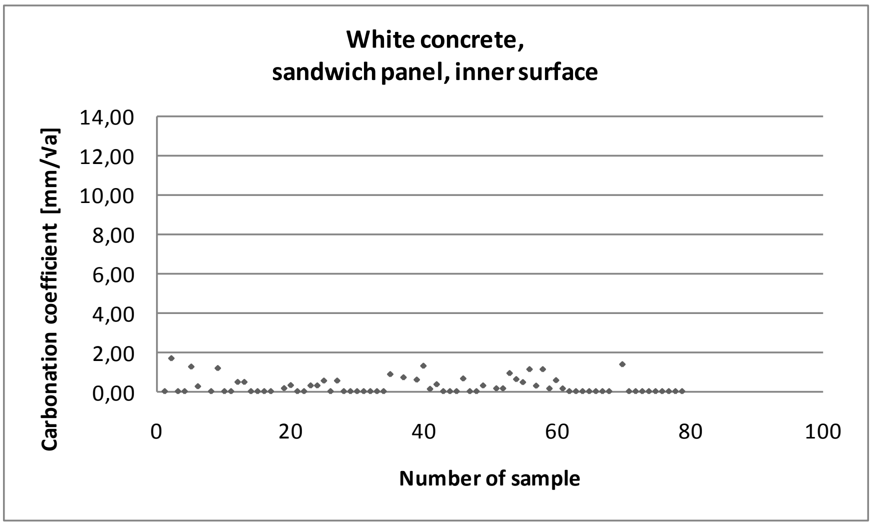

Uncoated plain concrete,
thin-shell panel, inner surface
Carbonation coefficient [mm/√a]
14,00
12,00
10,00
8,00
6,00
4,00
2,00
0,00
0
50
100
150
200
Number of sample

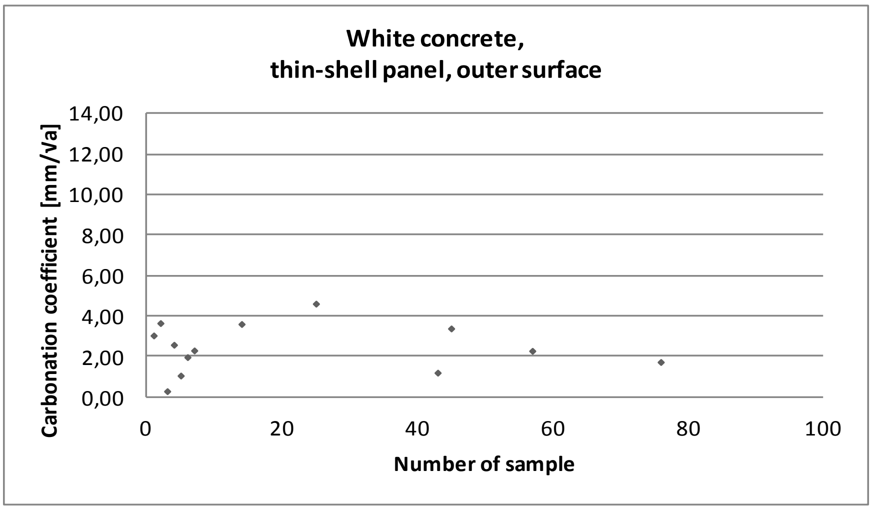

Brick panel-clad finishing,
outer surface
Carbonation coefficient [mm/√a]
14,00
12,00
10,00
8,00
6,00
4,00
2,00
0,00
0
100
200
300
400
500
600
Number of sample

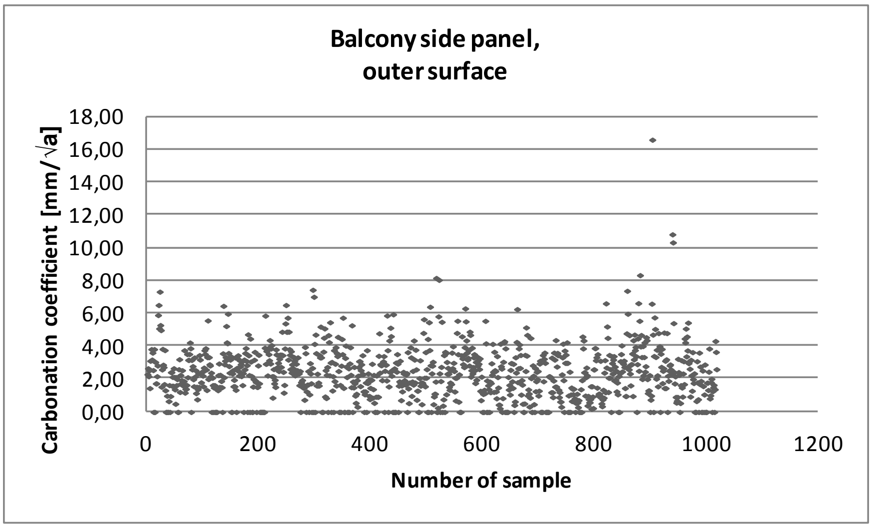

Brick panel-clad finishing,
sandwich panel, inner surface
Carbonation coefficient [mm/√a]
14,00
12,00
10,00
8,00
6,00
4,00
2,00
0,00
0
100
200
300
400
500
600
Number of sample

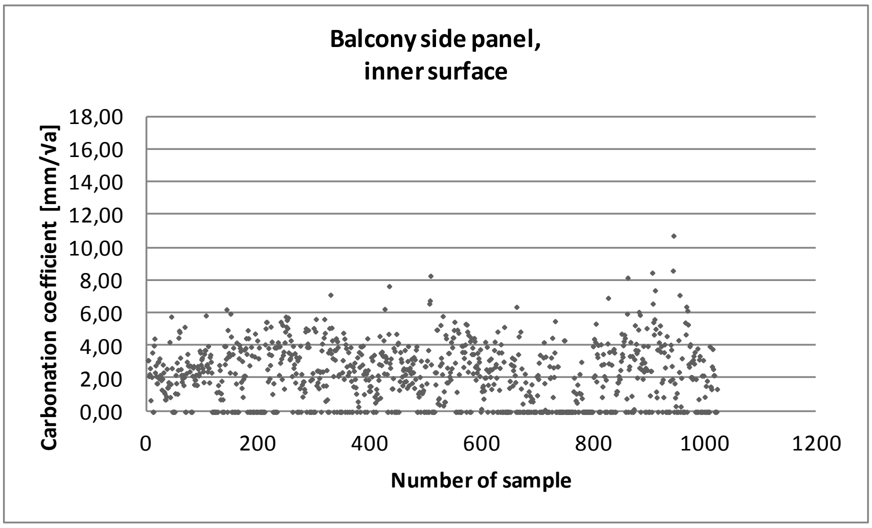

Brick panel-clad finishing,
thin-shell panel, inner surface
Carbonation coefficient [mm/√a]
14,00
12,00
10,00
8,00
6,00
4,00
2,00
0,00
0
100
200
300
400
500
600
Number of sample

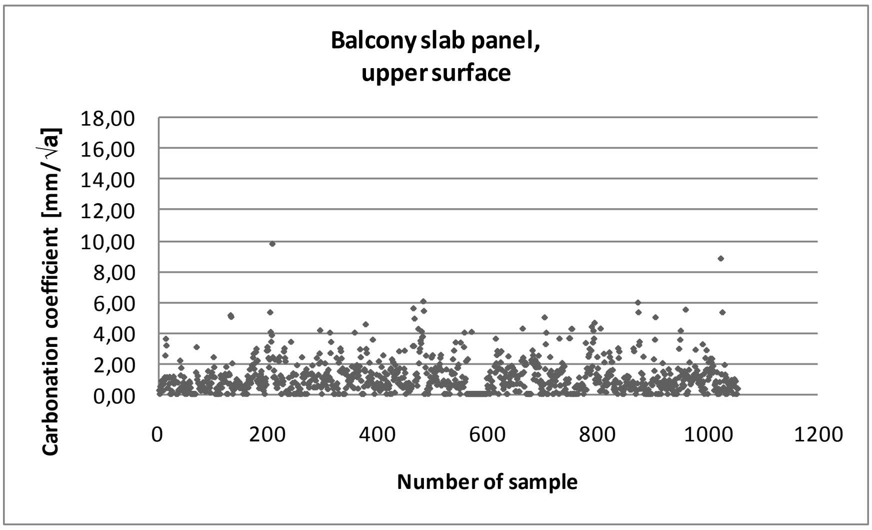

White concrete,
outer surface
Carbonation coefficient [mm/√a]
14,00
12,00
10,00
8,00
6,00
4,00
2,00
0,00
0
20
40
60
80
100
Number of sample

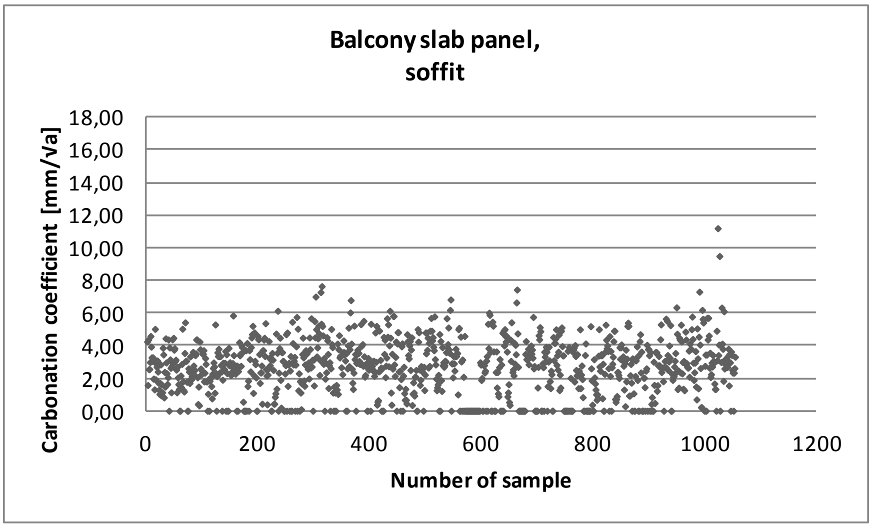

White concrete,
sandwich panel, inner surface
Carbonation coefficient [mm/√a]
14,00
12,00
10,00
8,00
6,00
4,00
2,00
0,00
0
20
40
60
80
100
Number of sample


White concrete,
thin-shell panel, outer surface
Carbonation coefficient [mm/√a]
14,00
12,00
10,00
8,00
6,00
4,00
2,00
0,00
0
20
40
60
80
100
Number of sample

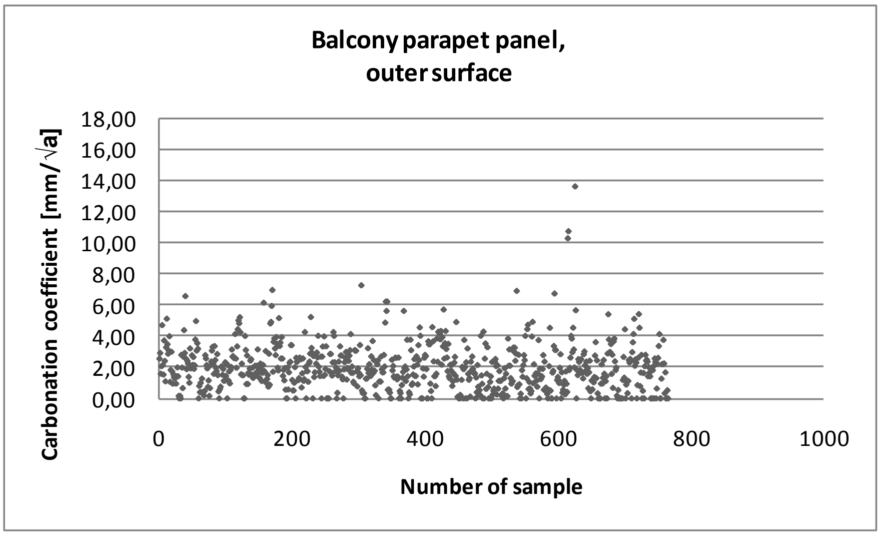

Balcony side panel,
outer surface
18,00
16,00
14,00
12,00
10,00
8,00
6,00
4,00
2,00
0,00
Carbonation coefficient [mm/√a]
0
200
400
600
800
1000
1200
Number of sample


Balcony side panel,
inner surface
18,00
16,00
14,00
12,00
10,00
8,00
6,00
4,00
2,00
0,00
Carbonation coefficient [mm/√a]
0
200
400
600
800
1000
1200
Number of sample

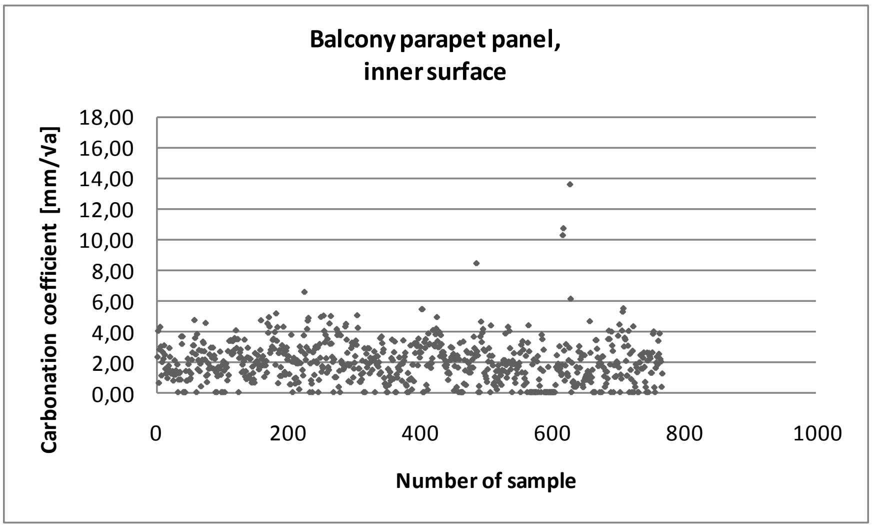

Balcony slab panel,
upper surface
18,00
16,00
14,00
12,00
10,00
8,00
6,00
4,00
2,00
0,00
Carbonation coefficient [mm/√a]
0
200
400
600
800
1000
1200
Number of sample



Balcony slab panel,
soffit
Carbonation coefficient [mm/√a]
18,00
16,00
14,00
12,00
10,00
8,00
6,00
4,00
2,00
0,00
0
200
400
600
800
1000
1200
Number of sample

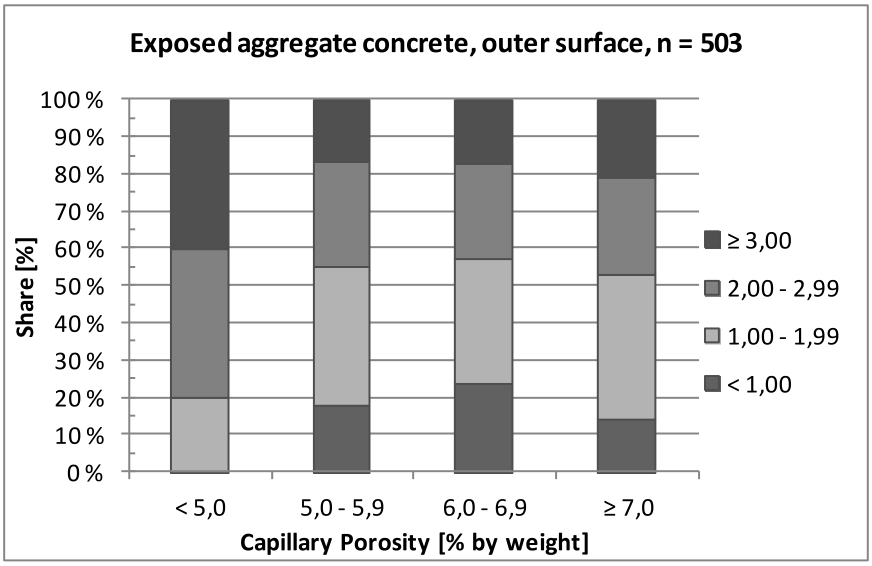


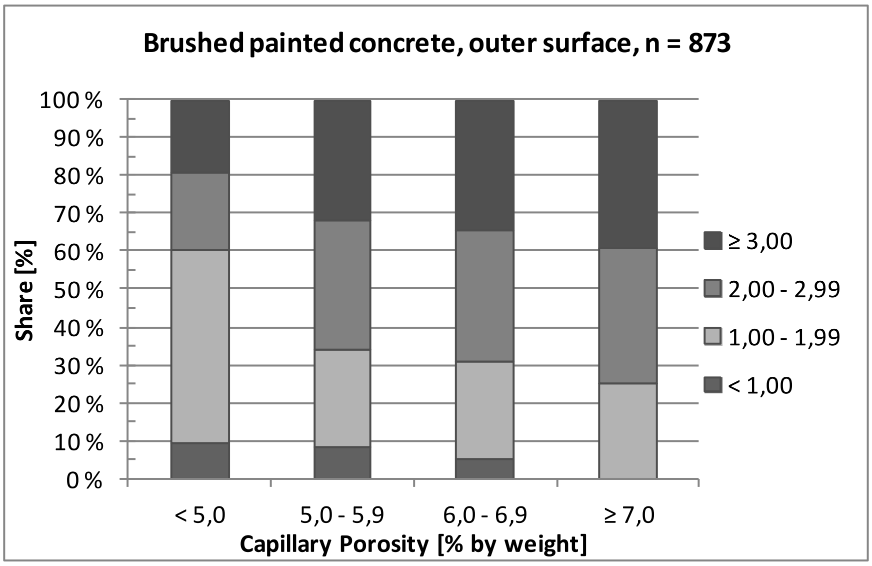
Balcony parapet panel,
outer surface
Carbonation coefficient [mm/√a]
18,00
16,00
14,00
12,00
10,00
8,00
6,00
4,00
2,00
0,00
0
200
400
600
800
1000
Number of sample

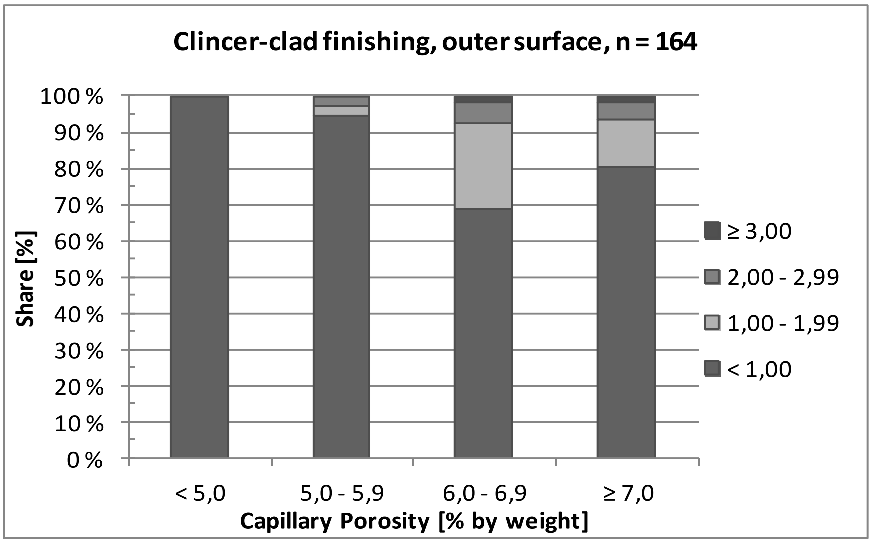


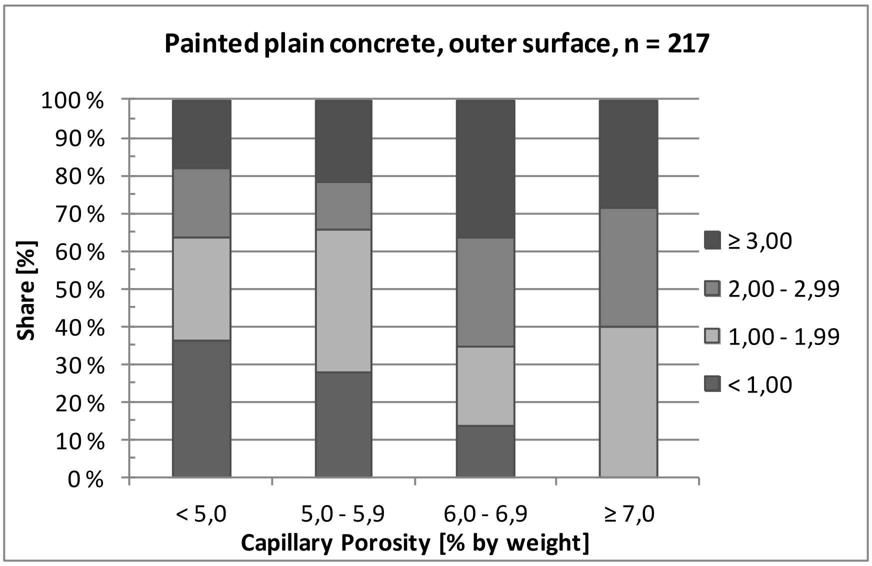
Balcony parapet panel,
inner surface
Carbonation coefficient [mm/√a]
18,00
16,00
14,00
12,00
10,00
8,00
6,00
4,00
2,00
0,00
0
200
400
600
800
1000
Number of sample

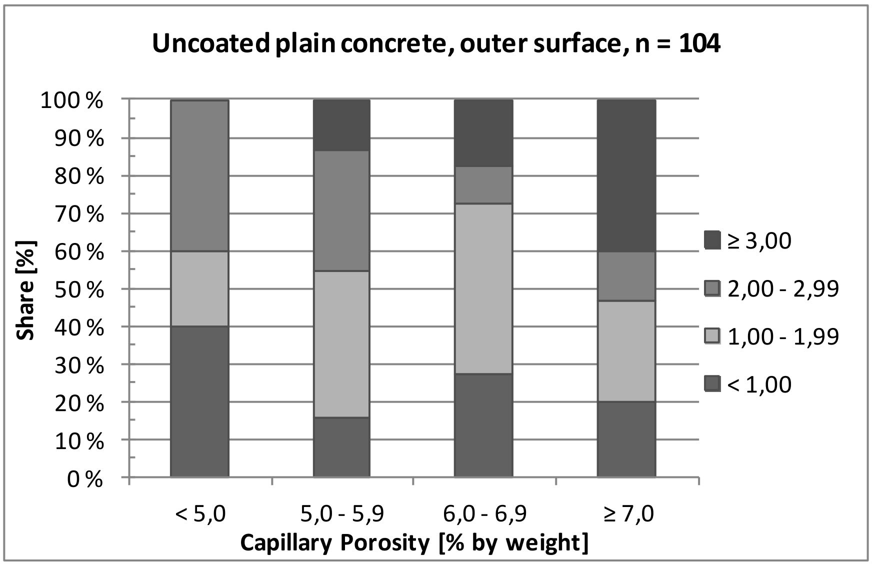
Appendix 5 Carbonation coefficient relative to the capillary porosity of concrete in different facade surface types and balcony elements

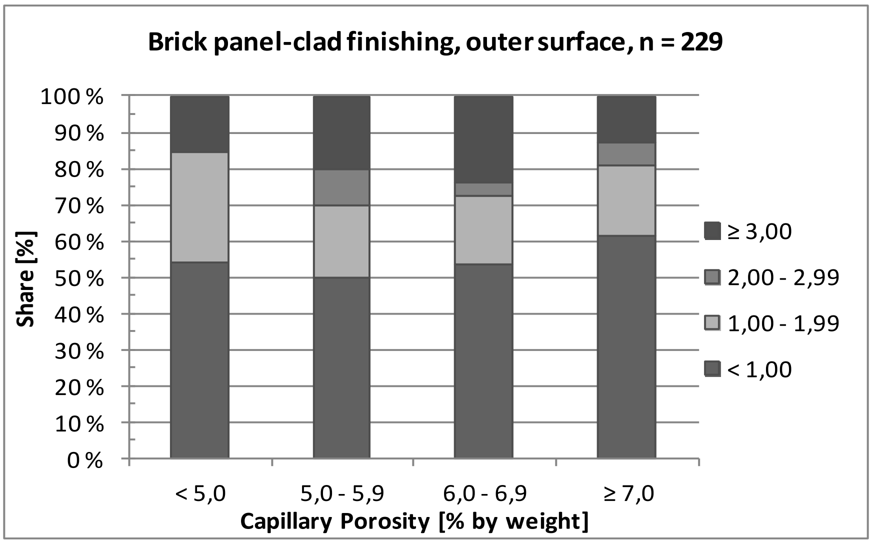


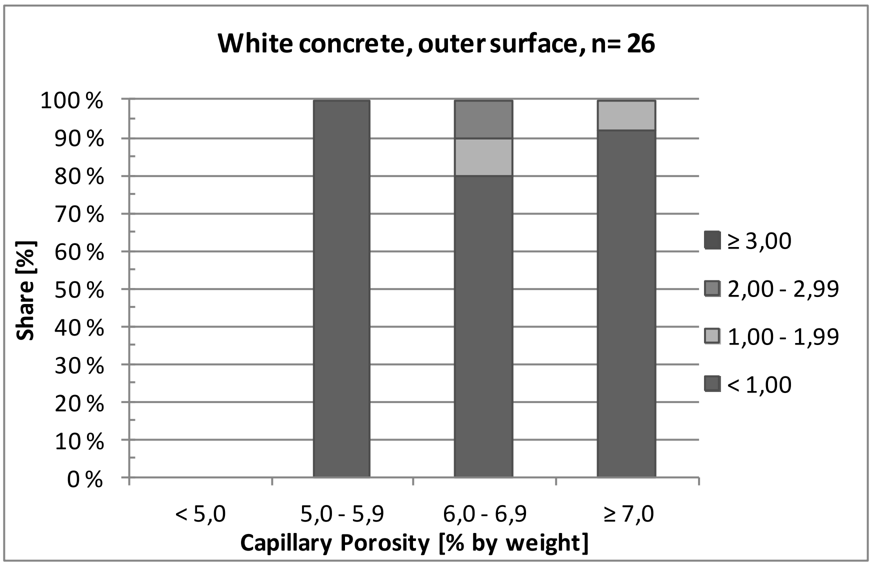


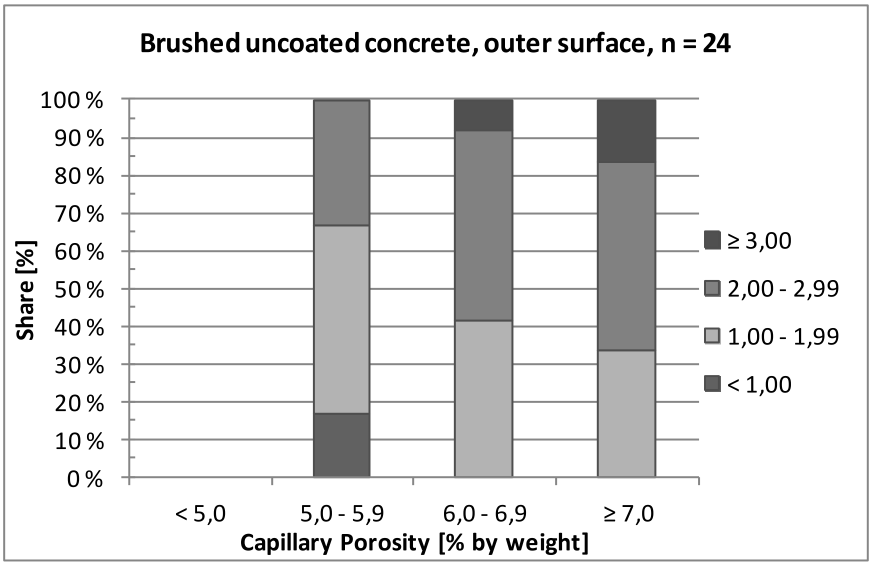


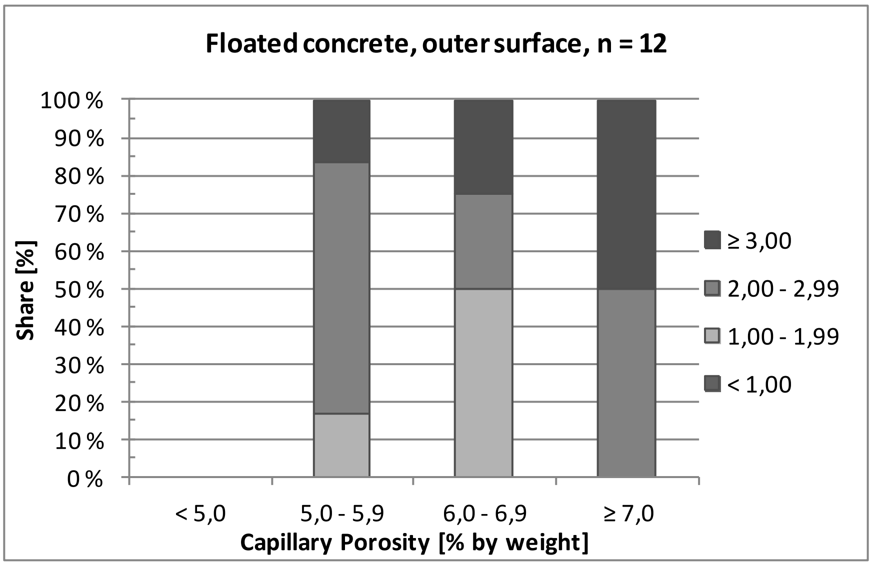

Painted plain concrete, outer surface, n = 217
100 %
90 %
80 %
70 %
60 %
50 %
40 %
30 %
20 %
10 %
0 %
Share [%]
< 5,0
5,0 - 5,9
6,0 - 6,9
≥ 7,0
Capillary Porosity [% by weight]
≥ 3,00
2,00 - 2,99
1,00 - 1,99
< 1,00

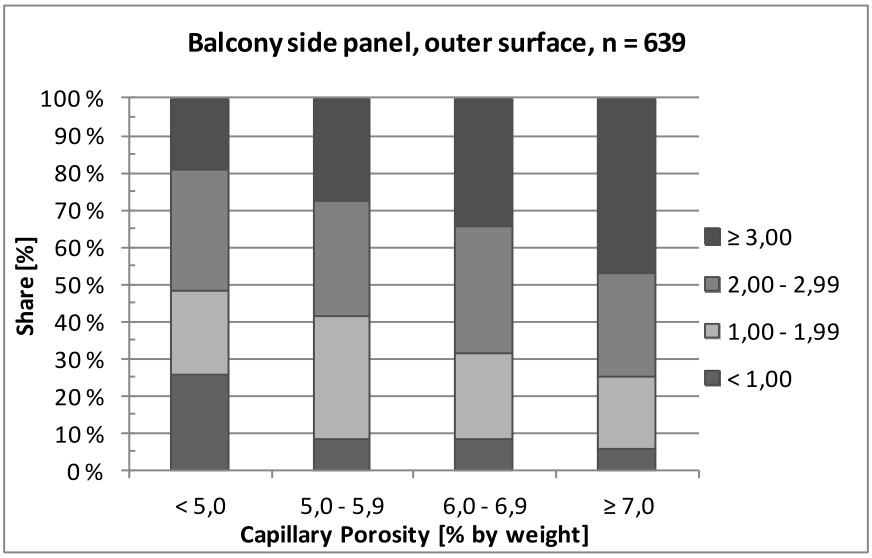

Uncoated plain concrete, outer surface, n = 104
100 %
90 %
80 %
70 %
60 %
50 %
40 %
30 %
20 %
10 %
0 %
Share [%]
< 5,0
5,0 - 5,9
6,0 - 6,9
≥ 7,0
Capillary Porosity [% by weight]
≥ 3,00
2,00 - 2,99
1,00 - 1,99
< 1,00


Brick panel-clad finishing, outer surface, n = 229
100 %
90 %
80 %
70 %
60 %
50 %
40 %
30 %
20 %
10 %
0 %
Share [%]
< 5,0
5,0 - 5,9
6,0 - 6,9
≥ 7,0
Capillary Porosity [% by weight]
≥ 3,00
2,00 - 2,99
1,00 - 1,99
< 1,00

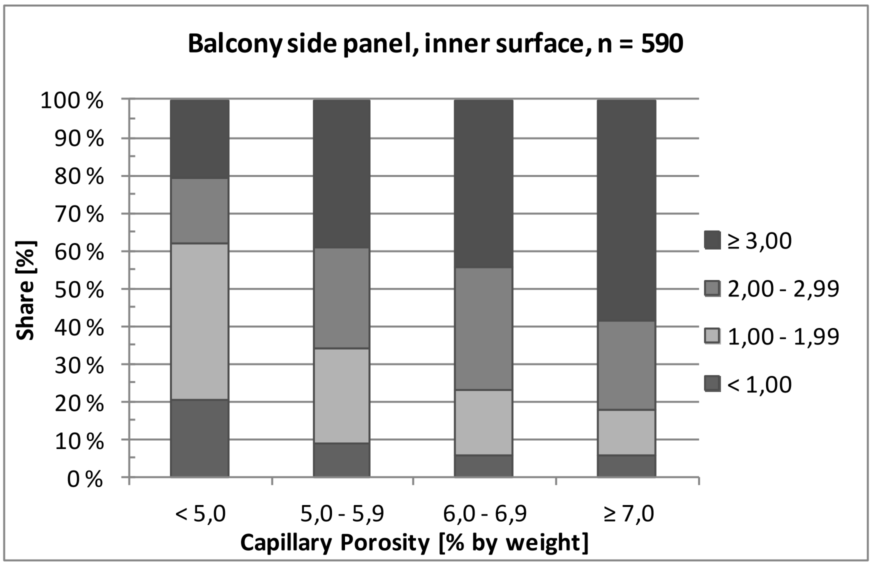

White concrete, outer surface, n= 26
100 %
90 %
80 %
70 %
60 %
50 %
40 %
30 %
20 %
10 %
0 %
Share [%]
< 5,0
5,0 - 5,9
6,0 - 6,9
≥ 7,0
Capillary Porosity [% by weight]
≥ 3,00
2,00 - 2,99
1,00 - 1,99
< 1,00


Brushed uncoated concrete, outer surface, n = 24
100 %
90 %
80 %
70 %
60 %
50 %
40 %
30 %
20 %
10 %
0 %
Share [%]
< 5,0
5,0 - 5,9
6,0 - 6,9
≥ 7,0
Capillary Porosity [% by weight]
≥ 3,00
2,00 - 2,99
1,00 - 1,99
< 1,00

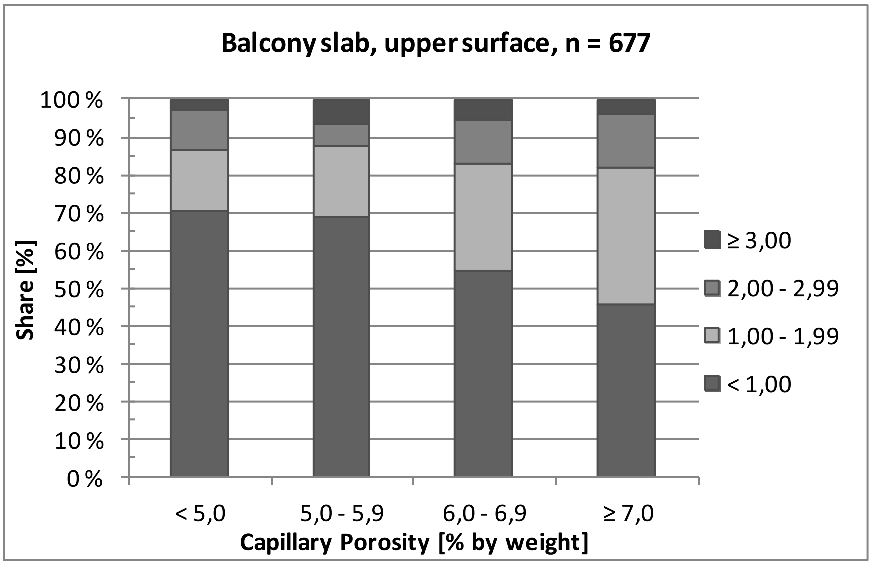

Floated concrete, outer surface, n = 12
100 %
90 %
80 %
70 %
60 %
50 %
40 %
30 %
20 %
10 %
0 %
Share [%]
< 5,0
5,0 - 5,9
6,0 - 6,9
≥ 7,0
Capillary Porosity [% by weight]
≥ 3,00
2,00 - 2,99
1,00 - 1,99
< 1,00



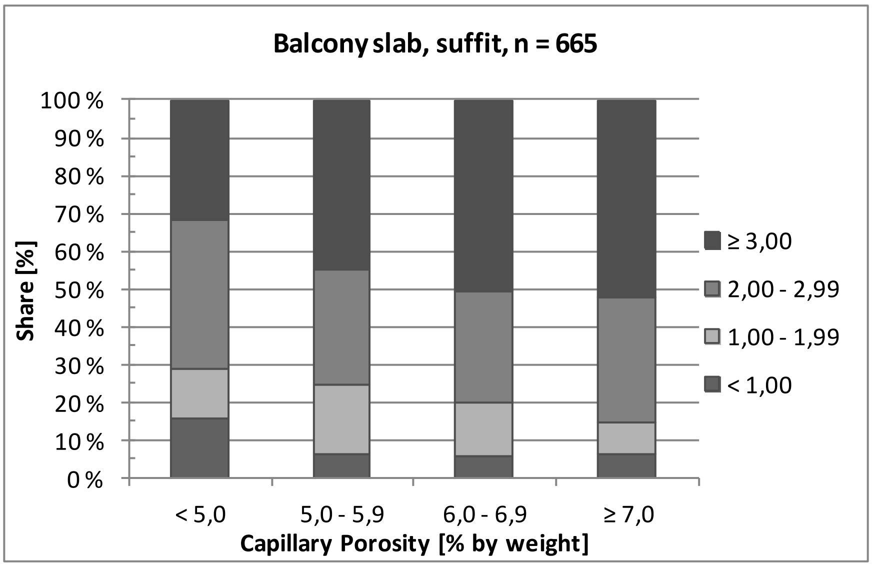
Balcony side panel, outer surface, n = 639
100 %
90 %
80 %
70 %
60 %
50 %
40 %
30 %
20 %
10 %
0 %
Share [%]
≥ 3,00
2,00 - 2,99
1,00 - 1,99
< 1,00
< 5,0
5,0 - 5,9
6,0 - 6,9
≥ 7,0
Capillary Porosity [% by weight]



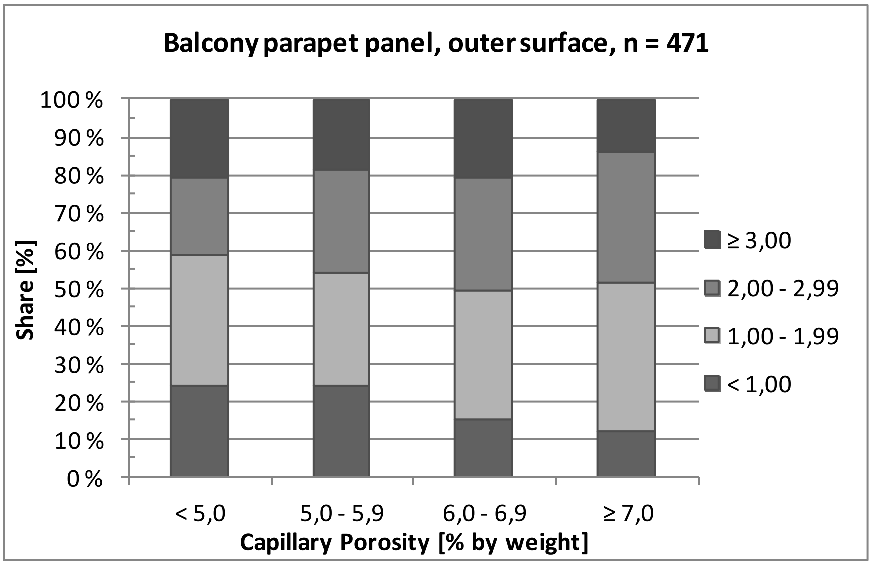
Balcony side panel, inner surface, n = 590
100 %
90 %
80 %
70 %
60 %
50 %
40 %
30 %
20 %
10 %
0 %
Share [%]
≥ 3,00
2,00 - 2,99
1,00 - 1,99
< 1,00
< 5,0
5,0 - 5,9
6,0 - 6,9
≥ 7,0
Capillary Porosity [% by weight]



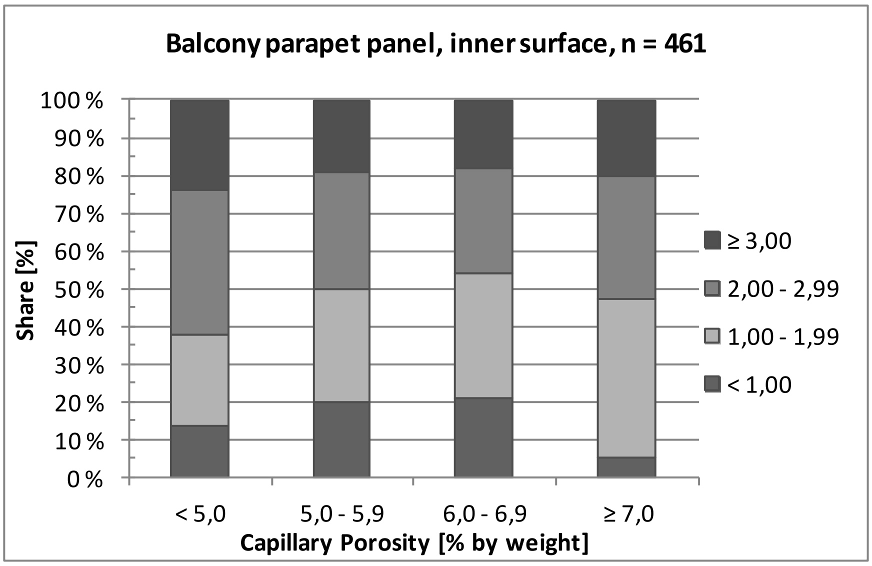
Balcony slab, upper surface, n = 677
100 %
90 %
80 %
70 %
60 %
50 %
40 %
30 %
20 %
10 %
0 %
Share [%]
≥ 3,00
2,00 - 2,99
1,00 - 1,99
< 1,00
< 5,0
5,0 - 5,9
6,0 - 6,9
≥ 7,0
Capillary Porosity [% by weight]



Balcony slab, suffit, n = 665
Share [%]
100 %
90 %
80 %
70 %
60 %
50 %
40 %
30 %
20 %
10 %
0 %
< 5,0
5,0 - 5,9
6,0 - 6,9
≥ 7,0
Capillary Porosity [% by weight]
≥ 3,00
2,00 - 2,99
1,00 - 1,99
< 1,00



Balcony parapet panel, outer surface, n = 471
Share [%]
100 %
90 %
80 %
70 %
60 %
50 %
40 %
30 %
20 %
10 %
0 %
< 5,0
5,0 - 5,9
6,0 - 6,9
≥ 7,0
Capillary Porosity [% by weight]
≥ 3,00
2,00 - 2,99
1,00 - 1,99
< 1,00



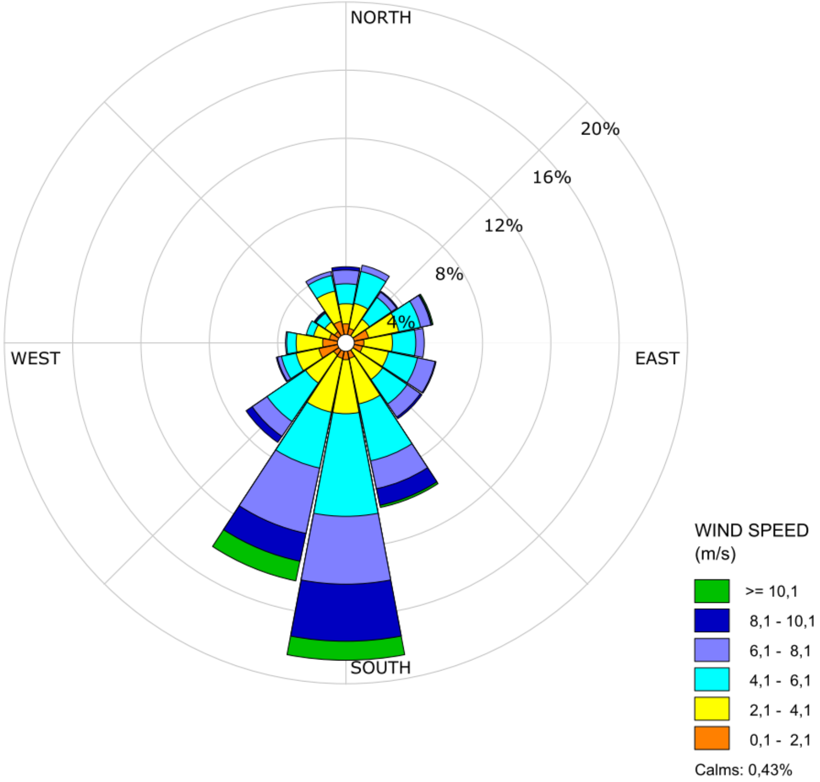
Balcony parapet panel, inner surface, n = 461
Share [%]
100 %
90 %
80 %
70 %
60 %
50 %
40 %
30 %
20 %
10 %
0 %
< 5,0
5,0 - 5,9
6,0 - 6,9
≥ 7,0
Capillary Porosity [% by weight]
≥ 3,00
2,00 - 2,99
1,00 - 1,99
< 1,00

Appendix 6 Annual precipitations without snowfall during 1961 and 2005

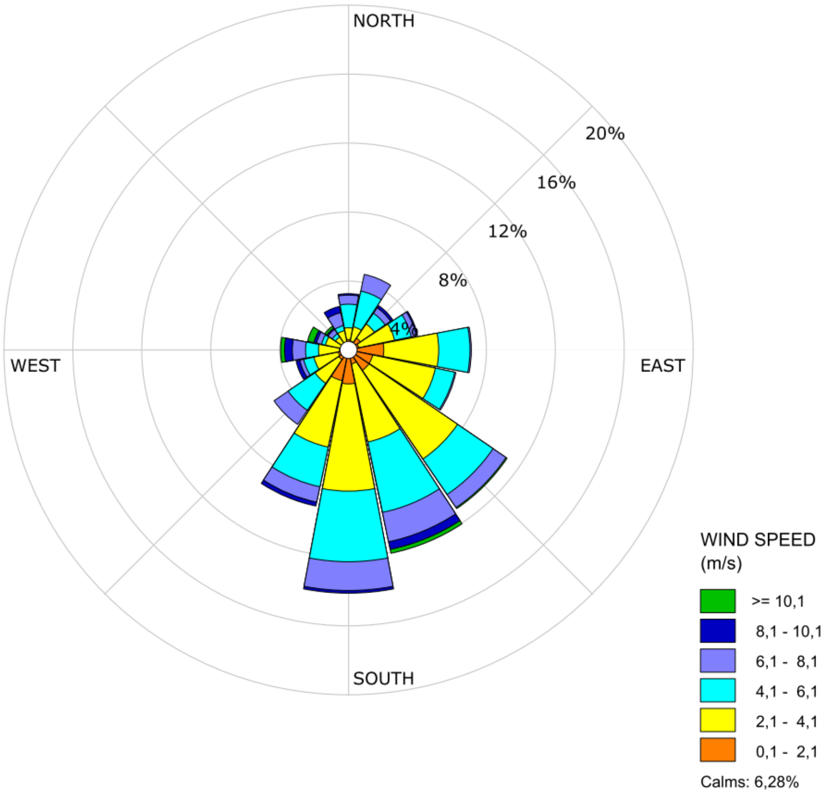
Year	Helsinki-Vantaa		Turku		Jyväskylä	
	Number of rainy days	Amount of water [mm]	Number of rainy days	Amount of water [mm]	Number of rainy days	Amount of water [mm]
1961	164	562.8	170	597.7	151	569.6
1962	174	723.0	178	554.3	161	497.6
1963	135	457.9	135	468.0	131	416.6
1964	143	367.4	146	463.7	145	445.0
1965	156	451.0	151	372.1	132	404.0
1966	144	423.7	162	482.2	148	439.7
1967	156	572.0	176	607.8	159	542.6
1968	138	468.6	129	463.4	133	465.0
1969	125	412.5	137	468.2	114	375.7
1970	138	501.4	157	471.3	131	392.3
1971	135	330.7	176	359.8	120	423.0
1972	149	528.4	181	605.1	134	437.7
1973	142	388.2	156	377.4	134	438.4
1974	168	685.2	204	698.5	147	565.1
1975	147	377.5	167	392.1	128	348.1
1976	117	307.6	127	311.5	100	334.3
1977	168	575.2	182	603.9	150	423.5
1978	142	414.3	172	524.6	127	293.6
1979	143	440.3	174	527.5	149	415.8
1980	132	529.2	159	644.9	100	426.9
1981	152	649.3	142	635.0	134	589.0
1982	158	507.0	153	562.8	130	390.4
1983	160	479.7	172	662.1	146	521.2
1984	167	577.0	181	696.9	141	449.4
1985	149	544.4	141	466.4	118	440.2
1986	159	693.3	155	673.6	156	561.9
1987	144	522.3	141	580.1	130	513.4
1988	148	487.6	163	587.1	131	489.6
1989	198	575.1	183	568.6	172	504.4
1990	178	630.4	177	693.2	151	522.9
1991	182	517.1	192	646.3	165	513.4
1992	181	607.1	171	553.1	147	436.8
1993	137	471.1	154	533.5	132	339.4
1994	165	608.2	151	537.8	141	477.9
1995	157	449.3	154	502.7	140	384.7
1996	152	655.6	135	538.7	135	480.9
1997	133	422.0	137	566.7	124	392.4
1998	172	675.6	170	593.7	160	524.0
1999	153	440.1	143	561.4	146	409.2
2000	175	597.2	179	626.7	176	482.2
2001	165	542.8	150	667.9	148	425.9
2002	103	283.7	125	401.2	122	338.2
2003	169	394.6	152	480.0	170	497.3
2004	176	712.9	179	673.3	165	546.4
2005	152	525.3	140	615.7	143	472.1

Appendix 7 Wind directions and wind speed during annual liquid precipitation during 1/1981 and 12/1985

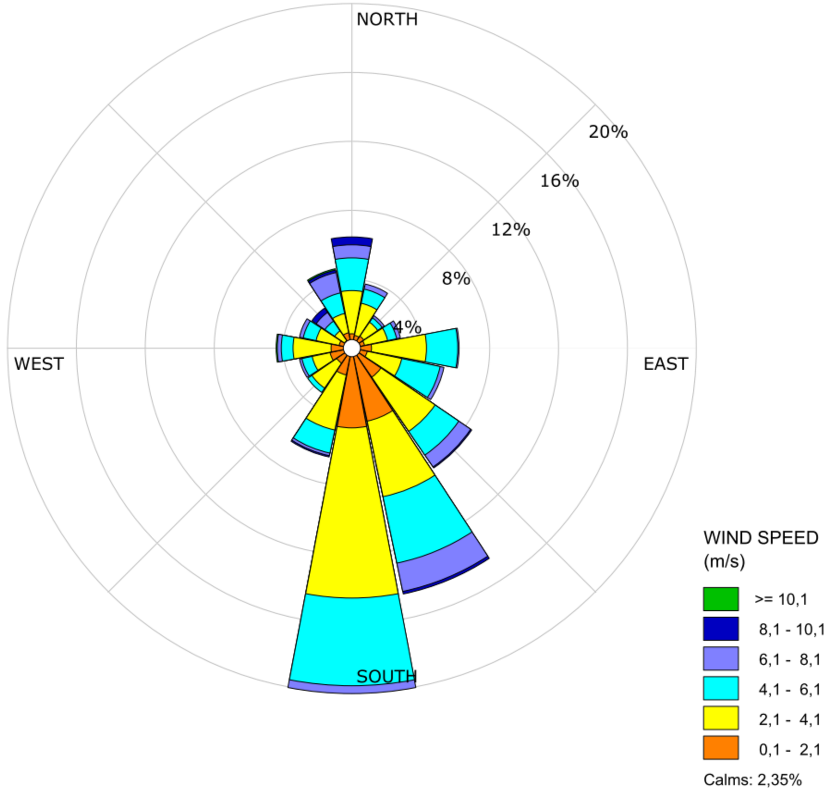
Helsinki-Vantaa airport



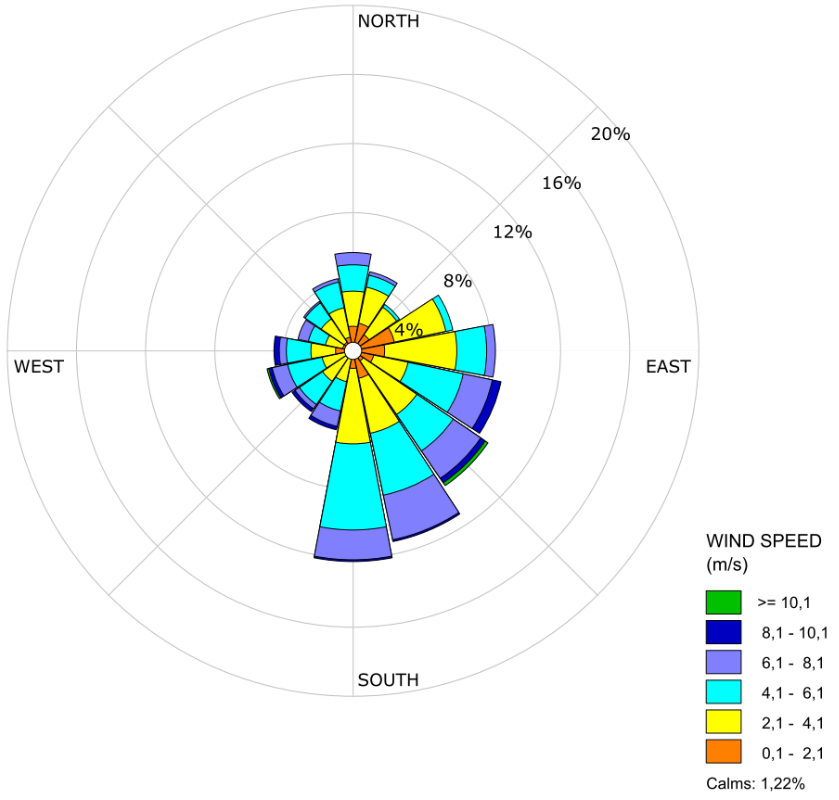
Turku airport



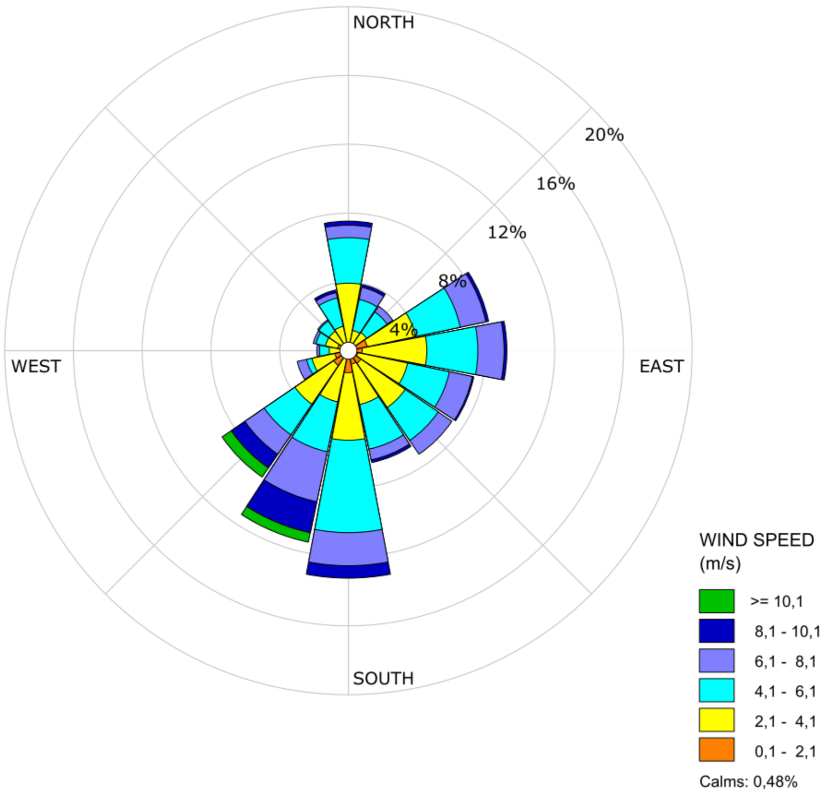
Jyväskylä airport



Oulu airport

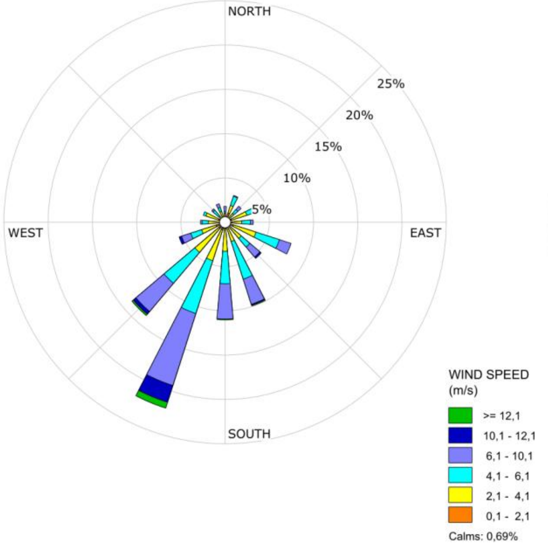


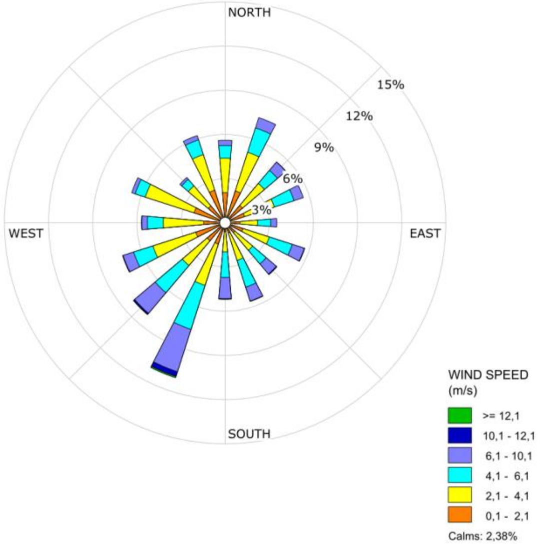
Rovaniemi airport



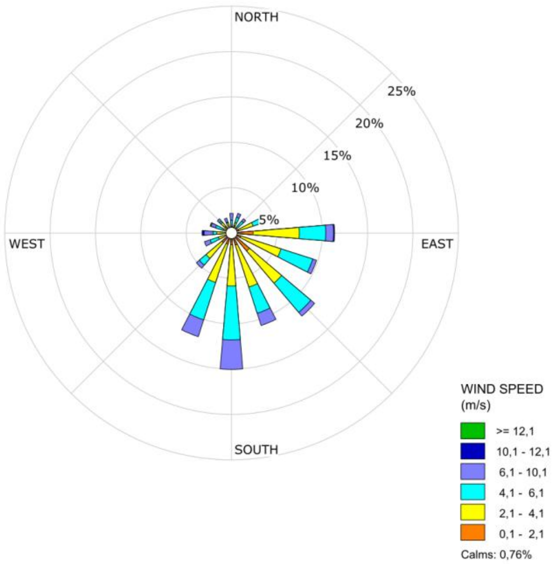
Appendix 8 Wind directions and wind speed in winter time during liquid precipitation on the left and in all times including snow fall and dry weather on the right during 9/1975 and 4/1980.

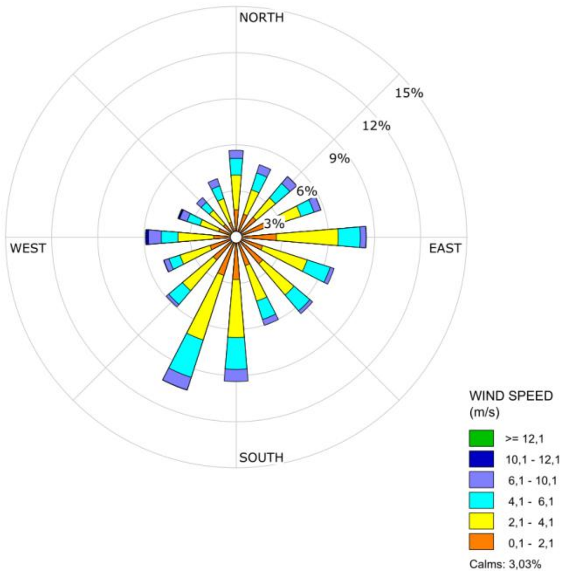
Helsinki-Vantaa airport



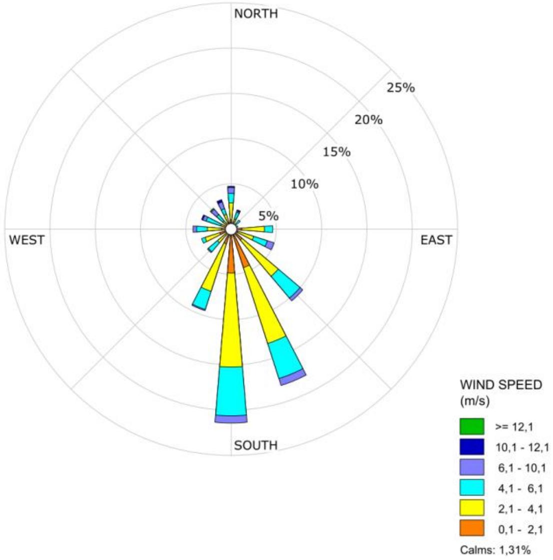


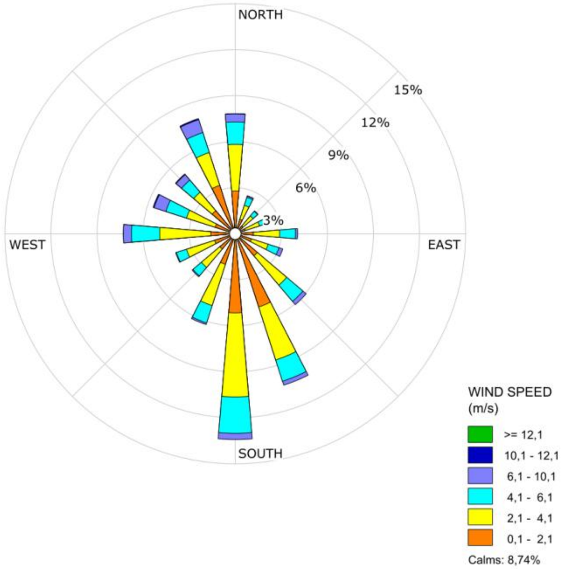
Turku airport



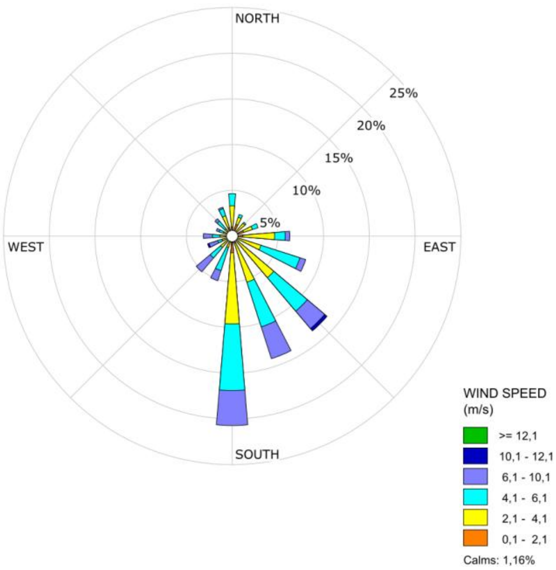


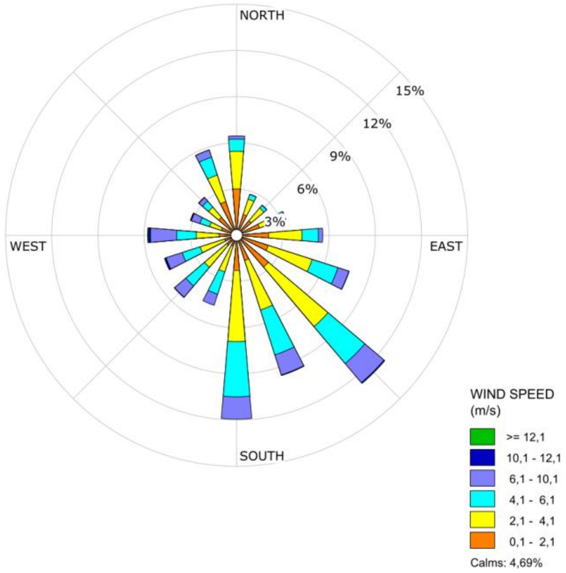
Jyväskylä airport



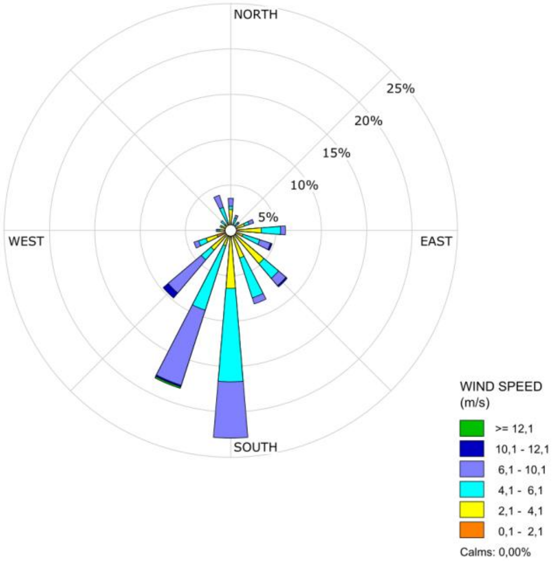


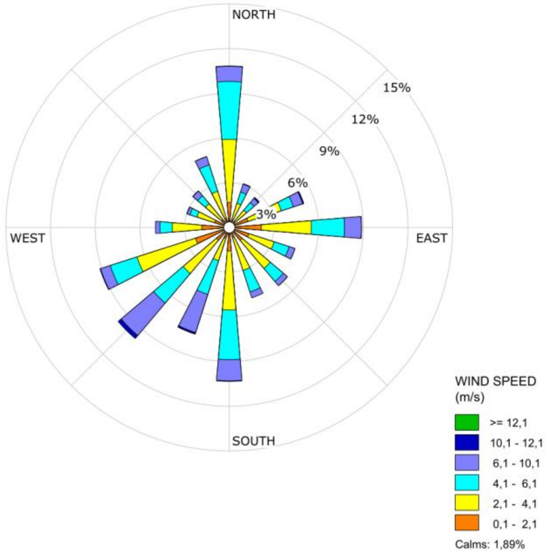
Oulu airport





Rovaniemi airport





Appendix 9 Annual freeze-thaw cycles during 09/1961 and 04/2006

Helsinki Kaisaniemi

		Number of freeze-thaw cycles			
		Lowest temperature and outdoor circumstances			
		All	**Rain ≤ 3 days before freezing**		
Sept. 1st	April 30th	**< 0 °C**	**≤ -2 °C**	**≤ -5 °C**	**≤ -10 °C**
1961	1962	64	24	15	7
1962	1963	54	20	14	7
1963	1964	91	22	14	7
1964	1965	69	20	16	6
1965	1966	77	24	18	12
1966	1967	69	25	20	6
1967	1968	42	23	15	8
1968	1969	79	23	18	10
1969	1970	70	19	13	5
1970	1971	86	26	17	6
1971	1972	88	30	19	9
1972	1973	81	26	16	1
1973	1974	122	31	23	7
1974	1975	100	16	10	4
1975	1976	72	24	15	6
1976	1977	70	25	15	5
1977	1978	59	23	16	5
1978	1979	70	13	6	2
1979	1980	43	17	6	5
1980	1981	73	25	18	10
1981	1982	91	22	11	4
1982	1983	65	17	11	4
1983	1984	71	27	16	8
1984	1985	78	18	11	4
1985	1986	55	11	8	4
1986	1987	63	15	10	5
1987	1988	67	17	14	7
1988	1989	82	24	14	6
1989	1990	67	21	12	3
1990	1991	103	23	14	4
1991	1992	95	34	18	3
1992	1993	94	24	14	2
1993	1994	77	20	12	4
1994	1995	100	26	10	2
1995	1996	79	12	7	4
1996	1997	79	23	11	4
1997	1998	99	26	14	8
1998	1999	72	22	12	9
1999	2000	88	24	17	5
2000	2001	70	13	7	4
2001	2002	96	28	18	8
2002	2003	79	24	16	10
2003	2004	89	30	19	13
2004	2005	97	19	11	4
2005	2006	67	26	18	8

Helsinki-Vantaa airport

		Number of freeze-thaw cycles			
		Lowest temperature and outdoor circumstances			
		All	**Rain ≤ 3 days before freezing**		
Sept. 1st	April 30th	**< 0 °C**	**≤ -2 °C**	**≤ -5 °C**	**≤ -10 °C**
1961	1962	82	19	14	6
1962	1963	58	15	9	5
1963	1964	112	22	14	5
1964	1965	83	13	8	3
1965	1966	81	26	15	7
1966	1967	114	22	11	4
1967	1968	68	16	15	8
1968	1969	81	23	14	8
1969	1970	73	14	8	4
1970	1971	105	28	18	7
1971	1972	98	24	14	8
1972	1973	95	26	14	3
1973	1974	138	31	20	6
1974	1975	119	16	8	2
1975	1976	101	28	19	7
1976	1977	89	18	11	4
1977	1978	102	22	12	6
1978	1979	107	21	9	3
1979	1980	68	15	8	4
1980	1981	102	27	21	9
1981	1982	97	18	9	6
1982	1983	98	21	10	4
1983	1984	75	25	17	8
1984	1985	73	15	7	2
1985	1986	51	8	4	3
1986	1987	76	15	10	4
1987	1988	75	19	14	7
1988	1989	106	28	16	10
1989	1990	90	26	12	7
1990	1991	112	29	16	6
1991	1992	111	27	16	8
1992	1993	110	29	18	4
1993	1994	79	22	14	4
1994	1995	111	29	16	4
1995	1996	85	15	12	5
1996	1997	96	24	11	9
1997	1998	82	28	20	12
1998	1999	77	26	15	10
1999	2000	82	24	15	7
2000	2001	75	19	12	7
2001	2002	94	33	26	10
2002	2003	82	15	13	6
2003	2004	94	22	16	9
2004	2005	91	21	14	6
2005	2006	65	26	18	9

Turku airport

		Number of freeze-thaw cycles			
		Lowest temperature and outdoor circumstances			
		All	Rain ≤ 3 days before freezing		
Sept. 1st	April 30th	**< 0 °C**	**≤ -2 °C**	**≤ -5 °C**	**≤ -10 °C**
1961	1962	72	25	15	5
1962	1963	62	19	11	5
1963	1964	82	22	16	10
1964	1965	68	22	16	9
1965	1966	57	21	15	10
1966	1967	104	31	24	9
1967	1968	65	22	20	10
1968	1969	79	30	18	12
1969	1970	78	22	11	4
1970	1971	89	22	19	11
1971	1972	100	33	25	12
1972	1973	125	31	21	8
1973	1974	128	22	16	4
1974	1975	117	16	8	3
1975	1976	73	21	15	11
1976	1977	82	18	11	6
1977	1978	81	19	9	4
1978	1979	106	20	13	2
1979	1980	96	19	11	3
1980	1981	89	20	18	12
1981	1982	104	18	11	4
1982	1983	132	28	18	7
1983	1984	75	16	11	6
1984	1985	89	18	8	4
1985	1986	68	15	12	5
1986	1987	84	19	16	4
1987	1988	78	18	12	4
1988	1989	114	29	21	13
1989	1990	90	29	14	8
1990	1991	97	29	21	15
1991	1992	114	32	23	10
1992	1993	95	29	23	11
1993	1994	89	26	19	6
1994	1995	125	29	20	9
1995	1996	91	25	17	9
1996	1997	97	29	22	14
1997	1998	92	24	18	12
1998	1999	69	21	16	6
1999	2000	114	24	21	11
2000	2001	91	18	12	5
2001	2002	106	24	20	10
2002	2003	112	33	23	12
2003	2004	105	24	17	14
2004	2005	98	25	16	9
2005	2006	69	22	17	11

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