PORT AND OCEAN ENGINEERING UNDER ARCTIC CONDITIONS
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DURABILITY OF MARINE CONCRETE STRUCTURES UNDER ARCTIC CONDITIONS

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SUMMARY

A few years ago a comprehensive investigation was carried out with the purpose of evaluating the durability of concrete structures located along the Norwegian seaboard. Under the combined effects of marine and arctic conditions these structures are exposed to a number of serious deteriorating processes, and each year large amounts of money are spent on repairs and protective maintenance.

The observations indicate that chemical deterioration of concrete in seawater may also represent a problem under arctic conditions, particularly in connection with underwater placing of concrete. By proper use of air-entraining agents action of freezing and thawing should cause minor trouble, while corrosion of reinforcing steel does represent the most extensive and serious problem.

INTRODUCTION

During the past 50 to 60 years a steadily increasing proportion of wharves and piers have been constructed in Norwegian harbors in the form of reinforced concrete decks on slender, reinforced concrete pillars poured under water (cross-sections of 80 to 100 cm square and lengths of up to 25 metres). Among the relatively large wharves constructed for heavy traffic, more wharves of this type have been built along the Norwegian seaboard than of all the other types of wharves put together. Combined with a simple system of prefabricating the wooden forms on shore and placing them with the reinforcement as complete units on foundations prepared in the sea-bed, the method has made such structures cheaper than most other types of wharves provided that the soil conditions have not been too unfavourable (Ref. 1).

A few years ago a comprehensive field investigation was carried out on 219 structures of the above type, constructed along the Norwegian seaboard during the past half century. The structures included more than 190.000 m^2 of reinforced concrete decks on more than 5.000 slender, reinforced concrete pillars poured under water.

In order to obtain a more adequate basis for evaluating the behavior of the structures inspected, samples of the concrete were removed from above as well as below water for chemical and physical analysis. In addition to the samples taken out from the structures inspected, further information about the effect of seawater was obtained from a comprehensive series of long-time tests carried out at a field station in Trondheim harbor (Ref. 2,3). At this station more than 2.500 concrete test specimens were exposed to seawater up to a period of 25 to 30 years. In order to obtain information about the heat exchange between the structures and the surrounding air, detailed temperature investigations were also carried out on two structures in Bodø harbor (Ref. 4).

On the basis of the comprehensive field investigation carried out, a very good overall condition of the structures was observed (Ref. 5,6). Even after more than 50 years of service the structures still showed a remarkable ability to withstand the combined effects of the most severe marine exposure and heavy structural loads.

It must be pointed out, however, that the condition of the individual structures varied considerably. Also, large amounts of money had been spent each year on repairs and protective maintenance of the structures. Under the combined exposure of marine and arctic conditions there are a number of serious deteriorating processes taking place, of which the most important effects will be discussed briefly in the following.

CHEMICAL ATTACK

From that part of the field investigation which was carried out under water, 4 percent of the structures were observed with one or more pillars seriously impaired, while for 87 percent, all the pillars were found to be in very good condition. Repairs had been carried out on 6 percent of the structures.

This overall good condition below water proves that concrete can be successfully placed under water even for reinforced concrete members with relatively narrow cross sections. Where defective areas were observed, the location, frequency, and appearance left no doubt that this damage primarily was the result of neglecting general know-how and recommendations for tremie placing of concrete. However, where dilution and segregation of fresh concrete had occurred due to faulty concreting, an advanced degree of chemical deterioration had also taken place, where up to 80 percent of the lime content in the paste was leached out while the magnesia content had increased up to 14 times the original amount. This means that the cohesive strength and the loading capacity of this concrete was completely broken down.

Normally, a cement content of 400 kg per m^3 of tremie concrete had been used, corresponding to a water-cement ratio of 0,50 to 0,55. Such a concrete quality had apparently been sufficient to withstand the action of seawater, as the majority of pillars were observed with a hard and smooth concrete surface even after more than 50 years of exposure. However, local dilution of the concrete during placing had sometimes raised the water-cement ratio above the critical level, and thus exposed parts of the concrete to a severe chemical attack.

It should be noted that sulfate resisting cements had not been used in any of the structures inspected. It has widely been assumed that chemical deterioration of concrete in seawater does not represent any problem in colder regions. However, both the comprehensive field investigation as well as the long-time tests carried out in Trondheim harbor clearly demonstrate that attention to this problem should also be paid under arctic conditions. This should be particularly important for a type of underwater construction where the permeability of the concrete may rise above the critical level, at least locally, during the construction.

FREEZING AND THAWING

Exept for the deterioration observed within the tidal zone damage due to freezing and thawing was mainly confined to a small extent of scaling. Even though the great majority of structures had been cast without air-entrained concrete, the confined extent of frost damage must be considered in relation to the field conditions.

While the number of days per year with freezing temperatures vary along the Norwegian seaboard from 40 up to 200, most of the coastline has on the average less than 10 days with air temperatures below $-10^{o}C$, and only the northernmost coastline and the inner parts of the longest fjords reach so low temperatures for more than 20 days per year. Further, during the coldest part of the winter the exposure to wetting has also been rather moderate in the sheltered harbors. Thus, in these periods the moisture content in the concrete structures has frequently been below the critical limit for development of frost damage.

Since the majority of structures had been constructed before the time of air-entraining admixtures, even better conditions would have been observed, if air-entrained concrete had been more widely used. Where freezing temperatures occur in marine environments, air-entrained concrete should always be employed.

COMBINED EFFECTS IN THE TIDAL ZONE

Of the deterioration observed within the tidal zone only 14 percent of the structures had one or more pillars with reductions of cross-sectional area exceeding 30 percent, while 62 percent had only a small extent of damage or no damage at all. The rate of deterioration seemed to be very slow, and, on the average, only structures more than 35 to 40 years old had pillars with cross-sectional reductions of more than 20 percent. It should be noted, however, that it has been common practice since 1933 to leave creosote impregnated wooden forms in place as protection in the tidal zone. Further, repairs had been carried out on 26 percent of the structures.

In the chemical analysis of disintegrated concrete from the tidal zone, the same tendencies of ion-exchange between the paste and seawater was found as for the deteriorated concrete from under water, though somewhat less pronounced (Ref. 6). The salt concentrations observed in the concrete did not indicate that salt crystallization should cause any damage. However, the salt concentration at the surface of the concrete was high enough to substanitally lower the freezing point. Even at a depth of 75 to 100 mm inside sound, high quality concrete up to 6 grams of calcium dichloride per 100 grams of cement were observed. It should be noted that undiluted seawater has a freezing point of approximately -2°C, while for solutions of 2 or 5 times higher concentration, the freezing point is lowered to approximately -4 and -14°C, respectively.

From the analysis of long-time tests at the field station in Trondheim harbor, it was concluded that frost action was the most important factor in the deterioration of concrete in the tidal zone. However, a distinct correlation between the varying C_3A-content of the cements and the durability of the concrete was also observed in the same tests (Ref. 2). Thus, the deterioration in the tidal zone seems to be the result of a complex interaction of several processes taking place simultaneously.

The protective timber forms used in the tidal zone is believed to have improved the durability of the concrete in two different ways. First of all, the effect of freezing and thawing has been reduced. Further, the flow of seawater upwards through the concrete has also been reduced due to a reduction of evaporation from the concrete surface. The temperature measurements carried out in Bodø harbor clearly demonstrates that the timber form in the tidal zone provides an effective protection of the concrete against frost action (Ref. 4). For a temperature variation of 11 to 13°C outside the form, 1½" thick, a variation of no more than 1,5°C was observed on the concrete surface inside the form. Even when the air temperature was as low as -10 to -12°C, the surface temperature of the concrete never dropped below -4°C.

VOLUME CHANGE DUE TO MOISTURE MOVEMENT AND TEMPERATURE CHANGE

Due to lack of sufficient information it was generally difficult to determine whether or not cracks and fissures were due to volume changes from moisture movements and temperature changes or due to overloading from structural loads. However, relatively few cases of cracking were classified according to the former cause. In the deck slabs most of these cracks started from corners where the slab had been cast monolithically to the seawall. Such cracks represent a well known problem pertaining to all concrete structures in larger monolithic units.

Due to the generally troublesome maintenance of expansion joints in wharf decks, there has been a tendency in recent years to increase the distance between such joints. One of the largest monolithic deck units constructed in Norwegian harbors is found in Bodø. Here a flat slab deck was built in 1965 with a length of 191,5 metres without any structural joints. Under such sircumstances a correct evaluation of the effect of temperature changes on the total length of the deck is important.

For calculation of temperature stresses according to the Norwegian concrete code, assumed values for fall of temperature shall never be less than 15 to 20 and for rise never less than 15°C. However, meteorological information indicates that the difference between the highest and the lowest temperatures registered along the Norwegian seaboard varies from 50 to 60°C in the southeast to about 40°C in the southwest (Ref. 4). Northwards, the range increases up to 60°C with differences of approximately 70°C in the region around Kirkenes. From the temperature measurements carried out in Bodø harbor, extreme temperatures of approximately -20 and +30°C were observed in the middle of the deck slab, 26 cm thick, while the corresponding temperatures in the surrounding air were -17 and +25°C, respectively (Ref.4). Under certain circumstances the radition of sun could raise the middle temperature in the deck slab up to 6 to 8°C above the ambient air temperature, while cooling due to outwards radiation or evaporation correspondingly could lower the slab temperature 2 to 4°C below the air temperature.

CORROSION OF REINFORCING STEEL

Of the various breaking-down processes taking place the observations indicate that corrosion of reinforcing steel does represent the most extensive and serious problem to the durability of the structures inspected. Independant of age, environment and structural design, a varying extent of steel corrosion was observed on the deck beams in 84 percent of the structures, on the deck slab 68 percent, and on the pillars in 28 percent of the structures. The most extensive and serious corrosion was observed on the deck beams, this despite the fact that the most extensive repairs also had been carried out on these members. Damage due to steel corrosion was only observed above spring high water level. However, this does not exclude that a certain extent of steel corrosion also took place in the continuously submerged or the completely water saturated parts of the structures.

As a number of the structures had repeatedly been overloaded, the question arises to what extent this had affected the deck corrosion. Even though no simple and general relation exists between crack width and corrosion of reinforcing steel, overloading had probably been a contributing factor towards the extensive beam corrosion. However, the first corrosion had frequently occurred after just a short time of service before any overloading of the structures had taken place. When searching for an explanation of why particularly the deck beams had been so much more vulnerable than the deck slabs, other circumstances therefore has to be considered. A study of the pattern of the damage due to steel corrosion as well as of the mechanism of the electrochemical corrosion, indicates that all those parts of marine concrete structures which are the most exposed to intermittent wetting and drying, also will be the most vulnerable to steel corrosion (Ref. 6).

Even before 1930 experience indicated that the deck slabs showed a higher durability than the deck beams. The practical consequence of this was drawn i 1932, when the first concrete wharf of the flat-slab type was introduced in Norwegian wharf construction. From then on the so-called "flat-slab wharf" was used to a considerable extent along the entire coast. However, as the flat slab is not always the cheapest type of deck construction, there has been a tendency in recent years to return to the beam-and-slab deck. It has been assumed that a deck of the beam-and-slab type may last as long as a flat-and-slab type, provided that the beams are formed in such a way that the best quality of concreting of the bottom part of the beams can be assured.

Certainly, a concrete cover of high quality is essential to inhibit steel corrosion in the deck beams. However, for all the structures inspected built after 1940, there was no significant difference in the extent of irregularities due to faulty concreting neither between those beams having deep, rectangular cross-sections and those beams having more shallow, trapezoidal cross-sections, nor between the deck beams as a group compared to the deck slabs. As a matter of fact a thicker concrete cover over the steel in beams than in slabs was normally observed. In addition to the fact that deck beams always are more subjected to cracking from overloading than deck slabs, all beams projecting from the underside of a deck slab just above seawater level will be more exposed to intermittent wetting and drying than the sheltered slab sections inbetween. Therefore, all deck beams will promote differences in salt and moisture content within the deck and thus tend to develop a characteristic pattern of anodic and cathodic areas. Since in this pattern the more exposed deck beams will tend to be anodic in relation to the deck slab, all deck beams should be made as shallow as possible or preferably be avoided in all such structures.

In the Norwegian concrete code different thickness of concrete cover is recommended for different types of structural components such as pillars, beams, slabs and walls. For moist concrete having low electrical resistivity, there are certain combinations of impairment of the steel passivity and accessibility of oxygen which is decisive for development of steel corrosion. If varying thickness of cover is used over reinforcing steel being in electrical contact, this may result in different access to oxygen which in turn may promote formation of galvanic cell action. Hence, for a type of exposure which represent a particular danger to steel corrosion, all steel being in electrical contact should preferably have the same thickness of concrete cover.

In the litterature a number of different recommendations are given for adequate thickness of concrete cover. However, detrimental amounts of chloride-ions were found to have penetrated into high quality concrete beyond what would be a practical limit for the thickness of a concrete cover. Hence, attention should be paid to additional precautions for inhibiting steel corrosion when concrete structures are exposed to severe marine environments.

The observations further indicate that repairs of damage due to steel corrosion should never be considered as being just a question of recovering the steel where the concrete cover has spalled off. The electrolytical conditions within a concrete structure will always be more and less influenced and changed by any application of new concrete to the structure or by any application of a coating to the concrete surface. To obtain a more effective control of a deterioration due to steel corrosion, all consequences of changed electrolytical conditions should therefore be considered before any repairs or any coatings of the structures are carried out.

CONCLUSIONS

On the basis of the comprehensive field investigation carried out along the Norwegian seaboard, the following conclusions appear to be warranted:

1. Chemical deterioration of concrete in seawater may also represent a problem under arctic conditions, depending on the C_3A-content of the cement and the permeability of the concrete. For ordinary portland cements with high C_3A-contents, a water-cement ratio of not more than 0,50 should be used.

2. For underwater placing of concrete where the permeability of the concrete may rise locally during the construction, a sulfate resisting cement should always be employed.

3. In the deterioration taking place in the tidal zone freezing and thawing is supposed to be the most important mechanism. However, the observations indicate that chemical deterioration also plays an important role in this breaking-down process.

4. Apart from the deterioration observed in the tidal zone, damage due to freezing and thawing was mainly confined to a small extent of scaling. However, even better conditions would have been observed if air-entrained concrete had been more widely used.

5. Corrosion of reinforcing steel represents the most extensive and serious problem to the structures inspected. Damage was only observed above spring high water level, but this does not exclude that a certain extent of steel corrosion also took place in the submerged parts of the structures.

6. A study of the pattern of the damage due to steel corrosion as well as of the mechanism of the electrochemical corrosion, indicates that all those parts of marine concrete structures which are the most exposed to intermittent wetting and drying, also

will be the most vulnerable to steel corrosion.

7. Detrimental amounts of chloride-ions were found to have penetrated into high quality concrete beyond what would be a practical limit for the thickness of a concrete cover. Thus, attention to additional precautions for inhibiting steel corrosion should be paid when concrete structures are exposed to severe marine environments.

8. The observations further indicate that repairs of damage due to steel corrosion should never be considered as being just a question of recovering the steel where the concrete cover has spalled off. To obtain a more effective control of a deterioration due to steel corrosion, all the consequences of changed electrolytical conditions should be considered before any repairs or any coatings of the structures are carried out.

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