Performance vs. Prescriptive Durability Requirements

A Case Study

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Abstract
In 2005, a new city development in the harbor region of Oslo City began. This development project includes a number of concrete substructures in seawater with depths of up to 20 m, on top of which a number of business and apartment buildings have been built. All the concrete substructures were finished by 2010, most of which include large underwater parking areas in up to four levels. For all these concrete substructures, the owner and developer of the project required a service life of 300 years. For all the substructures in the first four parts of the project which were produced by one contractor, the performance-based durability requirements were based on a probability-based durability design. For all the other substructures in the last four parts of the project which were produced by another contractor, the prescriptive durability requirements were based on the current concrete codes for a 100-year service life with some additional durability requirements. For all the concrete substructures throughout the whole project, the developer required a performance-based concrete quality control with documentation of achieved construction quality; this project gave a unique opportunity, therefore, to compare the results and experience obtained by use of performance vs. prescriptive durability requirements.

Key Words: Concrete infrastructure; seawater; durability; probability-based durability design; performance-based durability requirements; prescriptive-based durability requirements; performance-based concrete quality control; achieved construction quality.

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1. INTRODUCTION

In 2005, a new city development on Tjuvholmen in the harbor region of Oslo City began. This development project includes a number of concrete substructures in seawater with depths of up to 20 m, on top of which a number of business and apartment buildings have been built (Figure 1). All the concrete substructures were finished by 2010, most of which include large underwater parking areas. In the shallower water, the structures include a solid concrete bottom slab on the sea bed, surrounded by external concrete walls partly protected by riprap or wooden cladding and partly freely exposed to the tides. In the deeper water, some structures include an open concrete deck on columns of driven steel pipes filled with concrete. In the deepest water, four large concrete caissons were prefabricated in dry dock, moved into position and submerged in water up to 20 m deep. Three of these structures provide up to four levels of submerged parking (Figure 2).

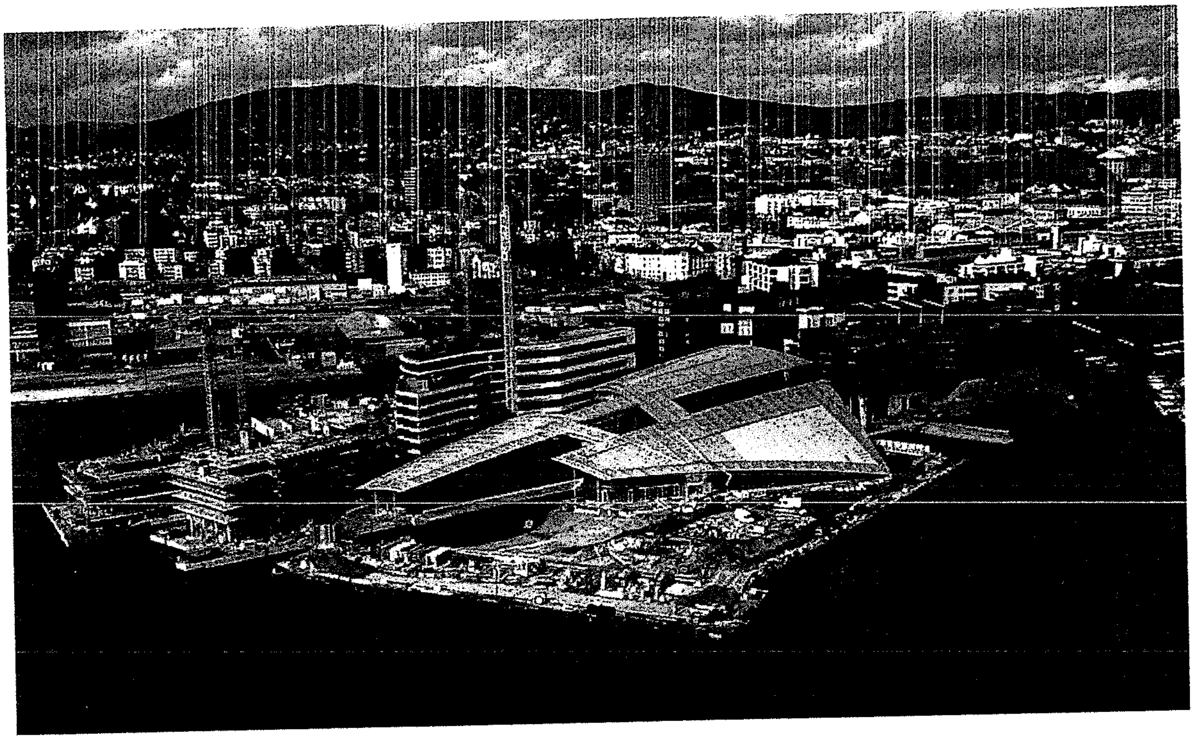

Figure 1. The new city development on Tjuvholmen in the harbor region of Oslo City.
(Photo: Terje Løchen)

For all the concrete substructures, the owner and developer of the project required a service life of 300 years, which means that a highest possible durability and long-term performance of the given structures were required. As minimum durability requirements to all structures, all requirements in the current concrete codes for a 100-year service life had to be fulfilled [1-3]. In order to obtain an increased durability of all the concrete substructures in the

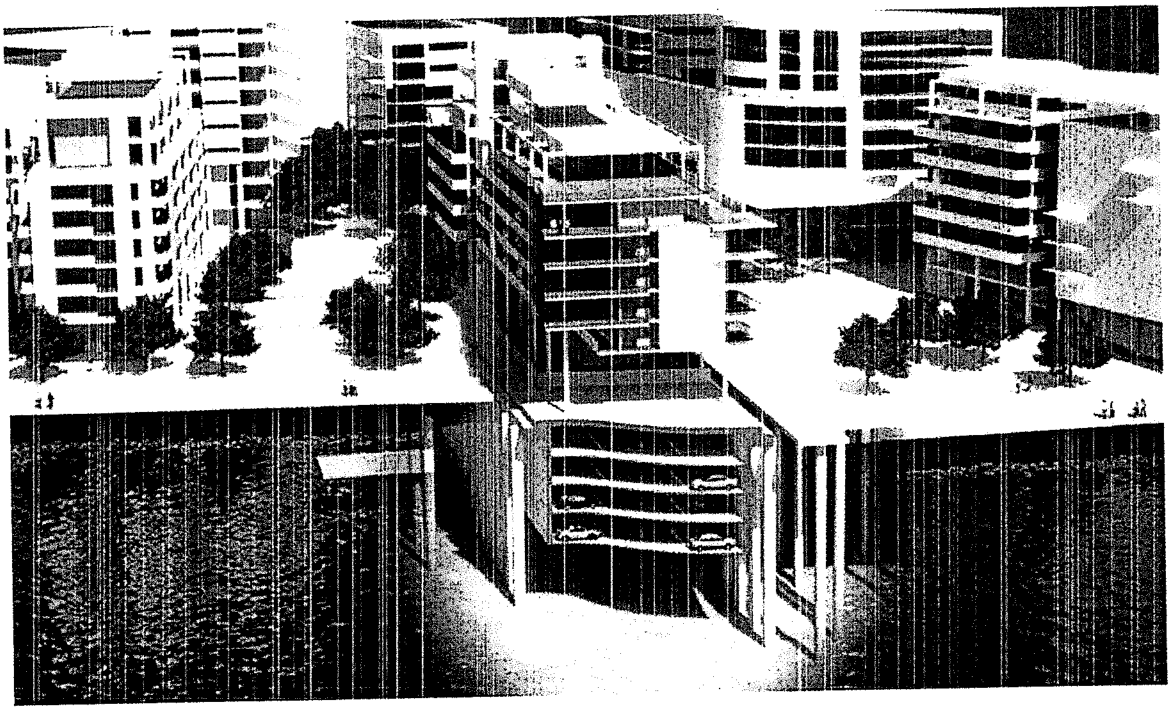

Figure 2. Section showing how the large, prefabricated concrete caissons after installation provide up to four levels of submerged parking.

first four parts of the project (Part A), a probability-based durability design with performance-based durability requirements according to the recommendations and guidelines from the Norwegian Association of Harbor Engineers was carried out [4,5]. All these structures which were produced by one contractor, mainly included solid concrete bottom slabs on the sea bed surrounded by external concrete walls in the tidal zone. All the other concrete substructures in the last four parts of the project (Part B) were produced by another contractor, and these structures mainly included four large caissons prefabricated in dry dock at two different construction sites. In addition, a number of open concrete decks were produced, partly as prefabricated elements but mostly in situ cast. For all these concrete structures, the contracts were primarily based on prescriptive durability requirements for a 100-year service life according to current concrete codes with some additional durability requirements. For all the concrete substructures throughout the whole project, the owner and developer also required a performance-based concrete quality control with documentation of achieved construction quality, also according to the above recommendations and guidelines from the Norwegian Association of Harbor Engineers. This provided a unique opportunity to compare the achieved construction quality of the various concrete structures produced with performance- vs. prescriptive-based durability requirements, respectively.

2. SPECIFIED DURABILITY

2.1 Part A: Performance-based durability requirements

Since the current procedures for probability-based durability design according to the Norwegian Association of Harbor Engineers are not considered valid for a service period of more than 150 years, the overall durability requirement was based on a probability of corrosion as low as possible and not exceeding 10% after a service period of 150 years. In order to further ensure the durability of the structures, some additional protective measures were also applied, which for the first structure in Part A.1 of the project included provisions for future cathodic prevention in combination with embedded instrumentation for chloride control. For the second structure of Part A.1 and for all the further structures in the first four parts of the project (Parts A.2-4), a partial use of stainless steel reinforcement (W.1.4301) was applied.

As a basis for selecting a proper combination of concrete quality and concrete cover which would meet the above durability requirement, an initial durability analysis was carried out based on current experience with the chloride diffusivity of different types of concrete [6]. On this basis, a concrete with 70% blast-furnace slag cement in combination with 10% silica fume typically giving a 28-day chloride diffusivity (D_{28}) of 2.0 x 10^{-12} m^2/s was adopted. A nominal concrete cover of 100 ± 10 mm was also adopted, while all the other input parameters needed for the durability analysis were based on current experience for a typical Norwegian marine environment [6]. As a result, a probability for corrosion of less than 0.3% after a service period of 150 years for the most exposed parts of the structures would be obtained. Therefore, the above values both for the 28-day chloride diffusivity and the nominal concrete cover were adopted as intended values for the first concrete substructure. A proper frost resistance of the concrete was also required, and in order to reduce the risk for early age cracking of the 100 mm concrete cover, a proper dosage of synthetic fibres to the concrete was also applied.

While provisions for future cathodic prevention were required for all exposed walls in the first concrete substructure, no additional protective measure for the continuously submerged bottom slab was considered necessary due to the very low oxygen availability in this part of the structure.

For the second structure in Part A.1 which consisted of an open concrete deck on columns of driven steel pipes filled with concrete, an additional protective measure based on partial use of stainless steel (W 1.4301) was applied. Since this protective measure showed to be a much simpler and more robust technical solution, a partial use of stainless steel was applied for all further concrete substructures in first four parts of the project (Parts A.2-4).

When the black steel was replaced by stainless steel in the outer layer of the rebar system, the effective concrete cover to the black steel further in increased to more than 150 mm. As a consequence, the nominal concrete cover to the stainless steel was reduced to 85 ± 10 mm. Hence, any addition of fibres to the concrete for these parts of the structures was no longer considered necessary. For all the solid bottom slabs, however, both black steel with a nominal concrete cover of 100 ± 10 mm and concrete with synthetic fibres were still applied.

2.2 Part B: Prescriptive-based durability requirements

For all the concrete substructures in the last four parts of the project (Parts B.1-4), the durability requirements were primarily based on current concrete codes for a 100-year service life [1-3], including a water/binder ratio ≤ 0.40 and a minimum binder content of 330 kg/m^3. This also included nominal concrete covers for the permanently submerged parts and the tidal and splash zones of 60 and 70 mm, respectively. In order to increase the durability, the nominal concrete cover for the permanently submerged slabs of the caissons was increased from 60 to 80 mm, while for all external walls with tidal and splash exposure the nominal cover was increased from 70 to 100 mm. For the submerged parts of the structures, cathodic prevention in the form of sacrificial anodes was also applied, while above water, only provisions for a future installation of cathodic prevention in combination with embedded instrumentation for chloride control were carried out.

3. CONCRETE QUALITY CONTROL

As a basis for the performance-based concrete quality control, an on-going control of both the chloride diffusivity of the concrete and the concrete cover was carried out throughout concrete construction. For the testing of chloride diffusivity, several test methods exist, but as a basis for the current testing, the Rapid Chloride Migration (RCM) method [7] was applied. Since this is the only test method which can determine the chloride diffusivity in a rapid way, independent of concrete age, this test method has been adopted for quality control in the recommendations from the Norwegian Association of Harbor Engineers [5].

For all the concrete structures for which the probability-based durability design was carried out, a 28-day chloride diffusivity of $\leq 2.0 \times 10^{-12}$ m^2/s had been adopted, while for all the other concrete structures which were only based on prescriptive durability requirements, the 28-day chloride diffusivity of the concrete had to be determined before concrete construction started.

Although the RCM method is a very rapid test method which provides data on the chloride diffusivity already within a few days, this is not good enough for the regular quality control during concrete construction. Based on a calibration curve relating the chloride diffusivity and the electrical resistivity of the given concrete, however, the chloride diffusivity was indirectly controlled by a regular non-destructive testing of the electrical resistivity during concrete construction. Therefore, calibration curves relating the two tests had to be established for all the various types of concrete before concrete construction started. Measurements of the electrical resistivity were then carried out as a quick check on the same concrete specimens as that being used for the regular quality control of the 28-day compressive strength. All the control measurements of the electrical resistivity were then carried out by use of the four-electrode method (Wenner) [8], and the testing was carried out immediately before compressive strength testing.

Since the specified concrete covers were very thick and the reinforcement was mostly also highly congested, it was very difficult to control the cover thickness accurately based on conventional cover meters. The use of stainless steel reinforcement further complicated such control measurements, although cover meters based on pulse induction could then have been used. More sophisticated scanning equipment for control of thick concrete covers does also exist [9], but a more pragmatic approach based on manual readings of the cover thickness on protruding bars in the construction joints during concrete construction was applied. If the amount of such control measurements is sufficiently high to produce reliable statistical data, such a simple approach is considered sufficiently accurate for the regular control and quality assurance during concrete construction.

4.0 ACHIEVED CONSTRUCTION QUALITY

4.1 General

Upon completion of the concrete construction work for each structure, all data from the regular concrete quality control were used as new input parameters to durability analyses for documenting the achieved construction quality. Since the 28-day chloride diffusivity was only tested on small and separately produced concrete specimens cured in the laboratory for 28 days, this chloride diffusivity may be quite different from that obtained on the construction site. Therefore, additional documentation on achieved chloride diffusivity on the construction site during the construction period was required. Since neither the 28-day chloride diffusivity from the laboratory nor the achieved chloride diffusivity from the construction site reflects the potential chloride diffusivity of the given concrete, documentation of the long-term diffusivity of the various types of concrete was also required.

4.2 Compliance with specified durability

For all the concrete substructures in the first four parts of the project (Parts A.1-4), a probability of corrosion as low as possible and not exceeding 10% for a 150-year service period was required. To show compliance with such a durability requirement, a new durability analysis was carried out upon completion of each concrete structure with input parameters based on the achieved average values and standard deviations of both the 28-day chloride diffusivity and the concrete cover. For these durability analyses, all the other previously assumed input parameters which had partly been difficult to select, were kept the same. Hence, such documentation primarily reflects the results obtained from the regular quality control during concrete construction including the scatter and variability involved.

For the first concrete substructure of the project (Part A.1), a type of concrete was applied which was somewhat retarded compared to that of the intended type of concrete. Therefore, the achieved values for the 28-day chloride diffusivity of 3.0 and 5.0 x 10^{-12} m^2/s for the bottom slab and the external walls, respectively, were higher than the intended upper value of 2.0 x 10^{-12} m^2/s. Since the concrete in question showed a very rapid further reduction of chloride diffusivity, however, this concrete was still accepted.

For all the external walls in the first concrete structure where a nominal concrete cover of 100 mm was specified, an average concrete cover of 102 mm with a standard deviation of 8 mm was achieved. For one section of these walls, however, the quality control revealed a distinct deviation. For this particular section, an average cover of only 74 mm with a standard deviation of 8 mm were observed, and as a consequence, the contractor had to apply a special surface coating later on at his own costs. For this first concrete structure as a whole, however, probabilities of corrosion for the bottom slab and the external walls of 0.24 and 2.1%, respectively, were obtained (Table 1). For the open concrete deck in the second structure of Part A.1 where stainless steel was applied, a corrosion probability of 0.13% was obtained.

Table 1. Obtained probabilities of corrosion (%) based on regular control measurements of the 28-day chloride diffusivity and concrete cover.

Part A	Bottom slab	External walls	Open deck
A.1	0.24	2.1	0.13
A.2	0.92	0.02	-
A.3	0.64	0.002	-
A.4	0.01	<0.001	-

For all the other concrete substructures in Part A of the project, the 28-day chloride diffusivity typically varied from 2.0 to 4.1 x 10^{-12} m^2/s, which in combination with the achieved concrete cover gave probabilities of corrosion in the bottom slabs varying from 0.01 to 0.92%, and from less than 0.001 to 0.02% in the external walls, respectively. Thus, the results in Table 1 demonstrate that for all the concrete structures in Part A of the project, the specified durability was obtained with a very good margin.

For all the further concrete substructures in Part B of the project which were only based on prescriptive durability requirements, it was not possible to provide any documentation of compliance with the specified durability. In order to produce documentation of achieved construction quality also for these structures, however, durability analyses of the obtained corrosion probabilities after 150 years were carried out based on the achieved average values and standard deviations of both the 28-day chloride diffusivity and the concrete cover (Table 2). For the concrete structure in Part B.1, corrosion probabilities in the bottom slab and external walls of typically 15 and 3% were obtained, respectively, while for the open concrete decks, the obtained probability was about 6%. For the concrete structure in Part B.2, no control measurements for the bottom slab were carried out, but for the external walls of this structure, corrosion probabilities typically varying from 11 to 13% were obtained. For the bottom slab and external walls of the structures in Part B.3, probabilities of about 14 and 1.3% were obtained, respectively, while for the open concrete decks in Part B.4, the probability was about 4.5%.

Table 2. Obtained probabilities of corrosion (%) based on regular control measurements of the 28-day chloride diffusivity and concrete cover.

Part B	Bottom slab	External walls	Open deck
B.1	15	3	6
B.2	-	11-13	-
B.3	14	1.3	-
B.4	-	-	4.5

The generally higher corrosion probabilities obtained for all the concrete substructures in Part B of the project (Table 2) compared to that in Part A (Table 1) may be ascribed due to several reasons. For all the concrete structures in Part A the concrete was based on blast-furnace slag cement with 70% slag (CEM III/B 42.5 LH HS) in combination with 10% silica fume, while all the concrete structures in Part B were produced with concrete based on fly ash cements in combination with 5% silica fume. For most of these structures, a fly ash cement with 30% fly ash (CEM II/B-V 32.5 N) was applied, but partly also a fly ash cement with 20% fly ash (CEM II/A-V 42.5 N). It is well known that blast-furnace slag cements generally show a very rapid reduction of chloride diffusivity by time even at low curing temperatures, while fly ash cements generally show a very slow development of chloride diffusivity, in particular at low curing temperatures [6]. For all the external walls in Parts A.2 to A.4 of the project, stainless steel was also applied, while the much higher probabilities for the bottom slabs in Table 2 compared to that in Table 1 primarily reflect the different applied concrete covers of 80 and 100 mm, respectively.

Although the mixture composition of the various types of concrete applied in Part B of the project was basically the same, the obtained 28-day chloride diffusivities at the different construction sites were quite different from one construction site to another. Thus, for one of the construction sites, it typically varied from 6.4 to 8.9 $m^2/s \cdot 10^{-12}$, while for the other, it typically varied from 12.1 to 16.7 $m^2/s \cdot 10^{-12}$.

4.3 In situ quality

For documenting the achieved in situ quality during the construction period, a number of concrete cores were removed from each concrete structure under construction and tested for chloride diffusivity. In addition, a number of concrete cores from the corresponding dummy elements were also tested. Based on the achieved chloride diffusivities after one year of site curing combined with the achieved control data on concrete cover as new input parameters, new durability analyses were carried out for each concrete structure. Also here, all the other previously assumed input parameters to the analyses were kept the same. The typical values of achieved in situ quality expressed as corrosion probability after one year are shown in Table 3.

Table 3. Obtained probabilities of corrosion (%) based on in-place data during one year on the construction site.

Part A + B	Bottom slab	External walls	Open deck
A.1	<0.001	<0.001	0.02
A.2	<0.001	<0.001	-
A.3	<0.001	<0.001	-
A.4	<0.001	<0.001	-
B.1	70	25	35
B.2	-	30	-
B.3	20	0.6	-
B.4	-	--	1.2

For all the concrete substructures in Part A of the project, very low corrosion probabilities were obtained compared to that in Part B. For both the bottom slabs and external walls of the concrete structures in Part A, the corrosion probability was typically less than 0.001% and hardly detectable, while for the concrete substructures in Part B, the corrosion probability for the bottom slabs and external walls varied from 20 to 70% and from 0.6 to 30%, respectively. Also for the open concrete decks, a high variation in corrosion probability was obtained. The generally slow development of chloride diffusivity for concrete based on fly ash cements has already been pointed out. In particular, this is true for low curing temperatures during the winter concreting. For marine construction work, this may have some implications for an early age exposure of the concrete to seawater before the concrete has gained sufficient maturity and density [6].

For the concrete structure in Part B.2, it should be noted that the site data on achieved chloride diffusivity were only based on concrete cores from the separately produced dummy element. Thus, the obtained probability of 30% for the external walls is not very representative for this particular concrete structure. For one of the external walls of this structure, a severe segregation of the self-consolidating concrete during concrete construction took place. Separate investigations based on extensive concrete coring of this wall were later on carried out showing that the in situ strength of the concrete was still acceptable, but the durability properties were distinctly reduced.

4.4 Potential quality

For most binder systems, the development of chloride diffusivity tends to plateau after about one year of water curing at 20^0C in the laboratory. In order to provide information on the potential construction quality of the various structures, therefore, the chloride diffusivity was also determined on a number of separately produced and water-cured specimens in the laboratory for up to one year. These chloride diffusivities combined with the achieved site data on concrete cover were used as new input parameters to further durability analyses. Also here, all the other originally assumed input parameters to the durability analyses were kept the same. Typically obtained values of the potential quality of the various concrete structures expressed as corrosion probabilities are shown in Table 4.

Table 4. Obtained corrosion of corrosion (%) based on laboratory-produced specimens water cured in the laboratory for one year.

Part A + B	Bottom slab	External walls	Open deck
A.1	<0.001	<0.001	0.002
A.2	<0.001	<0.001	-
A.3	<0.001	<0.001	-
A.4	<0.001	<0.001	-
B.1	0.04	0.01	0.01
B.2	-	0.05	-
B.3	0.5	0.01	-
B.4	-	-	0.5

Both for the bottom slabs and external walls of the concrete substructures in Part A of the project, the corrosion probability was typically less than 0.001% and hardly detectable. For all these structures, therefore, the potential construction quality was extremely good. Also for the bottom slabs and external walls in all the other concrete structures of the project (Part B), the corrosion probability was generally very low typically varying from 0.04 to 0.5% and from 0.01 to 0.05%, respectively, while for the open concrete decks, it typically varied from 0.01 to 0.5%. Therefore, the results in Table 4 also show a generally good potential quality of all the concrete substructures in Parts B of the project although higher values for the corrosion probability were observed. These results demonstrate, that also the concrete based on fly ash cements reached very low chloride diffusivities after one year of water curing in the laboratory.

5.0 ADDITIONAL PROTECTIVE MEASURES

For all external walls of the first concrete substructure in Part A of the project, the additional protective measure included provisions for future installation of cathodic prevention. Since such a protective measure requires that the ohmic resistance between any two points in the rebar system does not exceed 1 Ohm [10], a special quality assurance program was carried out showing that the electrical continuity was properly achieved. For this particular structure, the requirements also included instrumentation for future chloride control based on embedded probes in one selected location of the structure. For all the other concrete substructures in Part A of the project, the additional protective measure included a partial use of stainless steel (W.1.4301).

Since all the four prefabricated caissons produced in Part B of the project were installed on structural steel tubes and elements which were cathodically protected, these cathodic protection systems were designed in such a way that they also cathodically protected the submerged parts of the installed concrete structures. Also for these concrete structures, the quality control showed a proper electrical continuity within the rebar system. For those parts of the concrete structures which were located above water, however, only provisions for future installation of cathodic prevention in combination with embedded probes for chloride control in one location of the structures were applied.

For all concrete structures in marine environments, extensive experience demonstrates that the risk for corrosion is primarily related to those parts of the structures which are located above water [6]. If these parts of the structures are cathodically protected right from the beginning, such a protective measure has shown to be very effective [11,12]. If only provisions for such a protective measure are made, however, both installation and activation of the cathodic prevention system must be implemented before the first chlorides have reached the embedded steel and corrosion starts. As a consequence, such a protective measure requires a very careful monitoring and control of the chloride ingress during operation of the structures. For all the given concrete structures, the documentation of achieved construction quality typically showed a high scatter and variability. Therefore, provisions for cathodic prevention in combination with embedded probes for chloride control in one selected location of the structures may represent a great challenge for the owner during operation of the structures.

6.0 CONCLUDING REMARKS

All procedures and methods for the above durability design and concrete quality control are described and discussed in more detail elsewhere [6,13]. It should be noted, however, that the above durability design does not provide any basis for prediction or assessment of any service life of the given structures. Beyond onset of corrosion, a very complex deteriorating process starts with many further critical stages before the final service life is reached. As soon as depassivation takes place and corrosion starts, however, the owner has got a problem, which at an early stage only represents a maintenance and cost problem but later on may also successively develop into a more difficult controllable safety problem. Both from a technical and cost point of view, therefore, efforts should be made to control the initiation period before the very complex electrochemical corrosion process starts. Such control has also shown to be a very good strategy from a sustainability point of view [6,14].

It should further be noted that the obtained "service periods" with probabilities of corrosion less than 10% should neither be considered as real service periods for the given structures. However, for all the concrete structures where the probability-based durability design was applied, the durability analyses supported an engineering judgment of the most important parameters related to the durability, including their scatter and variability. Hence, a proper basis for comparing and selecting one of several technical solutions for achieving a best possible durability was obtained. It may be argued that the durability analyses were only based on the 28-day chloride diffusivity of the concrete, while the chloride diffusivity of the various types of concrete with different binder systems developed very differently. For the current durability design, however, additional durability analyses based on chloride diffusivities obtained at later curing periods for the various binder systems were also carried out, but none significantly changed the relative basis for selecting the concrete with the best possible durability properties. As a result, the probability-based durability design provided a very good basis for specifying some performance-based durability requirements which could be verified and controlled during concrete construction.

For all the concrete structures where the probability-based durability design was carried out, the specified durability was achieved with a proper margin. For the owner and developer of the project, it was very important to receive this documentation of compliance before the structures were formally handed over from the contractor. Also, the required documentation of compliance distinctly clarified the responsibility of the contractor for the quality of the construction process. During concrete construction, any deviation from the performance-based requirements could be detected and corrected for, and the required documentation of compliance clearly resulted in improved workmanship and reduced scatter and variability of achieved construction quality.

For all the concrete substructures where the specified durability was only based on prescriptive requirements, it was not possible to provide any documentation of compliance with the specified durability. Although the mixture composition of the concrete was basically the same, the performance-based concrete quality control revealed that the achieved concrete quality varied from one construction site to another. When a severe segregation in one of the concrete structures also took place during concrete construction resulting in reduced durability properties, the owner and developer of the project had to accept this reduced construction quality. It was very difficult to argue against a contract based on prescriptive durability requirements which could not be verified and controlled during concrete construction.

In order to further ensure the required service life of the structures, some additional protective measures were also applied. For some of the structures, this included a partial use of stainless steel, while for others, either cathodic prevention or provisions for such prevention in combination with embedded probes for future chloride control were applied. For a cathodic prevention system to be effective, such a system must be implemented before the first chlorides have reached the embedded steel and corrosion starts. Since all the concrete structures typically showed a high scatter and variability of achieved construction quality, however, a very close control of the chloride ingress may represent a great challenge for the owner during operation of the structures. For all the concrete structures where a partial use of stainless steel was applied, this protective measure showed to be a much simpler and more robust technical solution. Even on a short-term basis, this solution also proved to be economic competitive with the other alternative of protective measure. On a long-term basis, additional high expenses both for installation and operation of a cathodic prevention system are involved.

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