Session A4

Concrete Durability I
Durabilité du béton I
Dauerhaftigkeit des Betons I

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Application of Super-Workable Concrete to an Arch Bridge

Emploi d'un béton de mise en place très aisée dans un pont à arc

Verwendung eines super-verarbeitbaren Betons bei einer Bogenbrücke

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SUMMARY

An arch bridge was designed to harmonize with the surrounding landscape. Steel bars were positioned with a spacing of 100mm to 250mm in the arch rib portion and concrete with high compactability was considered necessary. Due to the problems involved in placing, and to achieve labor saving, it was decided to place the entire cross section of concrete in one operation. To ease the construction process and speed up completion time, it was necessary to use super-workable concrete for both the arch rib and the crown portions of the bridge. This report describes of the preliminary placing experiments of the super-workable concrete carried out before the construction, and the results of quality control at the construction work.

RÉSUMÉ

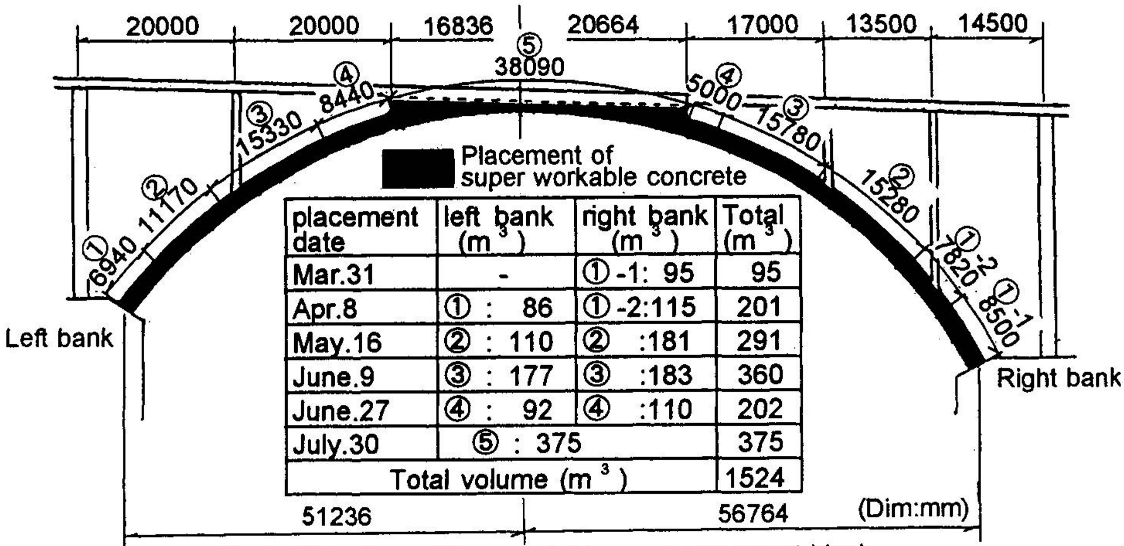
Dans le cas d'un pont en arc, devant bien s'intégrer dans son environnement, il a été pré-vu d'espacer les armatures de 10 à 25 cm et d'utiliser un béton à haute densité dans l'arc. Étant donné les problèmes techniques de mise en place et afin d'économiser des coûts de main-d'oeuvre, le bétonnage a été effectué en une seule étape. Pour accélérer la réalisation, il a été également décidé d'employer un béton de mise en place très aisée dans les nervures et à la clé de voûte du pont. L'article présente les essais préliminaires avec ce béton de mise en place facile et donne les résultats du contrôle de la qualité sur le chantier.

ZUSAMMENFASSUNG

Für eine Bogenbrücke, die mit der umgebenden Landschaft harmonieren sollte, wurden im Bogen Bewehrungseisen im Abstand von 10 bis 25 cm und ein Beton mit hoher Verdichtbarkeit vorgesehen. Wegen der Probleme beim Einbringen- und um Arbeitskosten zu sparen - wurde entschieden, den gesamten Betonquerschnitt in einem Arbeitsgang einzubringen. Dazu wurde für die Bogenrippen und den Scheitel besonders leicht verarbeitbarer Beton benötigt. Der Beitrag beschreibt die Vorversuche mit diesem "super-verarbeitbaren" Beton und die Ergebnisse der Qualitätssicherung auf der Baustelle.

1. INTRODUCTION

For super workable concrete, while having high fluidity it must have moderate stability at the same time, in addition it has self-compactability. So, it can flow the spaces between the reinforcing bars and forms without vibration.
The concrete using in this construction was super workable concrete with binary components containing blast furnace slag and it placed around 1500m^3. The two structural parts for which the super workable concrete were applied are the arch rib and crown portions shown in Figure 1. The arch rib is a triple box section of radius 63.406m, thickness 2m and width 9.4m, and due to different heights at the river banks the bridge is asymmetrical. The upper and lower slabs of the hollow arch rib are 300mm thick, and reinforcement of maximum diameter 35mm spaced at 125mm to 250mm across the width and at 100mm spacing vertically was used. From aesthetic and construction considerations it was decided to place the concrete in one operation, using the super workable concrete. The bar arrangement drawing can be seen in Figure 2.



placement date	left bank (m^3)	right bank (m^3)	Total (m^3)
Mar.31	-	①-1: 95	95
Apr.8	① : 86	①-2:115	201
May.16	② : 110	② :181	291
June.9	③ : 177	③ :183	360
June.27	④ : 92	④ :110	202
July.30	⑤ : 375		375
Total volume (m^3)			1524

Fig.1 Side view of the arch-bridge and placement block

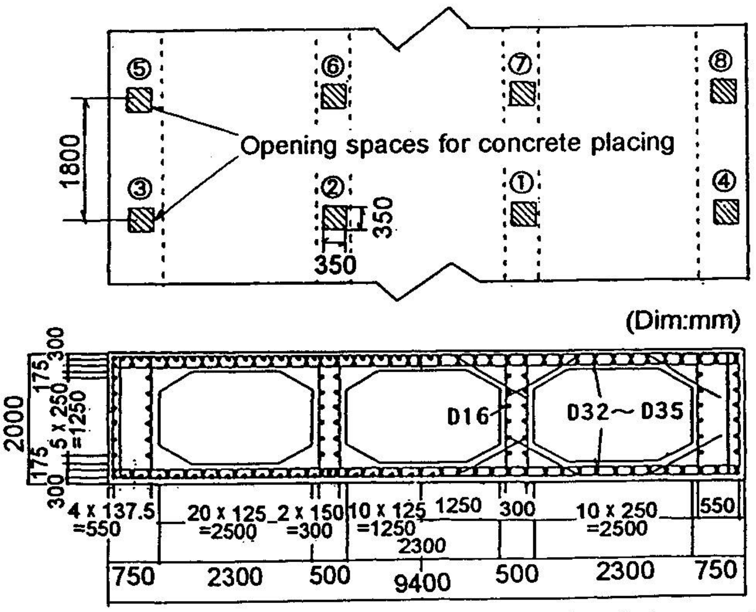


Fig.2 Bar arangement of arch-rib and order of placement

Cement (C)	Blast furnace cement B type (Blain value of blast furnace slag: 6000cm ^{2} /g, Replacement ratio :60%, Specific gravity:3.01)
Fine aggregate (S)	Sea sand (Specific gravity:2.55, F.M:2.63)
Coarse aggregate(G)	Crushed stone (Gmax:20mm, Percentage of solid volume:58.5%
Admixture	Superplasticizer(SP) and AE agent(AE)

Table 1 Materials

W/C (%)	s/a (%)	kg/m ^{3}				SP	AE
		W	C	S	G	(C X %)	(C X %)
37	50	170	461	806	825	2.0-2.4	0-0.012

Table 2 Mix proportion

*Time when slump flow reached 50cm

slump flow (cm)	* 50cm flow time (sec.)	Air (%)	V-funnel falling time (sec.)
60 ± 5	6.0 ± 3	4.5 ± 1.5	7.5 ± 2.5

Table 3 Items and value of quality control

2. CONSTRUCTION OF SUPER WORKABLE CONCRETE

2.1 Outline of the work

The materials are shown in Table 1. To improve the stability, blast furnace slag with a Blaine value of 6000cm^2/g was used as an additional binder to cement, the replacement ratio was set to 60%, the highest value for type B blast furnace cement.
The mixing proportions for the super workable concrete are shown in Table 2. For the construction work, the water content per unit volume of concrete was reduced to 5kg/m^3 less than the preliminary placing experiments to prevent the loss of stability caused by variations in surface moisture of fine aggregate. The concrete placed by pump and the outlet was positioned at each opening in turn, as shown in Figure 2, and the concrete placed in the order shown without the use of a vibrator for compaction. When the concrete level reached 40cm below the outlet the lid was placed to close the opening.
The rate of placing during the construction was about 1.5m/hr, and based on the results of lateral pressure measurements made during the preliminary placing experiments, and allowing for a margin of safety, a load of 70kPa was taken as the form design lateral pressure loading.

2.2 Quality control of super workable concrete

2.2.1 Quality control method

The values for concrete quality at the point of unloading and after pumping are given in Table 3. It was decided that in cases where the time from mixing to completion of placing exceeded 2 hours, (1.5 hours in hot weather concrete), the concrete would be disposed of, regardless of the conditions of fresh concrete. Therefore it was necessary to transport the concrete from the mixing plant to the site in 40min to 50min which meant the supply base had to be changed to suit the placing conditions. Concerning the frequency of quality testing, firstly the quality of concrete of 3 agitator trucks was confirmed immediately after shipment, then at a ratio of one in 3 to 5 trucks. If there was a considerable fluctuation in quality, then every truck would be checked until consistency of quality was restored.

2.2.2 Results of test for quality control

In Figures 3(a) to 3(e), are shown the results of quality tests carried out at shipment, unloading and after pumping.

As can be seen from Figure 3(a), the fine aggregate is a combination of two systems, one of which is used in the making of regular concrete, and the surface moisture is affected considerably by the amount and rate of shipment and results in considerable fluctuations. However, in the case of number 5 placement block (crown portion), the concrete was placed at night as countermeasure of hot weather concrete, and as a result the amount of surface moisture was relatively stable.

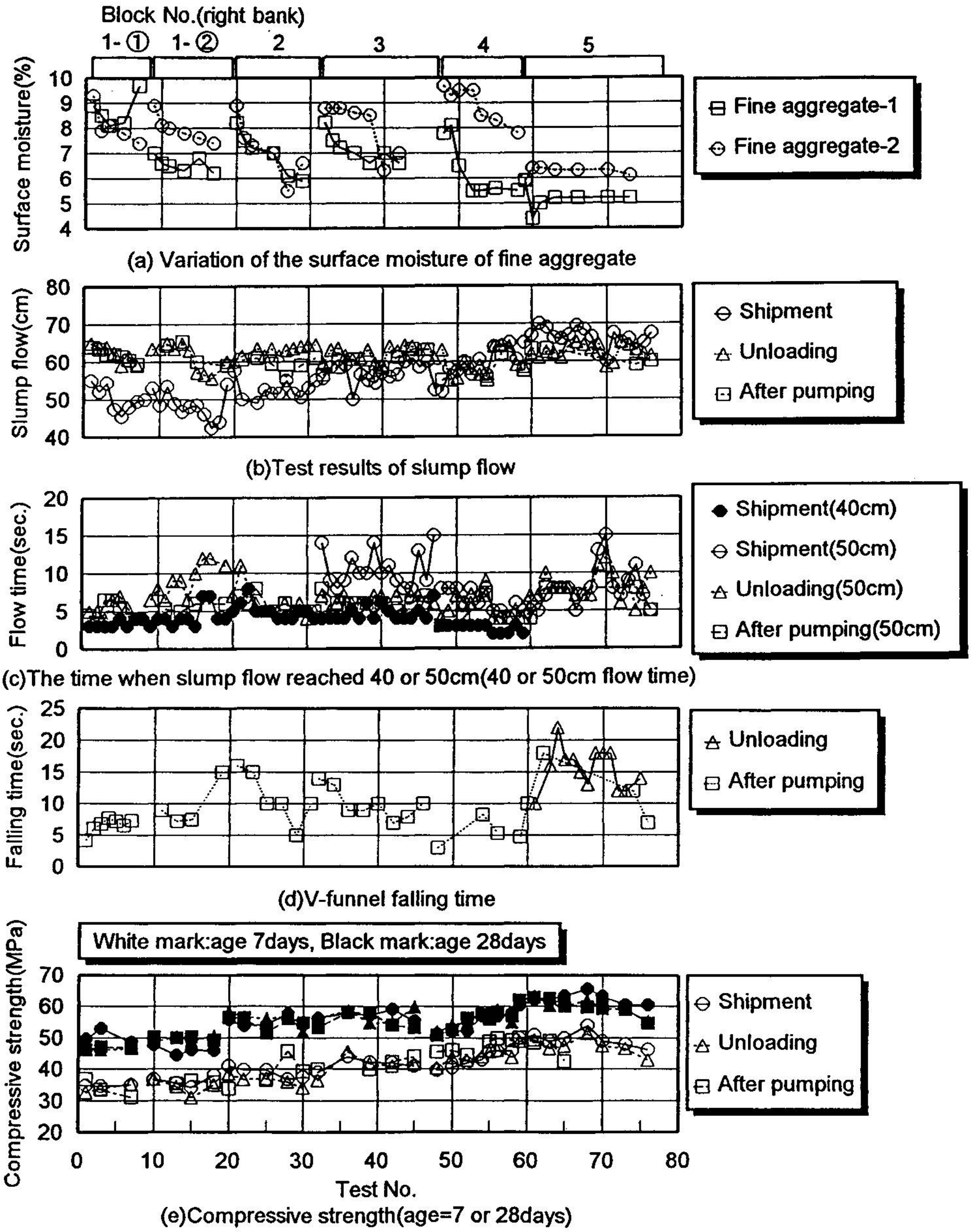


Fig.3 Results of test for quality control

The results of slump flow tests are shown in Figure 3(b). At the time of commencement of the concrete placement in March it was found that there was a tendency for the slump flow to increase by 10cm from the time of shipment to the time of unloading, which was the same as in the case of the preliminary placing experiment carried out in February. From this result, to achieve a slump flow of 60±5cm at unloading or after pumping, it was decided to make the slump flow value at shipment 50±5cm. However, due to the rise in environment temperatures (temperature when the concrete is mixed up) the increase in slump flow and the time taken to reach a peak value of slump flow became short, so to increase the value of slump flow, the amount of superplasticizer was increased. At the time of final placing (crown portion : July 30), there was virtually no difference between the slump flow value at shipment and the slump flow value at unloading or after pumping.

The time when the slump flow reached 40 or 50cm(abbreviate to "40 or 50cm flow time") is shown in Figure 3(c). For the case of a slump flow of around 55cm at shipment, 40cm flow time was measured, and during the latter half of the construction work, when the slump flow had increased to around 60cm, the 50cm flow time was measured. There was a large variation in 40 or 50cm flow time measured at the commencement of placing, at block 1, and during hot weather from the latter half of block 4 and during block 5, but was stable during the intermediary phase of the work, at shipment in 4 to 5sec.(40cm flow time), at unloading or after pumping in 5 to 7sec.(50cm flow time).

The V-funnel falling time measured at after pumping fluctuates considerably even when there is little variation in slump flow, this is seen as the effect of different viscosities of the placed concrete(see Figure 3(d)). Also, the V-funnel falling time value showed the same relative trend as the 50cm flow time after pumping, but the V-funnel falling time showed the differences most clearly. The main reason for the variations in V-funnel falling time, even though the super workable concrete is of fixed mix proportion and same slump flow, is believed to lie in the surface moisture content of fine aggregate. In such a case, it is necessary to measure the surface moisture and make fine adjustments to the set value.

In Figure 3(e), the results of compressive strength tests at 7 days and 28 days for shipment, unloading and after pumping are shown. The increase in environment temperature gave rise to difficulties in maintaining the ease of slump flow at after pumping, necessitating an increase in superplaticizer causing a corresponding gradual increase in the compressive strength.

2.3 Measurement of lateral pressures

In Figures 4 are shown the results of measurements of the lateral pressures made during the preliminary experiments, while in Figure 5 is shown the results of the measurements made during the actual construction work.

The test specimen was in the form of an unreinforced structural column 7.5m high, the concrete was placed in 5 layers allowing different interval of placement. The rate of placing in all cases was about 2.6m/hr.

In Figure 4 is shown the relationship between the time elapsed since shipment and the ratio of lateral pressure and liquid pressure on the preliminary placing experiments. It was found that the measured values showed a good correlation with the straight line of the regression equation. By setting the rate of placing in the regression equation , the maximum lateral pressure on the formwork can be obtained. Substitution of the respective values in equation (1) yields the quadratic equation (2).

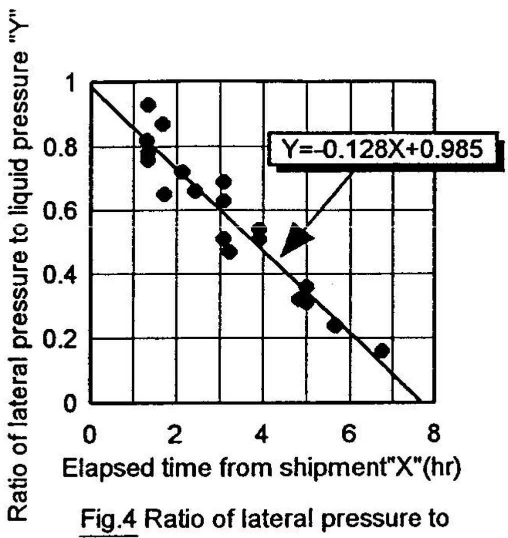
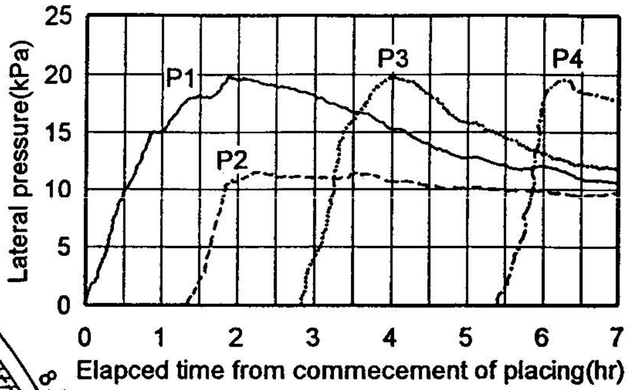
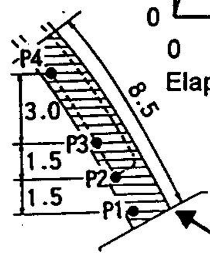


Fig.4 Ratio of lateral pressure to liquid pressure with elapsed time





Side view of arch-lib (Block No.1- ①)

Fig.5 Measurement of lateral pressure (construction work)

$$Y = Pc/Pw = -0.128X + 0.985 \quad \cdots\cdots\cdots\cdots (1)$$

$$Pc = -7.4X^2 + 63.8X - 57.0 \quad \cdots\cdots\cdots\cdots (2)$$

where:
- Pc : Concrete lateral pressure (kPa)
- Pw : $R \times Wc$ ..... liquid pressure
- R : Rate of placing 2.6(m/hr) $\times$ t
- Wc : Weight per Unit Volume of Concrete $2272 \times 9.8 (N/m^3)$
- t : Elapsed time from commencement of placing concrete = X - 1 (hr)

The lateral pressure occurring during the No.1 block placement in construction work can be seen in Figure 5. From the figure it can be seen that the pressure reaches a maximum of about 20kPa and then falls with time. The placing speed was around 1.0m/hr to 1.5m/hr, taking the average, about 1.1m/hr. Lateral pressures were measured at the placing of other blocks too, and in all cases the maximum pressure was found to be around 20kPa. In the design of the forms and falsework, considering the results of preliminary experiments and allowing for safety, a design load of 70kPa was adopted, however as the actual measured maximum was 20kPa, this was abandoned on the way and a value of 44kPa was adopted for design.

3. CONCLUSION

It was found possible to efficiently place approximately $1500m^3$ of suitably prepared super workable concrete for the arch rib and crown portion of the concrete arch bridge within the agreed completion time. By carrying out various preliminary experiments, and in particular by continuous measuring of the lateral pressures on formwork, it was possible to perform the placing successfully.

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Increasing the Durability of Concrete Exposed to Aggressive Ground Water

Durabilité accrue du béton exposé à des eaux agressives

Verlängerung der Lebensdauer von aggressivem Grundwasser ausgesetzten Beton

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SUMMARY

The Swiss Federal Railways now requires a planned service life of 100 years for its new "Rail 2000" tunnels. With normal maintenance, no basic repair work should be necessary during that period of time. These very stringent standards place equally high quality demands on the materials and the structure, together with high project design quality. For such important structures, questions of testing methods and construction materials must be answered in order to ensure satisfactory long-term quality.

RÉSUMÉ

Les Chemins de Fer Fédéraux Suisses exigent, pour les nouveaux tunnels du projet "Rail 2000", une vie de service de 100 ans sans travaux de réparation et de restauration majeurs. Cette exigence sévère présuppose une qualité élevée des matériaux utilisés, de la main d'oeuvre et du projet. En présence de projets importants, le problème du choix des méthodes d'essai appropriées et des matériaux se pose, afin de garantir la longévité exigée des ouvrages.

ZUSAMMENFASSUNG

Die Schweizerischen Bundesbahnen fordern für die neuen Tunnel der "Bahn 2000 " ein Nutzungsziel von 100 Jahren, d.h. bei sichergestelltem, üblichem Bauwerksunterhalt soll in der geforderten Zeitspanne keine grundlegende Sanierung notwendig werden. Diese sehr hohen Ansprüche setzen eine entsprechende hohe Qualität der Baustoffe und der Projektgestaltung voraus. Für bedeutende Bauwerke stellt sich somit die Frage nach der geeigneten Prüfmethode und Baustoffwahl, um eine genügende Langzeitqualität zu sichern.

1. CONCRETE STRUCTURES WITH A PLANNED SERVICE LIFE OF 100 YEARS

Economy is the key issue for practically all construction projects.Relevant for the economy of a project are not only the original construction costs but also the long-term costs accumulating during its service life. For large tunnel projects in Switzerland, the risks deriving from heaving anhydrous gypsum and clay rock formations, as well as the effects of sulfate and chloride containing underground waters, have to be accounted for already in the design stage and when letting the contract.

For the tunnels of the multi-billion project "Rail 2000", the target set by the Swiss Federal Railways (SBB) Administration is the following:[1]

"No basic restoration work should become necessary during the planned service life of 100 years, provided that normal maintenance is guaranteed."

Such exacting demands require adequate quality of materials and workmanship.

2. THE CORROSION OF CONCRETE

The durability of concrete is mainly determined by its resistance against the penetration of liquid and gaseous substances, i.e. the denser the cement stone, the higher the resistance of the concrete against chemical or mechanical attacks.[2] Most Standards therefore demand high watertightness besides the specific project-related requirements. Corrosion of concrete and masonry in Swiss tunnels is caused mainly by:[1]

- Soft water, above all if it is carbonated.
- Water with an aggressive amount of carbonic acid.
- Water containing sodium-, magnesium- or calcium sulfates.
- Water containing chlorides corroding re-bars and steel parts of the installations.
- Chlorides from de-icing salts and water containing polluants from agriculture or other remains from "civilization".

Evaporation of water on the free surface of the structure leaves the harmful substances of the underground waters inside the concrete and leads to an increasing concentration of such substances. Highly corrosive solutions may result from this, or cristallizing mineral salts may cause spalling.

3. STATE OF THE ART IN CONSTRUCTION PRACTICE

To improve the durability of concrete various measures are taken in practice to cope with the types of attack described above.

- The **dissolving attack** by soft and aggressive carbonated waters can quite successfully be countered by a very dense concrete matrix, i.e. decrease of the w/c ratio by means of high-range water reducers and addition of latent hydraulic silica fume.[3]
- As a rule, a cement with an as low as possible content of tricalciumaluminate (C_3A-content < 3%) is prescribed whenever there is a danger of **swelling attack** by sulfate containing water.[4] This should prevent the formation of ettringite ($3CaO\bullet Al_2O_3\bullet 3CaSO_4\bullet 32H_2O$), which increases 8 times in volume in the cement stone.

  In spite of such precautionary measures, considerable damages have shown up in several tunnels in Switzerland, caused by sulfate-swelling from thaumasite ($CaO\bullet SiO_2\bullet CaSO_4\bullet CaCO_3\bullet 14H_2O$). Thaumasite can develop independently from the C_3A contents. Cements with a low C_3A content may therefore in future not any more be considered as being sulfate resistant without further investigations. For important projects arises now the need for suitable materials selection criteria and test methods, which can give reliable results within acceptable time to guarantee a sufficient long-time durability of the project.

4. PRELIMINARY INVESTIGATIONS OF THE SBB FOR THE "RAIL 2000" PROJECT

Differences in the concrete mixes, the test solutions and the exposure durations made it impossible to directly compare the sulfate resistance tests done so far.
Therefore, in 1990, the Swiss Federal Railways initiated an extensive test programme with the aim to develop the necessary technology.The following conditions were stipulated:

A) **The concrete mix shall be designed to allow the continuous production of a consistent concrete quality on site without requiring excessive additional measures;**
i.e. Spread-table size of 45 ± 2 cm (slump ≈ 12 cm) has been determined for optimal workability.

B) **The requirements for the properties of the hardened concrete shall account for the nature of the chemical aggressors identified by trial borings as well as for the degree of importance of the different structural parts in the project;**
i.e. The final compressive strength on cubes shall be B55/45N/mm^2 (mean/min. value).

C) **High early strength shall allow economical production of precast tubbings;**
i.e. the compressive strength for demoulding of tubbings has been set at 15 N/mm^2 after 4.5 h. Standard test cubes have to develop f_{cw} > 10 N/mm^2 after 6 hours at 30°C in the laboratory for qualification.

D) **Testing of sulfate resistance shall account for the characteristics of underground water in Switzerland as well as for the high service life expectations for the project;**
i.e. ASTM C 1012-89 Standard Test [5] describes the testing of sulfate resistance (5% Na_2SO_4) of cement mortar. By analogy to this, an aggressive test solution of 5% Na_2SO_4 + 1%$MgSO_4$ has been selected for the testing of concrete .This corresponds to about 40 times of what was found in trial borings for different tunnels in Switzerland.[1]

For sulfate resistant concrete the Critical Length Change has been defined in due consideration of the smaller amount of cement paste in concrete and of the long service life expectations for the projects:

ASTM:	$\Delta l \leq 0.5‰$ after 1 year	⇒ high sulfate resistance
SBB:	**$\Delta l \leq 0.25‰$ after 2 years**	**⇒ high sulfate resistance**

The following test criteria have to be observed for qualification-testing of sulfate resistant and durable concrete in the laboratory:

- Laboratory mixes shall be prepared with materials (cement,aggregates...), which are representative of those available on site.
- Concrete mixes with insufficient workability shall be eliminated.
- Mix designs, which do not develop sufficient early or final strength shall not be considered any further.
- SBB requirements for sulfate resistance shall be met (6 test specimens per mix design).
- After each Length Change measurement the prisms shall be checked visually for cracks, spallation and deformation.

- If even only one out of six test specimens of a test series fails, the test shall be discontinued.
- Test series shall be followed by microscopic examination of thin sections to detect changes of the matrix and formation of ettringite/thaumasite. Strengthes shall be tested and compared to those of a virgin specimen.

5. SELECTION OF MATERIALS AND CONCRETE MIX DESIGN

89 different mix designs have been tested to gain confirmation that the chosen test method is correct.The large number of mixes came from the combinations of:

- 11 different cement types of Swiss and German origin (Portland cements, sulfate resistant cements, blast-furnace cements)
- 3 different aggregate gradings optimised to suit different uses (pumped concrete, heated concrete for precast tubbings, etc.).
- 2 different high-range water-reducers for w/c reduction and improved workability.
- 4 specially designed concrete additives based on silica fume for improved strength, watertightness and sulfate resistance.

Out of these 89 mixes, 61 had to be eliminated because of insufficient fresh concrete properties and poor workability.The remaining 28 mixes permitted to appraise the different materials as follows:[1], [6]

A) <u>Strength development</u>

The various types of cement differ above all in the <u>development of early strength</u>.
The **finely ground Portland Cements** (Type III) in combination with high-range water-reducers develop sufficient early strength, whereas "**Sulfate Resistant Cements**" (Type V) need special measures such as accelerators to reach the target. **Blast-Furnace Cements** do not develop sufficient early strength, not even with accelerators.

The required <u>final strength</u> is developed by **Portland** as well as **low C_3A Cements** (Type V) in adequate combination with high-range water-reducers, whereas **Blast-furnace Cements** do not gain sufficient final strength, not even if silica fume is added.

B) <u>Sulfate resistance.</u>

Concrete produced with **Portland Cement** and high-range water-reducing admixture have only a very modest sulfate resistance, even with very low w/c ratio (< 0.43). Top sulfate resistance results have been obtained only when specially designed Sikacrete additives were used. (Fig.1) The measured Length Changes (without swelling caused by storage of the specimens in water) remain very modest and the standard deviation of the compressive strength is small. The cracking on the test specimens after 2 years exposure to sulfates is minimal.

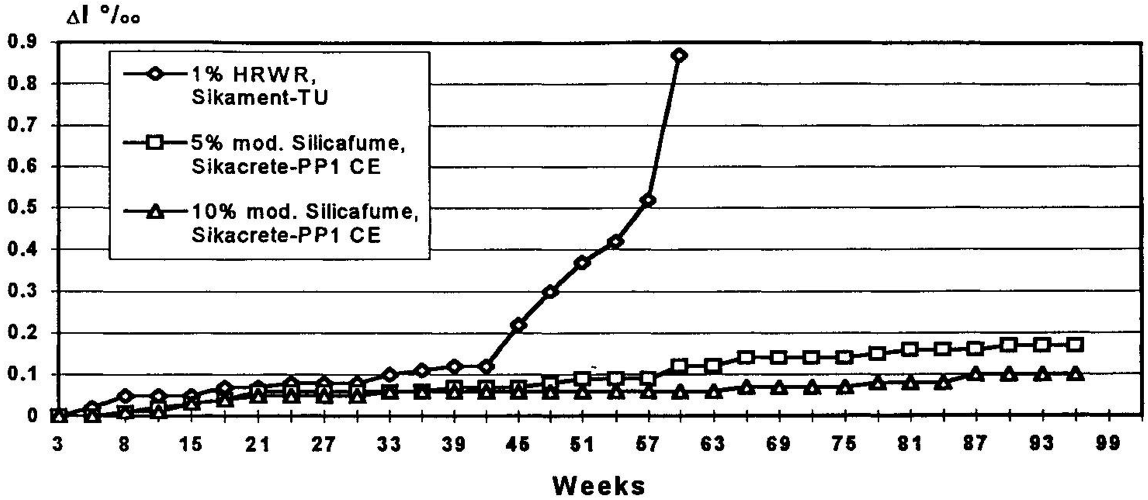

Fig.1 SBB Sulfate Resistance Test (modified ASTM C 1012-89)

Concrete mixed with the so-called **"Sulfate Resistant Cements"** gain only high sulfate resistance together with silica fume and the appropriate amount of high-range water-reducers. New accelerator technologies (aluminium-free) permit nowadays to compensate for insufficient early-strength development.

Water-reduction by means of appropriate admixtures has a beneficial effect on concretes produced with **Blast-Furnace Cements**. Addition of silica fume increases the sulfate resistance insignificantly only. Although many test specimens showed very little inner deterioration (little length change) their outer surfaces had been eroded by sulfate attack in such a way that the tests could not be carried on.

6. PRACTICAL RECOMMENDATIONS.

It has been established by the tests that High Early Strength Portland Cements (Type III) together with a specially designed additive (blend of silica fume, high-range water-reducers etc),give the highest sulfate resistance, higher even than low-C_3A Cements (Type V) together with the same additive.This fact is altogether very much in favour of precast-concrete production on site, because the required high early strength can be obtained without any problems.

To reach with the highest possible certainty the very high target of a 100 years service life for concrete structures under sulfate attack, the following recommendations must stricly be adhered to:

- The concrete matrix shall be very dense, i.e. continuous sieve curve with sufficient amount of very fines and a minimum of mixing water.
- The w/c ratio shall be < 0.42. In spite of all measures taken for sulfate resistance, the good workability of the concrete shall be guaranteed.
- Sulfate resistance is increased by the presence of a large number of micro-pores. Such pores, of the correct size (0.02 - 0.3mm), shall be introduced in an appropriate manner.
- Additives, specially designed to fit specific requirements, combining silica fume, plasticisers and air-entrainers, shall be used.

- Concrete shall be cured with utmost care to guarantee a tight surface and prevent early shrinkage cracks.
- In presence of underground waters containing sulfates **and** chlorides, re-bars shall need additional protection by impregnations, coatings or addition of integral corrosion inhibitors in the concrete. Appropriate coatings applied after demoulding, assure proper curing. Under no circumstances do such measures compensate for bad concrete mix design.
- ASTM C 1012-89 is suitable in principle for the testing of sulfate resistance, if complemented by a visual inspection of the test specimens (see Blast-Furnace Cement Tests under 5.B). The concentration of the test solution as well as the duration of exposure have to be chosen in accordance with the service life requirements of the project in question. The difference between the amounts of cement paste contained in mortar and concrete shall be duly taken into account.

7. FROM LABORATORY TESTS TO PROJECT SITE

Results and experience gained from several years of testing will be put to use for the first time on site for the construction of the 4.3 km long tunnel "Adlertunnel"of the "Rail 2000" project. The tubbings will be precast in a site factory, specially erected for that purpose. In order to increase production output, the concrete will be heated to 32 - 35°C by using hot water and steam and the elements will be steam-cured for 4 - 5 hours at 40°C. Compressive strength for demoulding shall be 15 N/mm^2 after 5 hours.

Laboratory testing as per SBB Specifications is completed.Tubbing production started early 1995. First results are not yet available at the time of printing of this paper, but will be presented at the time of the Symposium.

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Improved Durability of Concrete Structures in Hot Spring Districts

Amélioration de la durabilité des ouvrages en béton dans les stations thermales

Verbesserung der Dauerhaftigkeit von Betonbauten in Gebieten mit heissen Quellen

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SUMMARY

There are many bridges and other concrete structures in hot spring districts in Japan, despite harshness of corrosive environments in those areas. This paper reports on the results of a series of tests in which concrete specimens of various mixtures were exposed to different hot springs environments. The paper also discusses methods for extending the life of concrete structures built in hot spring areas.

RÉSUMÉ

Au Japon, de nombreux bâtiments et ponts sont construits dans les régions thermales malgré l'intensité de la corrosion dans cet environnement. Cet article rend compte des résultats d'une série de tests qui ont été effectués sur des échantillons de béton provenant de mixages variés et exposés à différents types de sources d'eau chaude. Cet article traite aussi des diverses méthodes appliquées pour prolonger la durée de vie des constructions en béton dans les stations thermales.

ZUSAMMENFASSUNG

Es gibt in Japan viele Bauten und Brücken aus Beton in Gegenden mit heissen Quellen, trotz der korrosiven Umgebung in solchen Gebieten. Es wird über eine Reihe von Versuchen berichtet, in denen Betonproben diverser Mischverhältnisse verschiedenen heissen Quellen ausgesetzt wurden. Die Arbeit diskutiert ausserdem Möglichkeiten zur Verlängerung der Lebensdauer von Betonbauten in derartigen Gebieten.

1. INTRODUCTION

There have been practices in Japan since olden days to take advantage of hot springs not only for medical purposes, but for social and recreational purposes, and people would get together around a hot spring and form a town. It is for this reason that the investment in social overhead capital including railways and roads is often found accumulated leading to the town where there is a hot spring. Included among the Japanese bridges since olden days were arch-type bridges built of stone, those built of wood and so on. In particular, they were overwhelmingly made of wood. Generally speaking, however, permanent bridges made of steel and concrete are quite common today. In this connection, hot-spring areas are no exception in that concrete is used for the construction of bridges. A concrete bridge in the hot-spring areas where a wooden bridge used to be free of any problem is laden today with a problem of chemical deterioration caused by substances contained in the hot spring. Thus, very harsh conditions prevail with respect to the durability of concrete structures in the hot-spring area in Japan and there exist many problems.

Therefore, this paper deals with measures for prolonging the life of concrete structures in the hot-spring district that have been taken on the basis of the result of a hot-spring exposure test involving concrete specimens.

2. ON HOT SPRINGS

When the temperature of the water which gushes out is over 25℃ or when it contains higher than a specific level of dissolved substances, it is defined as a hot spring in Japan. There are many methods of classifying hot springs. When classified by principal negative ion, they will be as Bicarbonate springs, Chloride springs, Sulfate springs. [1]

Furthermore, Fig. 1 [2] shows a distribution of hot springs in Japan, indicating hotsprings with less than pH 3 by ● and hot springs dealt with in this paper by ◉.

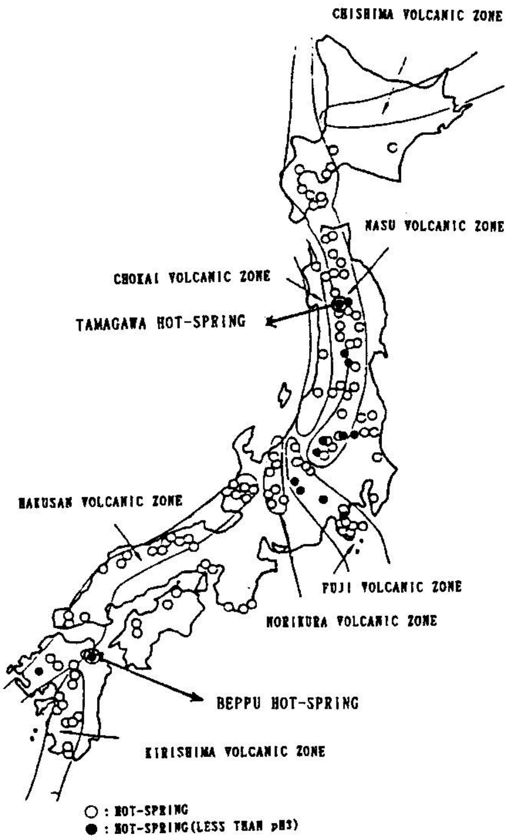


Fig. 1 VOLCANIC ZONE AND HOT-SPRING IN JAPAN

3. A FEW EXANPLES OF INPROVEMENT IN DURABILITY OF CONCRETE STRUCTURES IN HOT-SPRING DISTRICTS

3.1 Summary

Concrete structures have been constructed in Beppu and Tamagawa Hot Springs shown by ◉ in Fig. 1 mentioned earlier. When there is no countermeasure taken, concrete will be destined to degradation due to hot-spring ingredients. Thus, various measures have been taken against erosion based upon the result of an exposure test of concrete and anti-erosion concrete specimens. The following is a discussion on the measures for improving durability of concrete structures:

3.2 Examples of bridge substructure on sulfuric-acid, hot-spring foundation

A new road network has been installed in the Beppu hot-spring district, and concrete structures such as bridges, tunnels, and box culverts have been constructed. Measures taken to improve durability of the substructure of a bridge will be a discussed as follows:

3. 2. 1 Erosive environment

Beppu Hot Spring is a term applied in general to M, T and H Hot Springs. A comparison of chemical components of Beppu Hot Spring with those of Tamagawa Hot Spring, which will be discuss-ed later, will be as shown in Table 1. The acidity of T Hot Spring is high with a pH of 1.5, while that of H Hot Spring is almost neutral with a pH of 6.5. M Hot Spring is under such environments with various factors contributing to deterioration of the soil. [3]

	UNIT	BEPPU HOT-SPRING GROUP				TAMAGAWA HOT-SPRIG WATER
		M-SPRING WATER	M-SPRING SOIL *	T-SPRIG WATER	H-SPRIG WATER	
pH	—	2.25	1.7~2.6	1.5	6.5	1.3
Water Temp	℃	82	97~50	75~12	64~31	65
Cl^-	mg/ℓ	1.07	———	0.49	16.70	2147~2470
SO_4^{2-}	mg/ℓ	550	78400~3310	3700	63	857~1296

* SURFACE~40(cm)

Tab. 1 The analysis results of hot-spring water and soil

3. 2. 2 Method of examining preventive structure

Cyrindrical specimens of concrete, $\phi 10\times 20$ cm (M, T and H hot-spring water soaking test specimens) and prism specimens of concrete, $10\times 10\times 80$ cm (no n-painted M hot spring soil exposure test specimens), mixed respectively as shown in Table 2, were produced and a hot-spring water tank test and a soil exposure test were conducted.

No	W/C (%)	S/a (%)	A UNIT WEIGHT W kg/m3	C kg/m3	TEST RESULTS Slump (cm)	Air (%)	$*\sigma_{28}$ (kgf/cm²)	USED CEMENT	ADMIXTURE
I	31.0	30	155	500	19	0.2	600 (577)	POLTLAND CEMENT	HIGH WATER REDUCING AGENT
II	40.0	38	160	400	6	4.0	439 (411)	POLTLAND CEMENT	WATER REDUCING AGENT
III	52.3	39	157	300	6	4.0	347 (316)	POLTLAND CEMENT	WATER REDUCING AGENT
IV	79.0	41	190	240	6	1.2	153 (134)	POLTLAND CEMENT	WATER REDUCING AGENT
V	46.4	39	158	340	7	4.0	342 (328)	SULFATE RESISTING CEMENT	WATER REDUCING AGENT

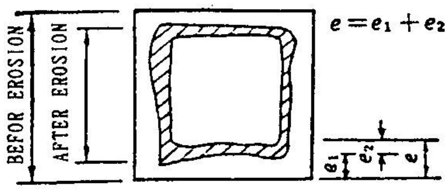
$*\sigma_{28}$ UPPER LINE: STADARD CURING AT 20℃
UNDER LINE: CURING IN THE FIELD

Tab. 2 Specification of concrete specimens in BEPPU

Moreover, an exposure test was carried out to the soil of M Hot Spring, using 6 prism specimens, $10\times 10\times 80$ cm, of concrete mixed as Mixture III shown in Table 2, the surface of which were coated with 6 kinds of anti-erosion measures as shown in Table 3 after a material age of 28 days. The conditions of deterioration of the soaking specimens and soil exposure test specimens were judged from factors such as external appearance observation, weight changes and erosion depths. As shown in Fig. 2, the depth of erosion has been defined as a combination of the part lost and the depth of carbonation.

	KIND OF ANTI-EROSION MATERIAL AND METHOD	THICKNESS
E	COATING WITH EPOXY-AROMATICPOLYAMIDE	1.2mm
AS	GLASS-CLOTH COATING WITH ASPHALT-EPOXY	3.0mm
EM	MORTAR LINING WITH EPOXY-AROMATICPOLYAMIDE	10.0mm
PM	MORTAR LINING WITH UNSATURATED POLY-ESTER	10.0mm
AM	MORTAR LINING WITH ASPHALT-EPOXY	10.0mm
CM	POLYMER-CEMENT MORTAR LINING	10.0mm

Tab. 3 Specification of anti-erosion in BEPPU



e :DEPTH OF EROSION (mm)
e_1:DEPTH OF LOST (mm)
e_2:DEPTH OF CARBONATION (mm)

Fig. 2 The definition of cprrosion depth

3. 2. 3 Relationship between concrete mixtures and deterioration

5 kinds of concrete specimens were soaked and exposed in 3 hot springs each with a different pH chosen from among the Beppu hot-spring group, after which weight changes and erosion depths were determined, the result of which is as shown in Fig. 3. The result has revealed that the deterioration of concrete is dictated more heavily by the kind of soaked hot-spring water rather than the kind of cement and its mixture used for the concrete used.

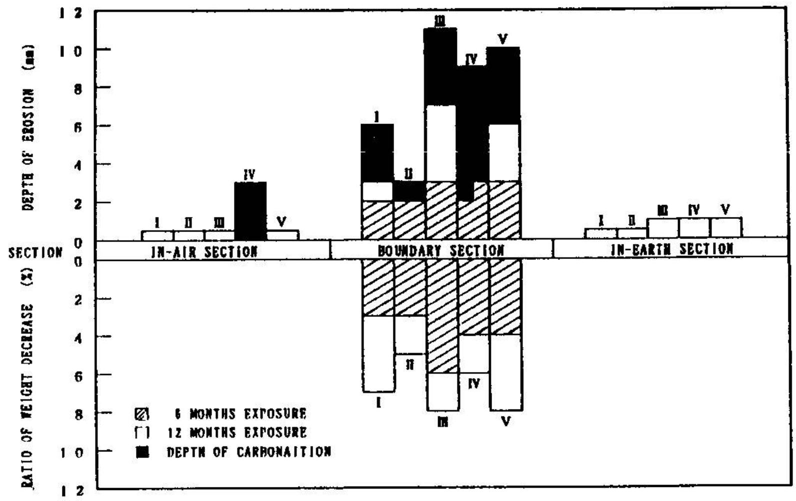


FiG. 3 Depth of errosion and ratio of weight decrease at soaked spacimens

As a next step, the result of a soil exposure test of non-coated prism specimens from M Hot Spring is as shown in Fig. 4. The degree of deterioration was conspicuously higher near the surface (5 cm above and 10 cm below the ground surface), whereas the section more than 5 cm above the ground surface showed not so much deterioration. Subsequently, Fig. 5 shows the depths of erosion and the rates of weight decrease with the specimen divided into the in-air section, boundary section, and in-earth section. The result indicates that the deterioration in the boundary section is the most conspicuous and the degree of deterioration is just about the same as the specimen soaked in the M hot-spring water. The depth of erosion in the in-earth section is about 1/7 that of the boundary section, and the deterioration in the in-air section is almost negligible. From above, the pH of the soil, the period of exposure, and the depth of erosion have been estimated as shown in Formula below:

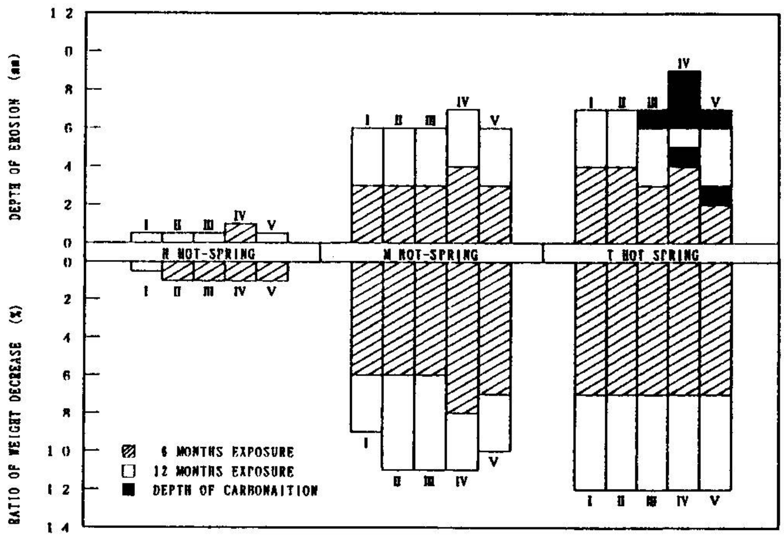


FiG. 4 Depth of erosion and ratio of weight decrease at soil exposur test spacimens

$\delta = K \cdot (7-pH) \cdot t$ ------(1) [4]

Where d: Depth of erosion (mm)

K: Coefficient corresponding to structural section (boundary section; 1.4, in-earth section; 0.4)

pH: Hydrogen ion concentration at exposed section

t: Years exposed

3.2.4 Samples coated with anti-erosion methods of construction

3.2.4 Samples coated with anti-erosion methods of construction

The result of measurement obtained from a 3-year exposure test to the M hot-spring soil of the concrete specimens coated with the anti-erosion material shown in Table 3 is as shown in Table 4. With the boundary section, the anti-erosive material was deteriorated other than EM and the anti- erosive function of concrete ceased to exist. In the case of the in-earth section, a 10-mm-thick lining specification was recognized as effective. However, the result was variable from specification to specification when the film was thin, and this coating cannot be adopted under existing erosive environments.

	SECTION	THE RESULT OF OFSERVATION	ADHESIVE	WEIGHT CHANGES	Carbonation (mm)
E	IN-AIR BOUNDARY IN-EARTH	No problem expect change of color Coating film damaged in 3.5years No problem expect change of color	▲ ▲ ▲	+2.0	0 0 0
AS	IN-AIR BOUNDARY IN-EARTH	Partial Blister Scaling (depth of erosion:9mm) Softening, mixing to soil	◎ × ◎	-3.4	0 25 0
EM	IN-AIR BOUNDARY IN-EARTH	No problem expect change of color No problem expect change of color No problem expect change of color	◎ ◎ ◎	+1.8	0 0 0
PM	IN-AIR BOUNDARY IN-EARTH	No problem expect chalking Thermal deterioration crack No problem expect change of color	◎ △ ◎	+0.5	0 0 0
AM	IN-AIR BOUNDARY IN-EARTH	Fall down Scaling (depth of erosion:13mm) No problem expect change of color	× × ×	-7.1	0 2 0
CM	IN-AIR BOUNDARY IN-EARTH	Fall down Scaling (depth of erosion:10mm) No problem expect change of color	× ▲ ◎	10.7	0 3 0

REMARK: ◎ VERY GOOD, △ GOOD, ▲ NO GOOD, × BAD

Tab.4 The result of a soil exposure test

3.2.5 Preventive measures [4]

Based on the above-mentioned result, preventive measures have been determined as follows:

1) Boundary section ... Thickness-added method of construction plus use of anti-erosive material

2) In-earth section ... Thickness-added method of construction

The thickness-added method of construction means that the cover concrete on the outside is increased in thickness for placement in one-piece, expecting that the erosion by hot spring will not reach the effective cross-section of the concrete itself, and the thickness to be added on was obtained from Formula (1) mentioned above.

It has been 10 years since the facilities were constructed in the anti-erosive method of construction discussed so far and there seems to have been no problem in the absence of conspicuous abnormality in appearance.

3.3 Examples of facilities to process neutralization of chloric hot-spring water

In order to construct a dam in the upstream of Tamagawa Hot Spring, it was decided to build a neutralization facility because the ingredients of the hot-spring water on the up-stream-side were highly acid as shown in Table 1. The following is a discussion on the measures to improve durability of the facility.

Slump	W/C	Air	S/a	UNIT WEIGHT (kg/m^3)				
(cm)	(%)	(%)	(%)	W	C	S	G	AE WATER REDUCING ADMIXTURE
8	55	4.5	45	160	291	819	1053	0.728

Tab.5 Specification of concrete specimens in TAMAGAWA

3.3.1 Erosive environment

In Table 1 mentioned above, the ingredients of Tamagawa hot spring are shown. The carbonation facility at stake was designed to take in the hot-spring water with a pH of 1.1 and neutralizes it prior to discharging. This was due to the fact that the hot-spring water was chloric, calcium chloride produced from the hot-spring water reacting to a cement hydrate was easily soluble and it was feared that there might be deterioration with the concrete flowing out.

3.3.2 Examination of anti-erosive construction method

In view of the result in the foregoing item, concrete specimens, 30×30×10 cm, were prepared in accordance with the mixture shown in Table 5. After the specimens were cured for more than 28 days, they were coated with the anti-erosive material shown in Table 6 and made anti-erosive specimens.
The result of observation of external appearance in a 2-year exposure test is as shown in Table 7.

	KIND OF ANTI-EROSION MATERIAL AND METHOD	THICKNESS
E	COATING WITH EPOXY-AROMATICPOLYAMINE	3.0mm
E C	GLASS-CLOTH COATING WITH EPOXY-POLYAMIDE	2.0mm
E M	MORTAR LINING WITH EPOXY-AROMATICPOLYAMINE	6.5mm
P	GLASS-FLAKES WITH UNSATURATED POLY-ESTER	2.5mm
S	SEAT LINING WITH PLLYVINYL-CHLORIDE	3.0mm
N	NON-COATING	—

Tab. 6 Specification of anti-erosion

The results discussed so far have led to the adoption of the paint group EM for the structures which come in contact with the raw water or diluted water mixture and the paint a group P for the facilities which come in contact with the water (pH 5 or higher). An observation in 2 years after construction has revealed nothing unusual about the paint group EM, but swelling or rising has been witnessed with the paint group P.

	THE RESULT OF OFSERVATION
E	Change of color Blistering (ϕ5mm)
E C	Change of color Blistering, (partial)scaling, cracking
E M	No problem expect change of color
P	Chalking Blistering(ϕ8~20mm)
S	Chalking Blistering(ϕ10~30mm)
N	flows out severely from the surface

Tab. 7 The result of a exposure test

4. CONCLUSIONS

This paper mainly deals with the experiments performed by the author as regards the measures for improving durability that have been applied in the hot-spring district of strong acidity in Japan. The ingredients are variable from hot spring to hot spring, and so are the erosive environments. In either case, many of them contain harmful ingredients flowing out of concrete. It can, at least, be said that in the case of hot springs with strong acidity, the kind of cement or the blending of concrete alone can not contribute to solve problems as seen from the standpoint of durability. Subsequently, it has been made clear that there will have to be anti-erosive measures taken in one way or another. The anti-erosive material itself is required to be durable and adhesive, apart from the ability to intercept any elements to facilitate the deterioration of concrete. It is hoped that this report will be able to contribute to the designing of durability in concrete structures in the hot-spring district.

In the preparation of this paper, several pieces of literature have been referred to. At the end of this report is a list of reference materials used, and the author would like to express appreciation to all those concerned for the opportunity given to refer to such data.

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Specifying Effective Curing during Construction to Ensure Durable Concrete

Durabilité du béton par une cure efficace en cours de construction

Festlegung wirksamer Nachbehandlung für dauerhaften Beton

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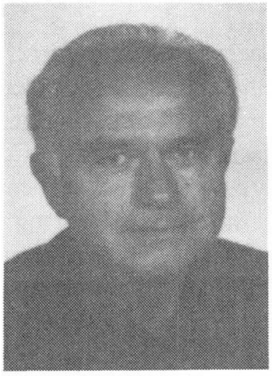


A graduate of the University of Cape Town in 1961, Gert Hoppe has been involved with the design, construction and rehabilitation of concrete bridges. He obtained an MSc Degree in 1981 for research on properties of site and laboratory concrete and has specialised in rehabilitating bridges and specifications for durable bridges.

SUMMARY

The erroneous assumption that concrete strength and durability are closely linked has now been disproved by research. A durable concrete is produced by ensuring that the mix is properly constituted, placed, compacted and cured. To ensure that durability is achieved on site, either much more rigorous supervision of the concreting must be done or tests must be developed which accurately measure the degree of durability of concrete. The proposed tests measure the oxygen permeability, water sorptivity and chloride conduction properties. The results are compared with specified acceptance criteria in accordance with required quality of cured concrete to ensure durable concrete.

RÉSUMÉ

Les récents résultats de la recherche remettent en cause l'hypothèse largement admise d'une étroite liaison entre résistance et durabilité. Pour qu'un béton soit durable, il est indispensable que la composition, la mise en oeuvre, le vibrage et la cure soient irréprochables. Il faut une surveillance rigoureuse des opérations de bétonnage et des procédés adéquats d'essais, afin de garantir la durabilité désirée. Les méthodes proposées servent à mesurer la perméabilité à l'oxygène, le pouvoir d'absorption d'eau et la conductibilité vis-à-vis des chlorures. Les résultats d'essais sont à comparer avec les critères de réception définis en fonction de la qualité de la cure et de la durabilité du béton.

ZUSAMMENFASSUNG

Die Annahme, dass Festigkeit und Dauerhaftigkeit von Beton eng miteinander zusammenhängen, wird jetzt von neueren Forschungsergebnissen in Frage gestellt. Um die Dauerhaftigkeit auf der Baustelle zu sichern, muss der Betoniervorgang sehr viel strenger überwacht werden, zusammen mit der Entwicklung geeigneter Testverfahren zur genauen Bestimmung der Dauerhaftigkeit. Die vorgeschlagenen Verfahren messen die Sauerstoffdurchlässigkeit, das Wasserabsortionsvermögen und die Chloridleitfähigkeit von Beton. Die Ergebnisse werden zu definierten Annahmekriterien für die Qualität der Betonnachbehandlung in Beziehung gesetzt.

1. INTRODUCTION

The erroneous assumption that concrete strength and durability are closely linked together has long been espoused by Engineers even though research has largely disproved the connection. Strength certainly does have a role to play in producing durable concrete but specifications which are strength criteria alone may result in concrete of poor durability characteristics. A durable concrete is produced by ensuring that the mix is properly constituted, placed, compacted and cured.

2. EFFECTS OF CURING ON CONCRETE PROPERTIES

2.1 Compressive Strength

Most of the early research into curing of concrete focused on the effect that curing has on compressive strength. This was mainly due to the emphasis Engineers placed on strength as the most important material property of concrete. Research on the effect of curing on strength development has been useful to highlight trends resulting from different curing histories. Continuous moist curing was found to provide higher long-term strength than concrete which was air cured at any stage.

Little research data is available concerning the effect of curing on compressive strength of concrete structures on site. Further work needs to be done in this regard as data available from laboratory investigations of curing can only be used for comparative purposes. The effect of curing on the overall structural integrity of the concrete structure needs to be quantified.

2.2 Durability

2.2.1 Durability Parameters

The most significant concrete parameters defining the resistance of concrete to deterioration are the permeation characteristics of the surface and near-surface concrete. Not only does the cover concrete interface and react directly with the environment but it is the concrete which has the poorest quality and most likely to suffer [1][2]. Permeation can be divided into three distinct but connected transportation phenomena for moisture vapour, dissolved ions, gases and aqueous solutions :

PERMEATION

Absorption Diffusion Permeability

Absorption describes the process by which concrete takes in a liquid, normally water or aqueous solution, by capillary attraction. The rate at which water enters is termed absorptivity (or sorptivity). The moisture may contain dissolved salts, such as chlorides or sulphates and dissolved gases, such as oxygen, carbon dioxide and sulphur dioxide. The transportation of ions is therefore often a combination of absorption and diffusion.

Diffusion is the process by which a vapour, gas or ions can pass through concrete under the action of a concentration gradient. Diffusivity defines the rate of movement of the agent and is the mechanism by which carbonation occurs and also characterises the ingress of chlorides and other ions. It is, therefore, closely linked to reinforcement corrosion problems.

Permeability is defined as the flow property of concrete which quantitatively characterises the ease by which a fluid or gas will pass through it, under the action of a pressure differential. This contrasts with absorption and diffusion which are caused by a concentration differential.

Durability properties investigated include permeability, sorptivity, carbonation, chloride diffusion.

2.2.2 Permeability

Permeability testing of concrete has long been recognised as an effective way of assessing the potential durability of concrete which is governed more by its porosity and permeability than by bulk properties such as strength and elastic modulus. Researchers have shown that the oxygen permeability of water cured concrete was significantly lower than similar air cured concrete.

2.2.3 Water absorption

Water absorption is a useful durability parameter as often the predominant mechanism causing water movement into concrete is capillary action rather than an applied water head. Researchers have reported that concrete that was not cured at all had up to ten times the surface absorptivity of well cured concrete. Extending the duration of water curing was found to reduce the amount of water absorbed by concrete. Intermittent water spraying was found to be far less effective for curing concrete than full immersion curing.

2.2.4 Water sorptivity

Water sorptivity is a measure of the rate of movement of a water front through a porous medium such as concrete, due to capillary action. It may therefore be considered to be very similar to water absorption. Continuous water curing was found to produce concrete with much lower sorptivity than uncured concrete. Similar reductions of sorptivity were found for concrete of higher compressive strength and concrete made with water reducing agents.

2.2.5 Chloride diffusion

Chloride initiated reinforcement corrosion is a major form of concrete deterioration in the marine environment. The penetration of chlorides into concrete is largely due to ionic diffusion and water movement caused by wetting and drying cycles. The speed at which these processes take place is dependent upon the quality of the cover concrete and research has shown that well cured concrete has significantly lower chloride contents than similar uncured concrete.

2.2.6 Carbonation

Carbonation of concrete can cause corrosion of reinforcement due to the resulting reduction of concrete alkalinity around the steel. Carbonation is determined by the rate of gaseous diffusion of atmospheric carbon dioxide into the concrete pore structure. Researchers using an accelerated carbonation test showed that the depth of carbonation decreases with increasing length of curing. It was found that curing for up to seven days produced dramatic reductions in carbonation but very little beyond seven days.

2.3 Different Cement Binders

Comparisons between OPC concrete and other types of concrete containing mineral extenders such as fly ash and GGBS are complicated by how the original mix design was formulated. Comparisons can be made on strength replacement, replacement plus addition or addition only.

2.3.1 OPC concrete

Most of the research on curing has focused on OPC concrete which is the most commonly used cementitious binder. The effect of curing on other cementitious binders has more recently received attention as these materials have gained greater acceptance in construction. Pozzolanic materials such as fly ash and granulated slag have traditionally been considered more vulnerable to poor curing practice as the pozzolanic reaction is generally slower than that of normal cement hydration. Research on the sensitivity of other cementitious binders to curing has generally been done by comparing their performance to that of similar OPC concrete.

2.3.2 Fly ash concrete

Researchers have investigated the effects of curing on durability properties of fly ash concrete. Durability indexes investigated were oxygen permeability and water sorptivity. Results indicated that fly ash concrete was more affected by curing than OPC concrete when oxygen permeability and sorptivity results were compared. Of more significance however was the fact that the fly ash concrete still had lower values of oxygen permeability and sorptivity than similar OPC concrete.

2.3.3 Slag concrete

Researchers using the oxygen permeability and sorptivity durability index tests to measure the ability of slag concrete to withstand poor curing practice and found that slag concrete was more vulnerable to poor curing than similar OPC concrete.

2.4 Concrete Structures in Service

Very little research has been done on the effect of curing on concrete structures in service. Curing of concrete has been investigated almost entirely by laboratory based research using small concrete specimens exposed to static environmental conditions. More field data is required to quantify the effect of poor curing on the durability performance of concrete in service.

There is a widely held view that the lack of durability of many concrete structures is due to inadequate curing but researchers have shown that there are hardly any examples in the technical literature to substantiate this opinion. This is because concrete deterioration is invariably caused by a combination of different factors, which make assessing the role of curing difficult. Other construction processes which can be detrimental to concrete durability include inadequate compaction, over vibration, low cover to reinforcement and bad design leading to excessive cracking. Poor curing may be responsible for more concrete deterioration than is generally recognised but the means of determining the effect of poor curing do not exist.

3. RECOMMENDATIONS FOR ENSURING GOOD CURING PRACTICE

These recommendations may be classified as being either prescriptive or performance methods.

Prescriptive Methods

These have been the traditional methods used to date to ensure adequate curing. Many people believe if the curing method is practical and strictly enforced on site there should not be a problem. Finding a practical curing method and guaranteeing that it is adhered to on site is not so simple and some argues that curing compounds ensure the most effective curing in many situations as other curing methods are difficult to maintain and supervise, while others do not recommend any particular curing method but states that the curing method must be fully specified by the designer and listed separately in the Bill of Quantities.

Performance Methods

Performance methods have recently been developed to assess the effectiveness of the curing method chosen. Methods include in-situ tests of surface quality and testing of samples extracted from the structure for later testing in the laboratory. As yet no test has been developed which has enjoyed any widespread acceptance in the construction industry.

The advantage of performance testing of concrete is that the contractor is free to choose the curing method most suitable for his circumstances provided the concrete achieves the compliance criteria. The difficulty of performance testing is reaching agreement with all parties about the performance method chosen and ensuring the test is suitable and reproducible on site. More research and testing is required in this area before performance testing of concrete curing can become viable. Considerable research effort is being focused on the problem and positive results from this work should become available for practical implementation in the near future.

4. METHODS OF DEFINING DEGREE OF POTENTIAL DURABILITY

4.1 Introduction

To determine the degree of durability of a concrete it is necessary to test the material for properties such as water absorption, chloride diffusion and permeability. Obtaining absolute figures on these material properties is difficult because the exposure and material conditions are constantly changing. Index tests overcome these problems by preconditioning the sample initially and standardising the exposure conditions. The test therefore simulates the transport mechanism causing deterioration and produces results which are reliable and can be used for comparative purposes. This can be done by correlating durability index results with the long term performance of concrete in a particular environment.

4.2 Laboratory-based Tests

Extracting samples from concrete structures and conducting durability tests in the laboratory has the advantage of being able to condition the concrete before testing and test in a controlled environment.

Laboratory based tests are generally more reliable than insitu tests but results can only be used as an indication of the likely durability performance of the structure. Care must be taken during sampling to ensure that the set of samples taken are representative of conditions on site.

4.2.1 Oxygen Permeability

The coefficient of permeability is controlled by the size and continuity of pores, the presence of cracks and microcracks and transition zone defects between aggregate and cement paste. Good quality aggregate is usually sufficiently dense to have little effect on the overall permeability of the concrete.

Testing for permeability has widely been recognised as a method of defining the degree of durability of concrete. The tests may be broadly divided into either through-flow, penetration and falling head tests. The falling head permeameter developed by Ballim [3] measures the coefficient of permeability quickly and economically. The test subjects oven-dried concrete samples to oxygen gas under pressure and the pressure drop caused by oxygen diffusing through the concrete is monitored. Preconditioning the concrete by oven-drying at 50°C for seven days does cause some microstructural damage but the conditioning tries to ensure that all concrete is tested from the same start point.

Permeability is generally acknowledged to be sensitive to the type of curing applied to concrete. Results from the oxygen permeability test show that dry cured concrete was more permeable than similar concrete which had been moist or wet cured. The difference in the coefficients of permeability for dry and wet cured concrete could be as high as one order of magnitude.

4.2.2 Water Sorptivity

Sorptivity is defined as the rate of movement of a water front through a porous material under capillary action and due to the mechanism of water absorption, sorptivity is dependent on the initial water content of concrete, the temperature and the type of fluid being used.

Ballim [4] developed a simple sorptivity test based upon Kelham's method. The test is performed on cored concrete samples which are oven-dried and then exposed to water on the top surface to allow unidirectional water absorption. By measuring the weight of water absorbed with time, the progress of the water front through the concrete and therefore the sorptivity can be determined.

Sorptivity is sensitive to the type of curing performed on concrete, with wet curing producing the lowest sorptivity values.

4.2.3 Chloride Conduction

Aggressive agents such as chlorides are generally transmitted by ionic diffusion particularly at depths greater than 20 mm where water absorption effects are negligible.

Ionic diffusion depends upon the concentration gradient driving the process, the degree of saturation of the concrete pores and the physical and chemical resistance of the concrete to the diffusion process. Diffusion tests are usually accelerated in the laboratory by applying a potential difference across the sample, increasing the concentration gradient or using a combination of both. Most accelerated diffusion tests take several days to run as the concrete needs time to reach steady state conditions before diffusion measurements can be taken. This time delay can result in inaccurate readings especially when rating curing efficiency as the cement may continue hydrating during the test. The chloride ions also react with the products of hydration during the test which effectively changes the pore structure.

Streicher [5] has developed an extremely rapid chloride conduction test which takes only a few seconds to run once the samples have been preconditioned by vacuum saturating with a concentrated salt solution. Theoretically it is possible to relate chloride diffusion to chloride conduction so that the method can be used as an index test. Experimental work has been done on the sensitivity of the chloride conduction test to the type of curing and was found to be sensitive to different types of curing. The test has also been found to be sensitive to changes in pore structure due to mineral extenders such as fly ash and slag.

4.3 Insitu Tests

A number of insitu tests have been developed to assess the durability of concrete structures in service. These tests are generally non-destructive in nature but may involve some drilling of concrete either to mechanically fix the apparatus or to prepare a hole through which the concrete will be tested. The ISAT test was originally developed to test concrete roof tiles but has been widely used and modified. The test measures the rate at which water is absorbed into concrete by clamping a perspex cap on the concrete surface. A drawback of the test was the effect of surface coatings and carbonation which affected the reliability of the results. To overcome this the Figg test was developed which uses a 5.5 mm diameter hole to measure water absorption or gas diffusion through the cover concrete. Many modifications of the Figg test were produced including the Covercrete Absorption Test (CAT) and the CLAM [6] test. All of these insitu tests were found to be significantly affected by the original moisture content of the concrete. Even though corrections can be applied to the results to allow for the variable moisture content, determining the actual moisture value insitu is unreliable. A variety of methods were proposed to overcome the problem and now Dhir et al [7] proposed a preconditioning process where the concrete was vacuum dried using a portable vacuum pump immediately before testing.

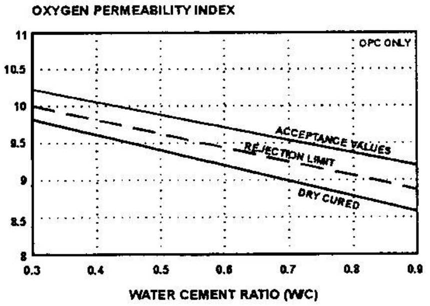
5. ACCEPTANCE CRITERIA FOR DEGREE OF DURABILITY ACHIEVED ON SITE

5.1 Introduction

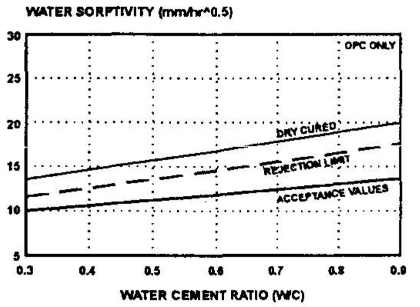
Based on the present state of research, it is proposed to measure the following properties using laboratory based tests, where the age of coring the concrete is 28 days, which implies that the concrete is tested at 35 days to allow for time taken to condition the samples.

In order to achieve acceptable durability the contractor's method of curing must ensure that the values for the oxygen permeability, water sorptivity and chloride conductivity and of the site concrete fall below the following graph values, which need to be established per region and country [1][8].

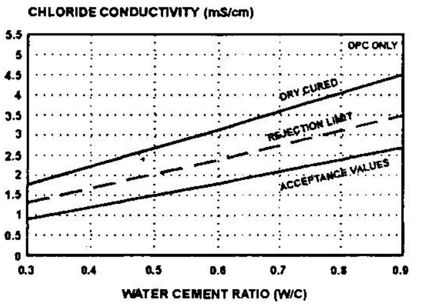
5.2 Oxygen Permeability



5.3 Water Sorptivity



5.4 Chloride Conduction



6. CONCLUSIONS

The present approach to ensuring durability of concrete structures is too haphazard to guarantee achieving the design life. Durability can only be confidently predicted once relevant material properties are specified and achieved during construction. Tests are needed to measure these material properties affecting durability and practical implementation of the tests into contract specifications must be done. Defining the degree of durability of concrete is complicated by the fact that different environments demand differing durability characteristics from concrete structures. No single test would adequately measure concrete durability for all forms of service functions and types of deterioration. It is therefore necessary to develop a number of tests and to use that test most appropriate to the exposure and service conditions for the structure in question. More data is required before any of the tests can be used as part of the project specification of a contract. This type of approach would need to be run in tandem with more traditional methods of ensuring concrete curing to assess the practical problems which might arise in applying the tests on site.

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High Performance Concrete for the Lacey V. Murrow Floating Bridge

Béton à hautes performances pour le pont flottant Lacey V. Murrow

Hochleistungsbeton für die Lacey-V.-Murrow-Schwimmbrücke

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SUMMARY

In early 1991, the Washington Department of Transportation began the design of twenty prestressed concrete pontoons for the replacement of the Lacey V. Murrow Bridge in Seattle, Washington. Designers of the pontoons were seeking watertight, durable concrete that would permit economical and high quality construction of these rather complex structures. The experience of the Owner and the Consulting Engineer indicated that these properties were achievable, but it would be necessary to develop specific material performance data and specifications. Research and development of the concrete mix designs, begun in mid-1991, allowed the beginning of construction in 1992.

RÉSUMÉ

Au début de 1991, le "Washington Department of Transportation" lança l'étude de 20 pontons en béton précontraints pour le remplacement du pont Lacey V. Murrow à Seattle. Il s'agissait alors de rechercher un type de béton à haute résistance et étanche à l'eau, dont la mise en oeuvre devait offrir une solution économique et de grande qualité pour ce genre de structure portante fort complexe. Si l'expérience du maître de l'ouvrage et de l'ingénieur-conseil tendait à confirmer la possibilité de satisfaire à ces exigences, il fallait encore établir un cahier des charges et des caractéristiques spécifiques pour les matériaux. Les recherches et le développement relatifs à la formulation de la composition du béton débuta vers la mi 1991, de sorte que la construction put démarrer en 1992.

ZUSAMMENFASSUNG

Anfang 1991 begann das "Washington Department of Transportation" mit dem Entwurf von 20 Spannbeton-Pontons als Ersatz für die Lacey-V.-Murrow-Brücke in Seattle. Für diese recht diffizilen Tragwerke wurde nach wasserdichtem, beständigem Beton gesucht, der eine wirtschaftliche und qualitativ hochstehende Bauausführung ermöglichen sollte. Nach Erfahrung von Eigentümer und beratendem Ingenieur waren diese Anforderungen erfüllbar, doch mussten spezifische Materialkennwerte und -pflichtenhefte erarbeitet werden. Die Entwicklung der Betonrezeptur begann Mitte 1991, so dass 1992 mit dem Bau angefangen werden konnte.

1. HISTORY OF FLOATING BRIDGES IN WASHINGTON

Four floating bridges have been designed and built in the State of Washington by WSDOT since 1940. The concrete in all of these has performed well, even under severe exposure conditions of saltwater, winds and waves, freezing and thawing, and abrasion. The state-of-the-art designs used for the two most recent floating bridges, I-90 and the LVM replacement, require high strength, low shrinkage, and low permeability concrete. These designs also include thin, deep bulkheads and walls, which are heavily reinforced and contain post-tensioning ducts. Thus, the demands for high performance concrete have increased.

The LVM replacement pontoons were necessitated by the sinking of the original 50-year old bridge in November, 1990. Design of the new pontoons with prestressed high performance concrete was to limit or prevent cracking and leakage. Corrosion of reinforcement and prestressed steel in the fresh water of Lake Washington, site of the LVM bridge, is not considered a severe threat to structural integrity, but can occur over a long period of time with normal concrete.

2. CONSTRUCTION DEMANDS ON CONCRETE

The greatest concern by WSDOT for water tightness and durability was in the outer shell of the pontoon which is directly exposed to water. This area of the pontoon requires fresh concrete properties be given special attention. Flowable concrete, with sufficient cohesiveness to prevent segregation, is needed for placing concrete in the heavily reinforced, deep walls. Workable concrete, with moderate slump and normal setting time, is required for flatwork in the slabs. While those characteristics are not necessarily incompatible, they do depend on a great deal of flexibility in the concrete mix.

Contractor incentive for early completion, and WSDOT's desire to reopen this vital link on the heavily-travelled Interstate 90 commuter corridor, dictated fast-track construction conditions. The design encouraged large, continuous concrete placements in walls and slabs in order to minimize construction joints. These factors combined to impose requirements for consistent and controllable concrete quality. Low slump loss and effective slump control were critical elements for consideration in the concrete mix design and development.

3. MIX DESIGN REQUIREMENTS

Parameters for the recommended final mix design, concrete placement, and curing were selected after all results from a mockup test were analyzed. A certain amount of flexibility was allowed in the mix design so the contractor could optimize the proportions to fit a particular supplier's materials. The table below compares the specified concrete properties to the contractor's final mix design and the WJE final test mix.

	WSDOT Specification	Contractor Mix Design	WJE Final Test Mix Design
Portland Cement, kg/m^3	371 min.	371	380
Silica Fume, kg/m^3	30 - 42	30	38
Fly Ash, kg/m^3	59 min.	59	83
Water-cementitious ratio*	0.33 max.	0.33	0.33
Max. Slump, in.	225	225	210

* Cementitious material includes cement, silica fume, and fly ash

Table 1 Comparison of LVM Mix Designs and Specifications

A maximum coarse aggregate size of 12 mm was specified because of tight clearances in the walls and knowledge obtained from the performance of other high strength concrete in the Seattle area and the LVM trial mixes. A coarse, WSDOT Class 1 sand was specified because it has exhibited the best workability and highest strength in previous applications of high strength concrete.

Trial mix tests and previous experience had shown that a combination of admixtures produced optimum workability, strength, slump retention, and density. It is well known that silica fume concrete requires the use of a high range water reducer because of the extra water demand created by the extreme fineness of the admixture. The introduction of a normal range, retarding water reducer at the batch plant, with a portion or all of the high range water reducer, is standard practice in the Seattle area for high strength concrete. The retarder aids in better slump retention and reduces the total amount of high range water reducer, thereby producing better and longer lasting workability. Thus, both types of water reducers were specified.

The decision to use non-air entrained concrete for the pontoons was somewhat controversial. There is not total agreement in the concrete industry that air entrainment is essential in high strength concrete with a very low water-cement ratio. It has been demonstrated in previous research [2] that some entrained air is necessary to produce concrete that is resistant to the severe exposure of standard rapid freeze-thaw tests. However, successful experience in the mild Seattle climate, and even the severe Alaska climates, with non-air-entrained high strength concrete in piling, pier decks, and other bridge pontoons, supports the argument for omitting entrained air. The air-entraining agent, besides adding a difficult control element in high performance concrete, produces stickiness that impairs workability and placeability.

4. MIX PERFORMANCE REQUIREMENTS

The ability of concrete to be watertight can be measured, to a large degree, by its permeability and shrinkage. The rapid chloride permeability test, AASHTO T-277, is now commonly used to measure the resistance of concrete to intrusion of chlorides. Low values of permeability are considered to be consistent with water tightness. The specifications required rapid chloride permeability tests for acceptance of the mix design, and also as a quality assurance test during construction.

Thermal shrinkage was a concern with this type of structure. Thermal shrinkage of newly-cast concrete against previously-cast concrete can cause cracking in thick sections, as the new concrete hardens and cools and is restrained by the older, cooler concrete. This could happen, for instance, at the base of a wall cast on top of the base slab. Temperature rise of the LVM mix was minimized by the use of Type II cement, lowest possible cement content, addition of pozzolans, and the cooling of formwork after concrete placement. The temperature of concrete in walls was required to be monitored at selected times during construction to assure that thermal shock would not occur as forms were stripped. The mild winter temperatures in Seattle assist in minimizing differential thermal shrinkage.

The specifications required that external vibration be used on the wall forms to assist in consolidation. This followed from results of the test mockup and previous experience in constructing other pontoon walls, with double layers of vertical and horizontal reinforcement, post-tensioning ducts, and blockouts for openings in the walls. It is known that wood formwork is not conducive to good transmission of form vibration into the concrete, but a constraint to use only steel forms was judged to be too costly. Other conditions for the pontoon construction, such as minimal use of vertical construction joints, and possible multiple construction sites, meant that economical reuse of more costly steel forms might not be possible. However, it was required that the design of the formwork produce the stiffness needed to transmit vibration and remain serviceable throughout construction.

Provisions in the specifications were made to allow the contractor to drop concrete more than 1525 mm in the walls, if it could be shown that segregation did not occur, and that dense, impermeable concrete could be produced. This economic and performance benefit was, of course, one of the objectives of WJE in designing the high performance LVM concrete mix. The 1525 mm restriction in the WSDOT Standard Specifications could be waived as a result of a more cohesive concrete and the use of external form vibration.

Water tightness of the pontoons was a primary concern of the designers when specifying construction joint locations and details. The combined experience of WJE and WSDOT signified that, when leaks occur, they usually are found at construction joints and around wall penetrations. Bond across construction joints by chemical adhesion and mechanical interlock was enhanced by requiring roughening of the hardened surfaces. The contractor achieved this by performing a high-pressure water blast of the surfaces before the concrete was too strong to resist removal of laitance, while still allowing roughening without weakening aggregate bond. Thorough compaction of concrete against the joints was emphasized, with a 600 mm height restriction of the bottom lift of wall placements. Specifications further limited the number of vertical and horizontal joints to reduce sources of potential leakage.

The LVM mix design produced concrete that yielded little or no bleed water to the surface. As a result, the top surface of flatwork, such as slabs, was difficult to finish to a closed surface, free of honeycombing. The lack of bleed water may result in plastic shrinkage cracking: the short, discontinuous cracks that occur before final set of the concrete slab. Both of these problems were mitigated by fog-spraying the surface immediately after the concrete was placed and screeded, and just before finishing and application of curing. It is imperative that a true, fine mist spray be used. The special provisions for this project permitted the fog spray, but did not require it.

Positive, moist curing of exposed concrete surfaces was stressed in the specifications in order to minimize permeability and cracking. Ponding of the slabs and top surfaces of the formed concrete was required. The outside wall forms were left in place for a minimum of 14 days after initial set had occurred. After inside form removal, walls were sprayed with curing compound. The very low water-cement ratio concrete used in the pontoons must be cured with water to avoid desiccation and disruption of the integrity of the internal cement paste.

5. CONCRETE CONSISTENCY AND PLACEMENT

The general arrangement at the time of placing concrete in the first pontoon in the Seattle graving dock is shown in Fig. 1. Previous experience in the construction of an oil exploration platform built in Japan made it clear that placement of concrete in the typical deep walls was best done by using modified tremie systems. The tremie is lowered into the top of the wall and slowly withdrawn as the level of concrete rises. The upper lifts of concrete can be easily placed from the top of the wall without the tremie. The outer layer of vertical reinforcement is spaced to allow room for the tremie insertion. The LVM bridge contractor followed this suggestion and used a modified structural tubing attached to the concrete pump hose to place concrete in all the remaining walls and bulkheads (see Figs. 2 and 3).

The lower lifts of concrete were placed with a slump of 175 to 240 mm, with best results at 225 mm or above. As the concrete level neared the top of the wall forms, the slump was decreased to 100 to 125 mm. Finally, a slump of 75 mm was used on the top lift. The lower slump eliminated practically all bleed water and laitance at the top and allowed earlier joint preparation by water blasting. The latter was possible because of less retardation of concrete set. Lower slump was achieved by substantially reducing the amount of high range water reducer. As the pontoon construction progressed, slump was easily adjusted and controlled to fit the placement needs; lower slumps were used where high slump was not required.

6. COMPRESSIVE STRENGTH TEST RESULTS

As expected, the achievement of the design compressive strength of was never a problem during construction. The average compressive strength of all tests at 28 days was 72 MPa. No attempt was made to establish strength beyond 28 days, but that determination was made during the LVM Mix Design Development testing. Those tests showed a strength gain of about 15 percent from 28 to 90 days, for mixes similar to the final LVM mix. Thus, the 90 day strength for the pontoon construction is estimated to average 83 MPa.

7. RAPID CHLORIDE PERMEABILITY TEST RESULTS

The AASHTO T-277 test for Rapid Chloride Permeability was conducted at a frequency of about one test for each 2620 cubic meters of concrete placed. The results were informational only. They were not used as a basis for acceptance of concrete. At least two specimens from each sampling were tested at 28 days and, in many cases, other specimens from the same sample were tested at 56 days and 90 days, and a few at 7 or 14 days. A statistical summary of the tests is shown in the following table:

STATISTIC	28d	56d	90d
No. of Tests	109	51	22
Average Perm, Coulombs	1327	785	577
Standard Deviation	523	230	135
Range of Results	517 - 2784	368 - 1608	310 - 804

Table 2 Statistical Summary of Rapid Chloride Permeability Test Results

It can be seen that the permeability is well below the targeted maximum of 1000 coulombs at 56 days, thought to represent concrete with excellent resistance to chloride intrusion. There is significant reduction of permeability with age, as can be seen in the table. Reference 3 in the Appendix describes some extensive research on permeability of various concretes, some containing silica fume and fly ash. It can be seen from Table 1 in the reference that rapid permeability results were 570, 340, and 168 coulombs at 28, 90, and 365 days, respectively. Those values are slightly lower than those on this project, but are about the same as those obtained during the mix development phase.

8. SUMMARY AND RECOMMENDATIONS

The risk of proceeding into construction with concrete specifications that had no history of previous performance on WSDOT projects was minimized by undertaking a rather extensive development program. Pre-construction testing further reduced the potential for major problems. However, those efforts would have been wasted had the contractor and concrete supplier not been willing to extend themselves and make these different approaches work. The successful conclusion of the pontoon construction was greatly assisted by the cooperative efforts of WSDOT, the contractor, the concrete supplier and WJE. Post-construction input from all of these parties has confirmed that the high performance concrete, external vibration, and other mitigative construction methods, were proven to be necessary.

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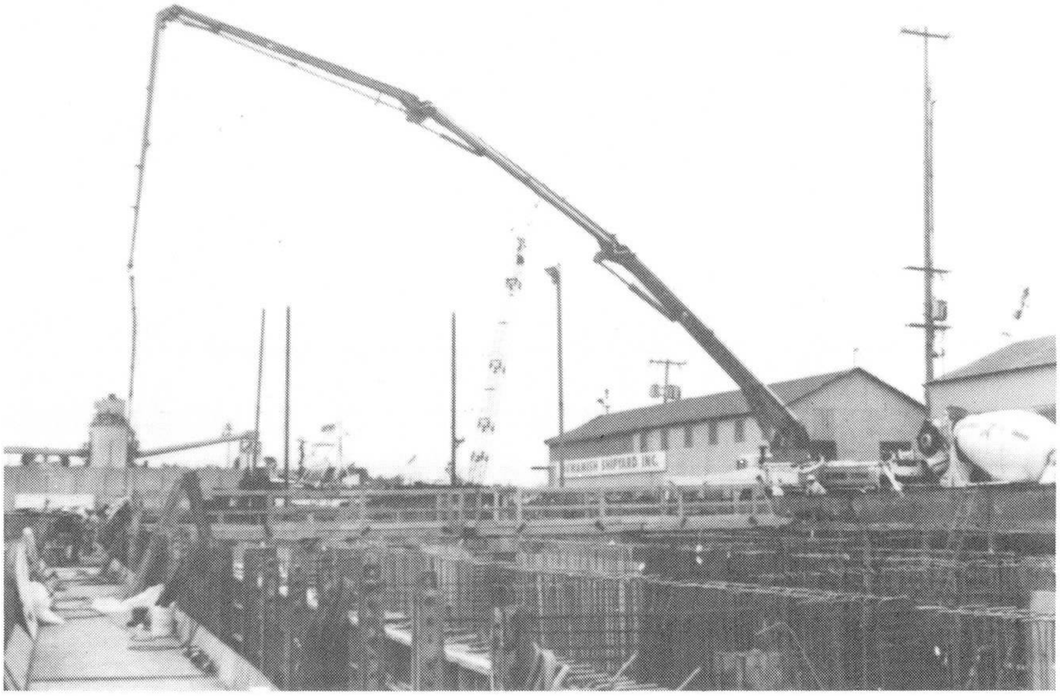

Fig. 1 - Seattle graving dock layout for concrete placement.

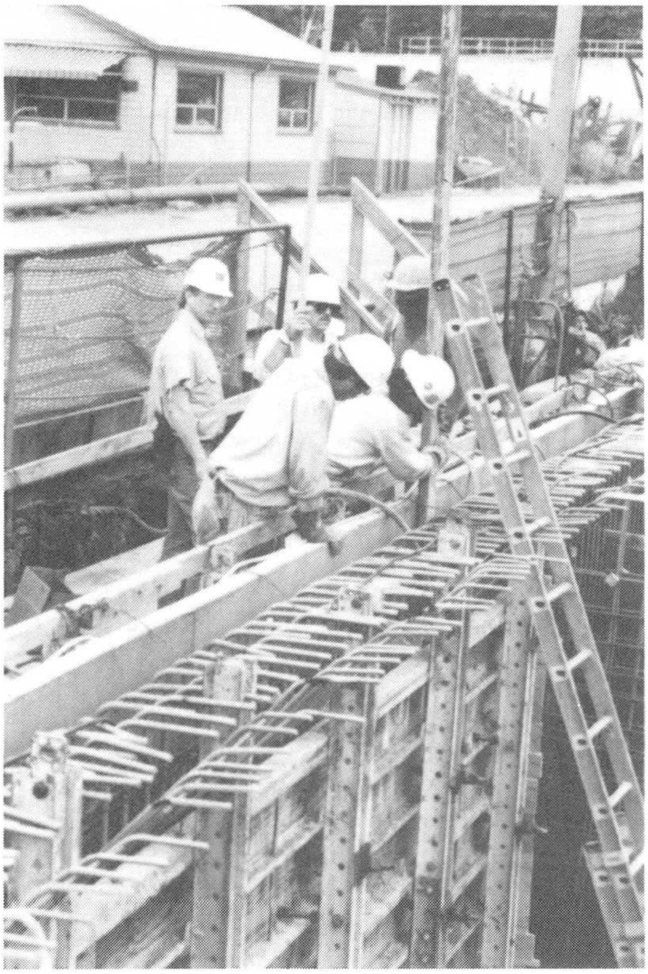

Fig. 2 - Closeup of tremie tube inserted in wall form next to form surface. Note internal vibrator on right.

Concrete Durability in the Arabian Gulf Region

Durabilité du béton dans la région du Golfe Arabique

Betondauerhaftigkeit im Gebiet des Arabischen Golfes

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SUMMARY

Deterioration of reinforced concrete structures in the Arabian Gulf region is an urgent problem that structural engineers must confront. Although durability of concrete is now recognised to be as important as its strength, structural engineers need to be more aware and better equipped for the challenge. There is a gap in design that must be overcome by developing national codes that reflect local conditions. The cooperation between materials specialists and structural engineers is essential to achieve this important goal. The paper highlights important elements in the process of "design and construct for durability."

RÉSUMÉ

La détérioration des structures en béton armé dans la région du Golfe Arabique est un problème urgent posé aux ingénieurs civils. La durabilité du béton est un élément aussi important que sa résistance, et les ingénieurs doivent en être conscients, et être équipés en conséquence. Des normes nationales doivent être établies, prenant en compte les conditions locales. La coopération entre les spécialistes en matériaux et les ingénieurs civils est essentielle. L'article met en relief les éléments essentiels dans le processus du projet et construction en vue d'une d'une bonne durabilité.

ZUSAMMENFASSUNG

Der Verfall von Stahlbetonbauten in den Ländern am Arabischen Golf ist ein akutes Problem für Bauingenieure. Obwohl die Dauerhaftigkeit des Betons heute weltweit für ebenso wichtig wie dessen Festigkeit gehalten wird, müssen die konstruktiven Ingenieure die Herausforderung bewusster und besser gerüstet annehmen. Die Entwicklung von nationalen Bauvorschriften gemäss den lokalen Bedingungen sollte die Lücken im Entwurf schliessen. Dieses wichtige Ziel ist nur durch Zusammenarbeit von Werkstoff- und Bauingenieuren zu erreichen. Dieser Beitrag beleuchtet wesentliche Punkte für ein dauerhaftes Entwerfen und Ausführen.

1 . INTRODUCTION

The durability aspects in reinforced concrete have gained a lot of significance lately.Worldwide attention has been reflected in recent issues of codes of practice [1-5] . One of the areas of the world that is in dire need of such attention is the Middle East and particularly the Gulf area. Saudi Arabia is one of the Gulf states; however it is a vast country that includes within its borders a wide variety of environments [6]. The country needs more than just adopting a foreign code of practice to be followed by engineers.The average structural engineer who is practicing in this area of the world , in the authors opinion , is neither aware of nor convinced with the problem . Those who are aware are not equipped with the knowledge necessary to enable them to design for durability . Table 1 shows values of salts and compressive strength of cores extracted from an office building in a coastal city on the Red Sea . The values prove without doubt that the designer and construction engineer had no idea about durability aspects. The building showed extensive corrosion and concrete deterioration only three years after it was handed over to the owner in 1991 . Design for durability is an attainable objective [7,8] ; it can become more and more possible through concerted efforts between materials scientists and structural engineers .

2. DURABILITY

2.1 Introduction to Durability :

ACI-201[9] defines the durability of hydraulic Portland cement concrete as its ability to resist weathering action , chemical attack , abrasion or any other process of deterioration ; i.e. durable concrete will retain its original form , quality and serviceability when exposed to its environment. Therefore concrete deterioration is not one phenomenon ; it is caused by many mechanisms. P.K Mehta [10] classifies the mechanisms into two major categories .

(a) Physical :

i) - Surface wear : abrasion , erosion and cavitation .

ii) - Cracking due to volume changes , structural loading and exposure to temperature extremes .

(b) Chemical :

i) -Soft water attack on calcium hydroxide .

ii) - Exchange reactions between aggressive fluids and components of hardened concrete paste .

iii)- Reactions involving formation of expansive products such as steel corrosion which is the most prominent cause in Saudi Arabia for concrete deterioration especially in coastal areas , and sulphates attack of concrete which also forms a considerable ratio of concrete problems in this country .

2.2 Awareness of The Problem :

There are many engineers who are under two major misconceptions ; first : strong concrete is durable , and second : using sulphate resisting cement is a guarantee against deterioration .The first concept may have some truth whereas the second is opposite in some cases such as when concrete is exposed to chlorides it makes the rate of deterioration faster . Misconceptions like this linger on while accumulation of scientific rational data continues to develop . The literature is presently full of papers ,essays , research and some books on the subject of durability . There is no doubt that materials specialists have done great efforts in this field but there is still a lot more to be done . However , there is an unintentional gap that separates them from structural engineers . In a recent conference in France on durability [11] that the author attended , he was one of five structural design engineers among close to three hundred materials specialists. This gap is perhaps even wider in the Middle East and the Gulf . The knowledge acquired by materials specialists over the past two decades are not anywhere near fully utilized by structural engineers. The average design engineer needs three important things ; first : to be convinced of the necessity of taking measures to promote durability of concrete , second : to be educated of what should be done,thirdly : he should be supported by local codes that guide him in design . The third objective must be the ultimate goal for all those who are working in the field of design and construction of concrete .

The implementation of durability measures in construction must rise to the level of correct design to achieve the desired effect .The site engineer must have the same awareness for he is the one who will make things work , together with the construction workers .All the above facts are axioms that are perhaps known in many parts of the world ; the lack of awareness in this area together with extremely aggressive environment have created problems such as those shown in Fig.1.

2.3 Durability - Responsibility of the Structural Engineer :

In the process of designing a building the structural engineer develops a very close cooperation with the geotechnical engineer . In durability related design a close relationship must develop between the structural engineer and the materials scientist . In Saudi Arabia and many parts of the world this relation is still in its infancy . Owners (clients) look at the structural engineer as the professional who is responsible about the structure's integrity , strength , serviceability and durability because he is the one that coordinates all design efforts . The structural engineer from his point of view must be able to provide his client with a design that he can claim to possess the above mentioned qualities .Therefore, it must be clear to the structural engineer in this area of the world that durability is ultimately his responsibility ; after the project leaves his desk nobody will think of durability unless he specifies the measures to be carried out . If not equipped to design for durability he must seek support from materials scientists . This is required intensively until codes are more clear , elaborate and specific about requirements for durability . This is also essential until local materials and local experiences are well established in the behavior related to durability .

3. DESIGN FOR DURABILITY

This section includes the steps that structural engineers should follow to have a rational strategy in designing for durability . For space constraints , it will be limited to the design for protection against the corrosion of reinforcing bars , since it is the most common form of building deterioration related to durability in Saudi Arabia and is number one cause in the Eastern Region[6]. However, the same logic may be applied for the other forms of deterioration.

3.1 Predesign Information

i) - Identify the environment , the possible cause of deterioration and the mechanisms involved , e.g. the distance from the sea water .. etc .
ii) - Recognize construction technology available in the region and affordable by the owner; a form of quality control and assurance must be applied .
iii)- Establish desired service life of the structure and level of maintenance affordable .
iv)- Evaluate the economics of the durability measures .
v) - Secure the help of materials scientist for consultations during design and construction .
vi)- Discuss durability measures with contractors who have good reputation in the field because the cooperation of the contractor is very important in the implementation phase.

3.2 Environment and Deterioration Mechanisms :

Characterizing the environment has recently been accepted as two-fold : macro-environment which deals with the whole region or area where the structure is built and micro-environment where the aggressivity of the environment around every single member or part of member in the structure is determined . Fig (2) depicts two examples: the first is a pier of a causeway where five micro-environments can be identified ; these are underwater , tidal , splash water , spray water and atmosphere . The second example of Fig (2) is a building column in which four zones of micro-environments are identified . This architectural form is not the best as far as durability is concerned ; it is much better in coastal environments to have columns indoors . Recognizing the possible mechanisms of deterioration and transportation in each member of the structure is extremely important . Steel corrosion mechanisms according to P.Schiessl [12] could be one of the following :

i) - Carbonation which causes reduction in the concrete cover alkalinty (e.g. industrial environment).
ii) - Chloride ions acting as a catalyst in the electro-chemical reaction that forms iron-oxides or rust (e.g coastal environment) .
iii)- Ingress of oxygen and/or humidity to the steel through cracking or porous concrete .

Three transport mechanisms are also recognised ; i.e diffusion , permeation and capillary action .
Identifing this information is necessary for the design to counteract the cause or provide needed protection .

3.3 Concrete Properties :

The following list includes the properties that provide high performance concrete that is capable of withstanding exposure to severe environments :

i) - Low permeability provided by low porosity structure of cement paste and sound aggregates .
ii) - Low heat of hydration that avoids the formation of microcracking during early ages of concrete.
iii) - Low water cementitious material ratio.
iv) - High early strength and continued development of strength .
v) - Shrinkage control at early ages until enough tensile strength is developed .
vi) - Low plastic shrinkage .
vii)- Provide factors that promote workability and control of slump loss .

3.4 Design and Construction Measures :

In order to achieve the above properties the structural engineer has, often, many options to choose from . However there are five items that he must provide ; these are :

i) - Use of correct amount and type of cementitious material and low water /cement ratio .
ii) - Provide enough concrete cover to prevent oxygen , water and other chemicals from reaching steel . He can follow as guideline international codes [1-5] until local codes are developed .
iii) - Ensure adequate compaction of concrete to provide dense concrete , adequate bond with reinforcement and eliminate entrapped air . Thus , promoting steel protection by concrete alkalinity .
iv) - Perform necessary curing .
v) - In cases of very severe exposure provide pore blocking inhibitors mixed in concrete or applied on the surface to minimize ingress of moisture and other chemicals through concrete.

3.5 Steel protection options :

Two of the methods of steel protection are epoxy or zinc coating . These methods are often reckoned as second lines of defense; their use must be weighed against maintenance cost if the first lines of defense fail ; namely , the impermeable , dense, alkaline concrete cover . In other instances zinc coating is essential in repair jobs where chlorides are involved .

3.6 Choices of Cementitious materials :

The use of slag (ASTM C989) or Fly-ash (ASTM C 618) may be considered in order to improve qualities of concrete . Silica fume can also be used with high range water reducers (Super plasticizers) . The local unavailability of these materials can affect the construction cost .They may be used only in parts of structure that are more prone to deterioration by corrosion such as the lower parts of exterior columns, concrete exposed to environment .. etc .

3.7 Economics :

The engineer must be aware of both macro-and micro - environments because durability measures are very expensive and should be applied only where needed . For example , in the building columns of Fig. (2.b), durability measures that are applied on different zones of the column are different ; underground part could be protected by coating , sulphur resistant cement and enhanced cover , whereas the lower part of the ground floor may have some pozzolan such as silica-fume as well as hydrophobic coating protection. Top floors do not need any particular protection . In Saudi Arabia , one of the areas that need special measures in most environments is the lower part of columns which are usually more exposed to weathering [13]. In this context it is worthy to mention that owners of new buildings must be educated not to go always for the " lowest bid " , they must know how to evaluate the bids with respect to durability, quality control , quality assurance and the anticipated maintenance cost required to extend the life of a deteriorated structure . A buyer of an existing structure must seek an evaluation of the structure vis-à-vis its expected life .

3.8 Quality Control - Construction :

Durability is the hostage of execution . Quality control and quality assurance are very important in achieving the properties of the structure planned by designer . Without proper construction methods durability will never be attained .The role of the construction supervisor engineer is very important .

4. CONCLUSIONS

The structural engineer is responsible to the client for the integrity , strength , serviceability and durability of the structure. In Saudi Arabia and other parts of the world durability has assumed importance in recent years due to many failures especially in the Arabian Gulf region . The structural engineer is urged by the author to assume the durability responsibility and invoke all available resources to design for assumed life of the structure . He should seek the cooperation of materials scientists as well as the contractor to provide the proper design and the correct implementation of durability measures in construction. Given in the paper are elements that engineers should be aware of to confort the problem of reinforced concrete deterioration .

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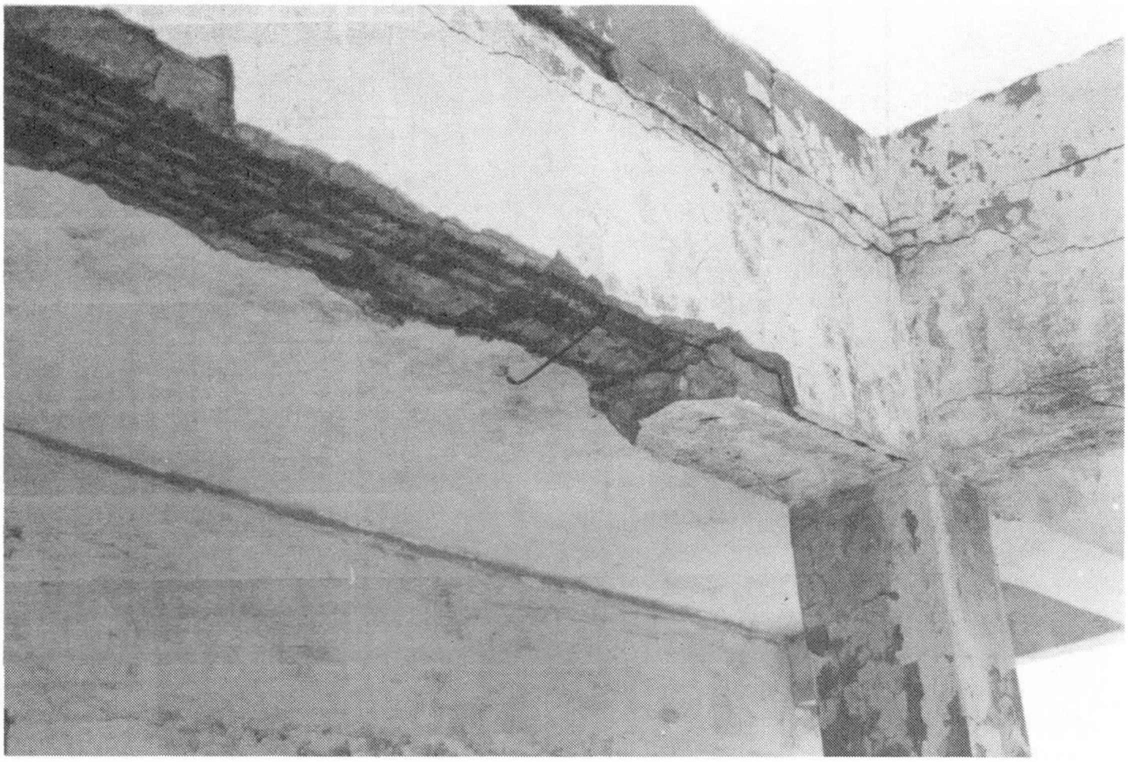


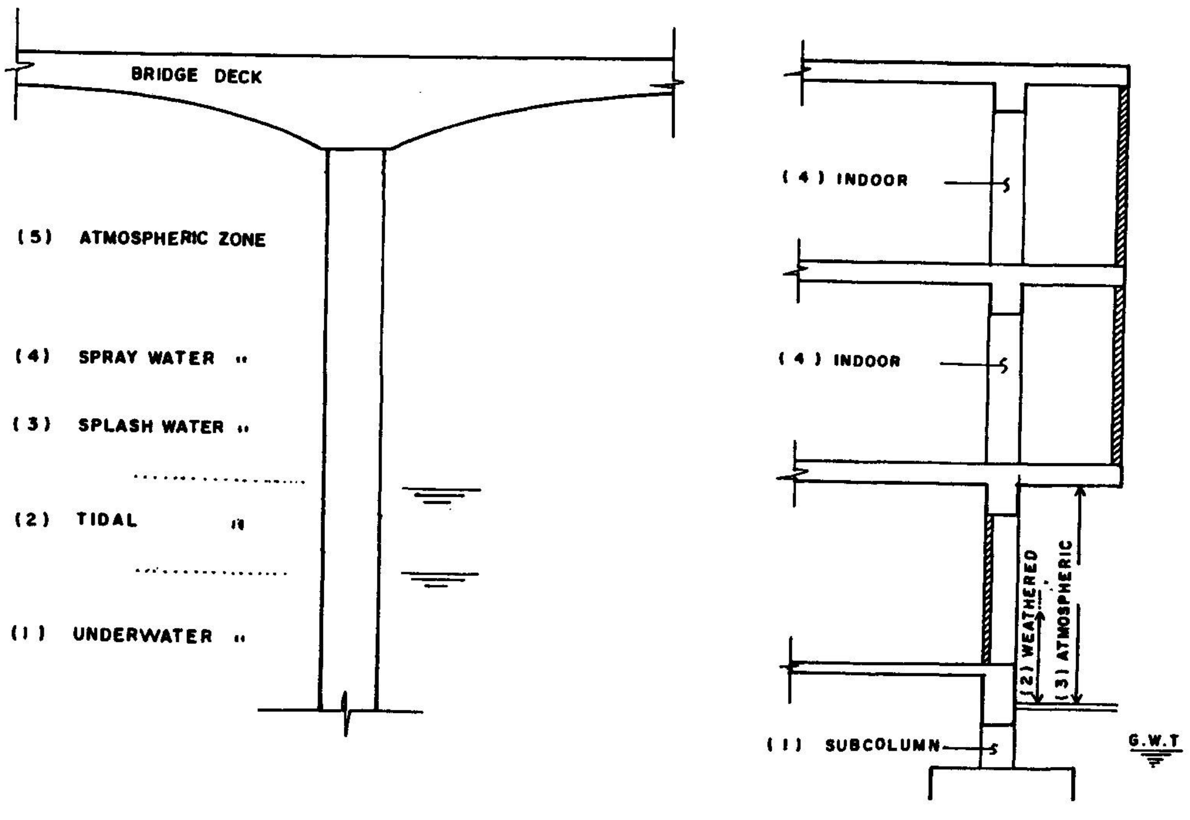
Fig. 1. DETERIORATED STRUCTURE IN A GULF CITY

TABLE 1 :- CHEMICAL AND CORE TEST RESULTS
OFFICE BUILDING - CITY OF WAJH - RED SEA COAST

LOCATION → / PROPS ↓	REQUIRED	GRD.FLR.COLS.	1st FLR. COLS.	2nd FLR.COLS.	FIRST FLOOR		SECOND FLOOR	
		C1 C4 C7 C2 C5 C8 C3 C6 C9	C11 C13 C10 C12 C14	C16 C18 C17 C19 C20	SLAB S1 TO 55	BMS B1 B2 B3	SLAB 56 TO 510	BMS B4 TO B6
CEMENT CONTENT KN/M3	3.5	3.53 2.90 2.44	X 1.85 2.32	– 1.99	2.78	3.46	2.47	2.68
CL/CEMENT %	≤ 0.4%	1.021 0.616 1.134	X 1.384 2.012	– 2.434	0.708	1.106	2.035	1.414
ALKALINITY pH	12 - 13	12.2 12.0 11.9	X 11.5 11.5	– 12.0	12.3	12.5	12.2	12.2
CORE TEST CUBE f_c' N/mm2 (MPa)	25 LOW	28.9 19.4 16.0 30.4 23.9 16.2 30.5 24.2 18.6	0.0* 11.3 X 19.8	0.0 9.5 0.0 19.3 0.0	14.9 30.7 27.9 34.7 28.6	12.9 16.2 27.5	16.7 30.3 22.0 31.3 25.4	8.1 11.3 28.1

X Sample was not extracted .
* Sample crumbled during extraction .

REMARKS :- 1st AND 2nd FLOOR COLUMNS. : WEAK CONCRETE .
– CHLORIDE / CEMENT RATIO GREATER THAN ALLOWABLE .
– CEMENT CONTENT VERY LOW FOR DURABILITY REQUIREMENTS .



(2.a) Pier of a Causeway **(2.b) A column in a building**

Fig.2 MICRO-ENVIRONMENT PRACTICAL EXAMPLES

Surface Treatments for Concrete Quantifying the Improvement

Traitement de surface du béton et mesure de l'amélioration

Oberflächenbehandlung von Beton mit messbaren Verbesserungen

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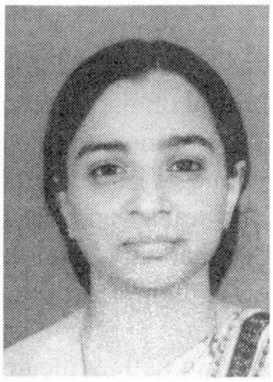

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SUMMARY

An experimental programme to investigate the influence of pore-lining surface treatments on new concrete is described. The focus is on the effect on chloride ingress and time to corrosion of embedded reinforcement. The results show that the characteristics of any concrete treated with a pore liner are considerably better than for a good quality concrete with no surface treatment and are almost independent of the concrete itself.

RÉSUMÉ

L'article décrit un programme de recherche sur l'influence d'un traitement de surface par remplissage et lissage des pores de bétons neufs. Il fait le point sur les effets du temps et de la diffusion des chlorides sur les armatures enrobées. Les résultats montrent que les caractéristiques de n'importe quel béton subissant un tel traitement sont considérablement meilleures que celles d'un béton de bonne qualité non-traité. Elles sont également quasiment indépendantes du béton lui-même.

ZUSAMMENFASSUNG

Es wird ein Versuchsprogramm beschrieben, um den Einfluss einer porenauskleidenden Oberflächenbehandlung auf neuen Beton zu untersuchen. Im Mittelpunkt steht die Auswirkung auf die Chloriddiffusion und die Zeitspanne bis zur Korrosion der eingebetteten Bewehrung. Nach den Ergebnissen zu urteilen, ist das Verhalten eines jeden Betons, wenn er mit Porenauskleidung behandelt wurde, bedeutend besser als für einen hochwertigen Beton ohne Oberflächenbehandlung, und zwar ziemlich unabhängig vom eigentlichen Beton.

1. INTRODUCTION

Under ideal conditions concrete will improve in quality with time due to the longer term reactions which produce the cement hydrates. However, the interaction with the environment can cause deterioration of concrete. A low ratio of water to cement and good curing are important factors in reducing future deterioration but in some circumstances even these cannot guarantee long service life. One such case is highway structures where the action of chlorides in deicing salts and, to a lesser extent, the action of freezing and thawing create an extremely aggressive environment. It is increasingly accepted that current design code recommendations for minimum concrete quality and cover to the reinforcement are inadequate and that for chloride ingress no combination of normal concrete and cover will provide a service life normally expected of bridges [1]. This has led to increased use of surface treatments to provide an effective barrier to the ingress of water and chloride ions.

The most commonly used surface treatments for this application are pore liners; in particular silanes and siloxanes. They can be sprayed on the surface and the liquid which enters the pores reacts with the cement matrix, in the presence of moisture, to form a lining to the capillary pores [2, 3]. The major advantage over other treatments is that, while they reduce the penetration of water and water-borne ions, the passage of air and water vapour is not prevented. Consequently, the concrete is still able to "breathe". While the fact that pore liners reduce chloride ingress has been reported elsewhere [4, 5, 6] the variation in their effectiveness for different concrete qualities is less well documented. This paper reports a test programme to study the increase in resistance to chloride ingress and the delay in corrosion initiation due to the application of surface treatments.

2. EXPERIMENTAL PROGRAMME

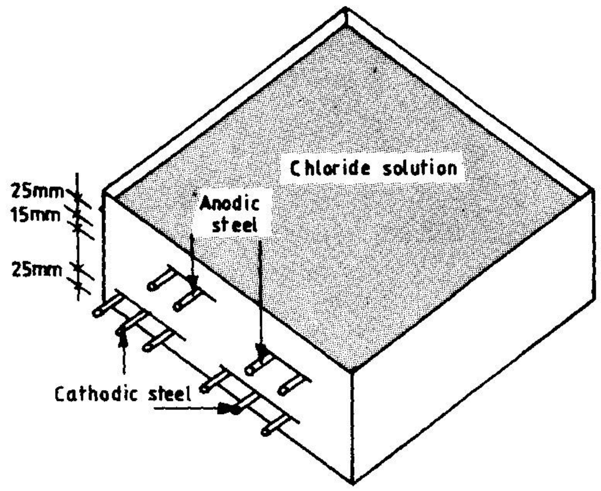
2.1 Test Regime

The ingress of chloride into concrete varies depending on the mechanism by which it is transported [7, 8]. If concrete is subjected to cyclic exposure rather than continuous exposure, the chloride ingress is greater. This is because absorption dominates in the former case whereas in the latter the mechanism is diffusion. In this study, cyclic exposure was adopted because the amount of chloride penetration would be greater within the time scale of the tests and because it is more representative of the conditions which prevail when sodium chloride is used as a deicing salt. The exact ponding cycle chosen was similar to that used by Pfeifer [4]; three days ponding with a 15% sodium chloride solution and four days drying (in this case all at 20°C and 55±5 °C relative humidity).

2.2 Test Samples

Concrete samples were 400 x 400 x 150mm with a 12mm deep reservoir formed on one face to contain the sodium chloride solution. Anodic reinforcing bars were positioned at 25mm and 40mm from the ponded face and cathodic bars near the bottom face (Figure 1). Where the bars projected from the concrete, they were protected with a rich mortar and a tar based corrosion protection coating. A 10Ω resistor was connected across each pair of bars. The sides of the samples were coated with epoxy resin to prevent any contamination via the sides.

The variables investigated in this study were the ratio of water to cement of the concrete and the type of surface treatment. Three concretes with different water to cement ratio (0.45, 0.55 and 0.65) were selected to represent the normal range used in practice. A 20mm maximum size crushed rock was used as coarse aggregate and a natural sand as fine aggregate. In addition to the control



Specimen size	-300x300x150mm
Details of bars	-10mm diameter. M.S.
Cover	-25mm & 40mm

Figure 1. Details of specimens used for the chloride ponding test

samples (ie. those with no surface treatment) four treatments were investigated; 100% silane, 40% silane, silane-siloxane and silane-acrylic. An earlier study had shown that the variation in the sorptivity of concretes treated with silanes of differing concentration was low and so it was decided to include only the extreme values in this study. After curing for 14 days in water and 14 days in air, the specimens were saturated and then conditioned for 3 days at 20 °C and 55±5 rh before application of the surface treatments. This was considered to represent typical conditions under which surface treatments might be applied in the British Isles.

2.3 Test Methods

2.3.1 Chloride content

Samples were drilled from the sides of the specimens at the same depths as the reinforcing bars ie. 25mm and 40mm. The water soluble chloride ion content of the samples was analysed in accordance with BS1881 : Part 124 [9] and expressed as a percentage of the cement content. A set of samples was collected from all samples when the bars at 25mm cover in the control (untreated) samples first showed some indication of corrosion. A second set of samples was obtained after about 30 weeks of ponding and a final set at the end of the 44 week test period.

2.3.2 Half-cell potential

Half-cell potential readings were taken every three weeks throughout the 44 week period using a copper-copper sulphate half-cell. Three points were selected along each of the anodic bars to see if there was any variation in corrosion activity along the length of the bars.

2.3.3 Macrocell corrosion

The voltage across the anodic and cathodic bars was monitored throughout the test period. From this, the macrocell corrosion current was calculated using Ohm's Law. No attempt was made to measure microcell corrosion activity.

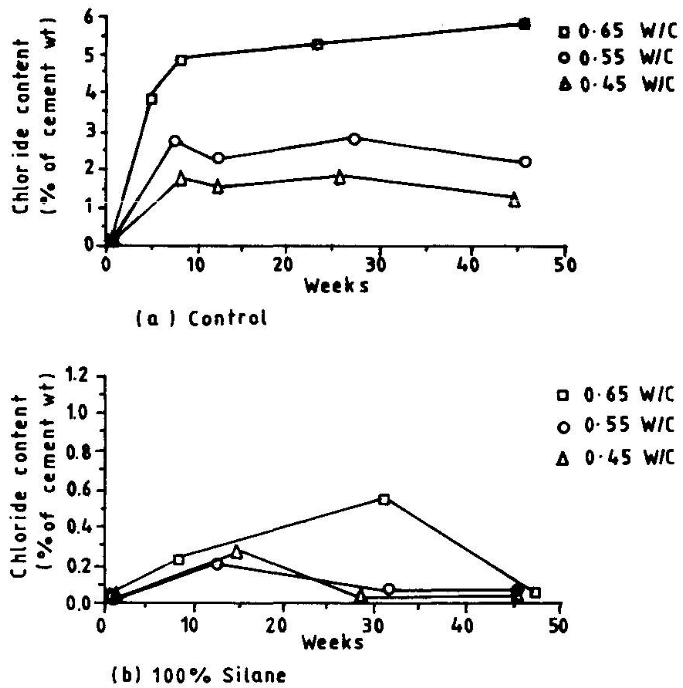


Figure 2. Chloride ingress at 25m depth in cyclic ponding test.

3. CHLORIDE PENETRATION

Figure 2 shows the variation with time of the measured chloride concentrations at 25mm from the surface subjected to ponding. For clarity, only the untreated samples and 100% silane have been included and it should be noted that different scales have been used. It is clear that the chloride concentration seems to level off after about ten cycles. At the end of the test, the ion concentration in the untreated samples was between 1.0% and 5.5%; increasing with increasing water-cement ratio. With 100% silane applied to the surface, the ion contents were considerably reduced. While there is some variability in the results, mainly due to the level of accuracy possible at such low values, the final ion concentration for each of the three concretes was about 0.2%. Similar results were obtained for the concretes treated with 40% silane, silane-siloxane and silane-acrylic.

For the 40mm depth, the trend is similar except that even at the end of the 44 weeks, the chloride ion content in the 0.45 water-cement ratio concrete was no higher for untreated concrete than for treated concrete. These results indicate that while surface treatments are effective on all concretes, the improvement gained is much greater on high water-cement ratio concretes. In fact, there is evidence that after treatment all concretes perform almost equally.

4. CORROSION ACTIVITY

For the control slabs, there was evidence of corrosion activity within the first 10 weeks. Figure 3(a) shows the half-cell and corrosion current readings for 0.55 water/cement ratio concrete. At the end of the test period, half-cell potentials were between -450mV and -600mV for all three concretes. The corresponding corrosion currents were -0.02mA to 0.20mA. At the end of the test, the samples

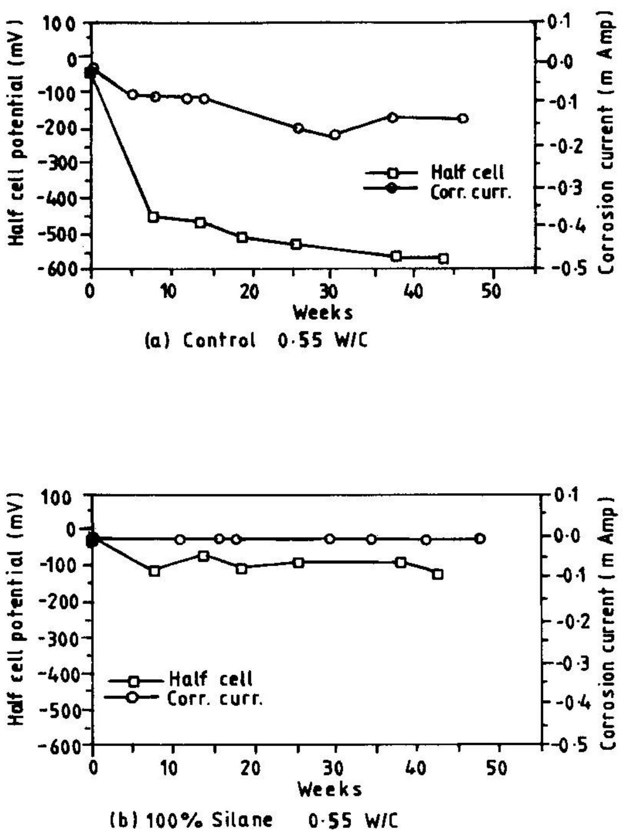


Figure 3. Corrosion at 25mm cover in cyclic ponding test

were broken and the bars inspected for rusting. This confirmed the above result with rusting evident on all the bars at 25mm cover in the untreated specimens. In fact, except in the case of the 0.45 water-cement ratio concrete, the bars at 40mm cover had some rust on the top surface; also confirming the potential and corrosion current readings.

The results for the 0.55 water/cement ratio concrete treated with 100% silane are shown in Figure 3(b). A similar trend was obtained for the 0.45 water/cement ratio concrete, but for the 0.65 water/cement ratio, there was a significant jump in the half cell potential to -500mV after 30 weeks [10]. The potentials and currents were borne out by the visual inspection at the end of the test with corrosion visible only on the reinforcing bars in the 0.65 water-cement ratio specimen. Two other surface treated specimens exhibited corrosion; the 0.65 water-cement ratio concrete with silane-acrylic and the 0.45 water-cement ratio with silane-siloxane. There is no clear explanation for why corrosion occurred in these particular specimens, but from the overall picture it can be seen that with surface treatments, the instances of corrosion within the test period were reduced; from 100% for untreated specimens to 25% among the treated specimens. However, even when corrosion activity occurred with surface treatments, the time lapse until it took place was greater; 20-30 weeks for the treated specimens compared to 10 weeks for those untreated.

For bars at 40mm cover, there was corrosion in the untreated specimens using 0.55 and 0.65 water-cement ratio concrete after about 20 weeks. In none of the treated specimens was there evidence of corrosion of the bars at this cover within the 44 week period.

5. CONCLUSIONS

From this study, it is possible to draw the following conclusions :

(1) Chloride ingress is reduced considerably by the application of surface treatments to new concrete. For the surface treatments investigated, there was no significant difference in their performance in relation to chloride ingress.

(2) The resulting resistance to chloride penetration is generally independent of the quality of the concrete.

(3) Corrosion of reinforcement due to chloride ingress is delayed due to the application of a surface treatment though no one treatment performed better than the others. For the ponding cycle used and reinforcement at 25mm cover the time to the start of corrosion was more than doubled.

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Effect of a Hydrophobic Agent as Repair Material for Concrete Structures

Produit hydrophobe pour la réparation de structures en béton

Wasserabstossendes Mittel zur Reparatur von Betontragwerken

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SUMMARY

Recently, many kinds of surface treatments have been applied to repair concrete structures deteriorated due to alkali-silica reaction and/or reinforcement corrosion. Among those materials used for surface treatments, silane is a typical hydrophobic impregnant to control water existing in concrete. However, since silane includes many kinds of branches which have various properties according to their molecular structures, it is necessary to identify which one has the best hydrophobic property. In this study, the effect of the molecular size, the type and the number of alkoxyl groups of silane on the hydrophobicity of concrete were investigated.

RÉSUMÉ

De nombreuses méthodes de traitement de surface ont été utilisées tout récemment pour réparer les dommages occasionnés au béton par une réaction de silicate alcalin et/ou par une corrosion des armatures. Parmi ceux-ci, le silan est un produit d'imprégnation typiquement hydrophobe, destiné à contrôler la teneur en eau du béton. Il existe toutefois de nombreuses combinaisons de ce produit ayant des propriétés différentes, en fonction de la structure moléculaire, ce qui implique de devoir trouver celle qui possède le meil-leur pouvoir hydrophobe. Les auteurs présentent ici une étude de la taille des molécules, du type et du nombre de groupes d'alcoxydes contenus dans le silan et qui ont une in-fluence sur le caractère hydrophobe du béton.

ZUSAMMENFASSUNG

In jüngster Zeit wurden vielerlei Oberflächenbehandlungen zur Reparatur von Betonschäden infolge Alkali-Silikat-Reaktion oder Bewehrungskorrosion angewendet. Von diesen ist Silan ein typisches hydrophobes Imprägnierungsmittel, um den Wassergehalt im Beton zu begrenzen. Allerdings gibt es viele Spielarten mit unterschiedlichen Eigenschaften, je nach Molekularstruktur, so dass jene mit der besten Wasserabstossung herausgefunden werden muss. Berichtet wird über eine Studie zum Einfluss der Molekülgrösse, der Art und Anzahl von Alkoxylgruppen im Silane auf die Hydrophobität von Beton.

1. Introduction

A number of cases of premature deterioration of concrete structures caused by alkali-silica reaction and/or chloride induced corrosion of reinforcing steel have been reported recently. It is well known that water plays one of the most important roles in these deterioration mechanism. Taking this into consideration, in order to avoid the deterioration, many kinds of surface treatments which can control water content in concrete are applied to concrete structures. Such surface treatments can be classified into two categories from the viewpoint of how to control the water in concrete. One includes the treatments which permit no water to ingress into concrete, and no water to get out from concrete in the same way. The other includes the hydrophobic treatments which permit liquid phase water to penetrate into concrete rather little, but can make vapor phase water in concrete get out on the contrary. Since the former systems may cause deterioration by the water fixed in concrete, the latter have got focused as superior methods [1].

In the latter systems, silanes are commonly used as typical hydrophobic impregnants. This paper describes the hydrophobic surface treatments for concrete using some types of silanes adopted in repair works of concrete structures. Silanes are silicone-based products of low molecular weight and used as the alkylalkoxylsilanes [2]. When a silane is applied to concrete surface, a series of chemical reactions between the silane and the silicate structure of concrete occurs in two steps, which are hydrolysis and condensation. During the hydrolysis, the moisture provided from concrete produces unstable silanol molecules. During the condensation, the unstable silanol molecules shake hands with available hydroxyl groups in the silicate structure and some crosslinkings occur. In this way, the silane-treated concrete becomes water repellent [3].

2. Outline of experiment

2.1 Molecular structures of silanes

By changing the kind and number of the alkyl and alkoxyl groups of silanes,nine kinds of silanes indicated in Table 1 were prepared. These were used as 1mol solutions of isopropyl alcohol.

Name	Molecular formula (alkyl) (alkoxyl)	Molecular weight	Note
dimethyldimethoxy silane	$(CH_3)_2Si(OCH_3)_2$	120	two alkoxyls
methyltrimethoxy silane	$CH_3Si(OCH_3)_3$	136	alkoxyl is methoxy.
ethyltrimethoxy silane	$C_2H_5Si(OCH_3)_3$	150	
iso-butyltrimethoxy silane	$C_4H_9Si(OCH_3)_3$	178	
n-octyltrimethoxy silane	$C_8H_{17}Si(OCH_3)_3$	234	
n-deciletrimethoxy silane	$C_{10}H_{21}Si(OCH_3)_3$	262	
n-octadeciletrimethoxy silane	$C_{18}H_{37}Si(OCH_3)_3$	374	
methyltriethoxy silane	$CH_3Si(C_2H_5)_3$	178	alkoxyl is ethoxy.
n-octadeciletriethoxy silane	$C_{18}H_{37}Si(C_2H_5)_3$	416	

Table 1 Silanes and their molecular structures

2.2 Preliminary experiment

Preliminary experiment was conducted to select from the nine kinds of silane, those used in the main experiment. The specimens were small concrete prisms (40x40x160mm) with non-reactive aggregate. After being cured in water for three months, they were dried in air for a week and then impregnated with the silanes. After two days from the impregnation, the specimens were placed under four different conditions - indoors, underwater, in the dry and wet chamber (20℃, 60%RH, 12Hr ⇔ 40℃ 100%RH, 12Hr), or outdoors. The hydrophobic performance of each silane under each condition was evaluated from weight changes of the specimens.

2.3 Main experiment

The specimens were concrete prisms (100x100x400mm). Table 2 shows the types of concrete, silanes, and the conditions. For corrosion series, three deformed bars (300mm) were embedded in the specimens. After being cured at 20℃, 80%RH for two weeks, each specimen got impregnated with one of the four silanes selected in the previous test, subsequently were placed in each condition. During this experiment, weight changes, strain and halfcell potential were measured.

concrete	non-reactive without chloride, non-reactive with chloride reactive without chloride, reactive with chloride
silane	non-treatment, 234,262,374,416 (molecular weight)
condition	outdoors, dry and wet chamber, partially immersing in chloride solution (NaCl: 3.13%)

Table 2 Factors for main experiment

3. Results and Discussions

3.1 Preliminary experiment

From the viewpoint of controlling the ingress of water into concrete from outside, the feature to make no water penetrate into concrete is needed. On the contrary, from the viewpoint of making water in concrete get out, the ability to permit water to move out of concrete is expected. The former is mainly related with liquid phase water, and the latter with vapor phase water. As the water in concrete has a great influence both on its expansion due to alkali silica reaction and on the rate of reinforcement corrosion, surface treatments are expected to have the ability not only to permit little liquid phase water to penetrate into concrete but also to permit a lot of vapor phase water in concrete to get out. As the weight changes of indoor specimens and those of underwater specimens can be regarded as indexes of "water vapor permeability" and "water liquid permeability" respectively, a larger ratio of "water vapor permeability/water liquid permeability" corresponds to better hydrophobic performance.

3.1.1 Water vapor permeability

Fig.1 shows the relationship between molecular weight and water vapor permeability. After 9 days of exposure, the silanes of smaller molecular weight in methoxy series had the larger water vapor permeability. The same tendency was observed in ethoxy series as well. The tendency was still observed after 30 days of exposure, although the influence of molecular weight on the water vapor permeability was reduced to some extent. These results indicate that the water vapor permeability of the silanes of smaller molecular weight was generally larger than that of larger ones during the above period.

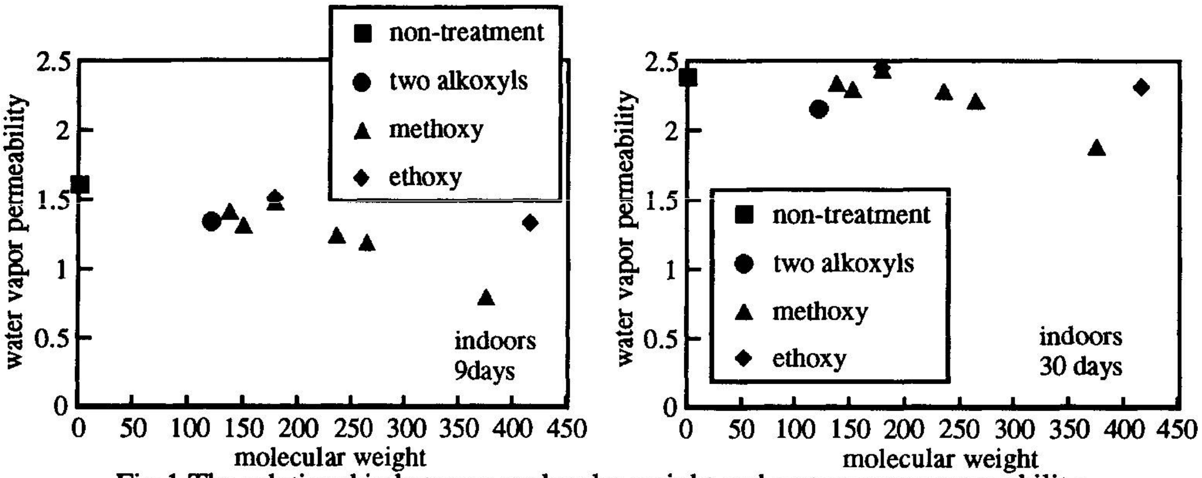

Fig.1 The relationship between molecular weight and water vapor permeability

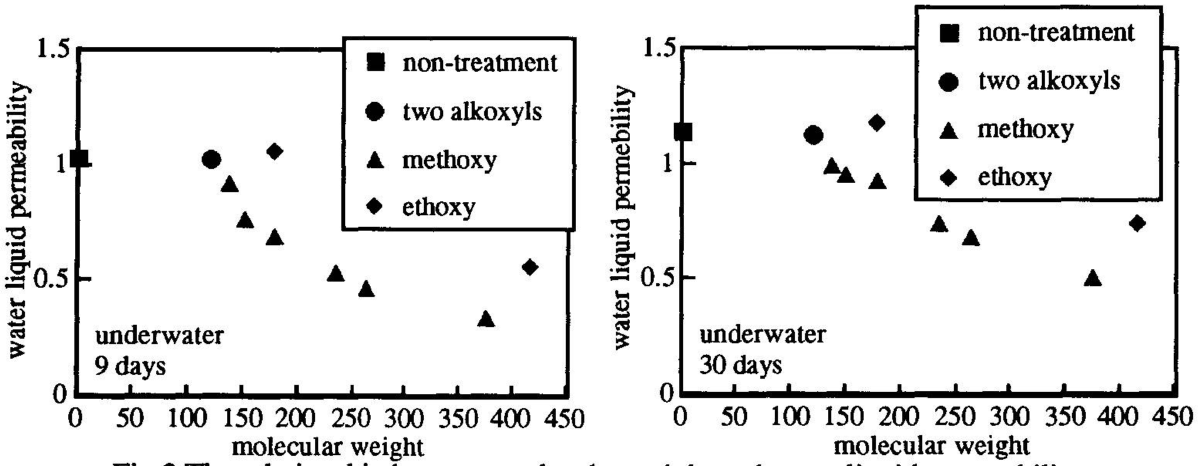

Fig.2 The relationship between molecular weight and water liquid permeability

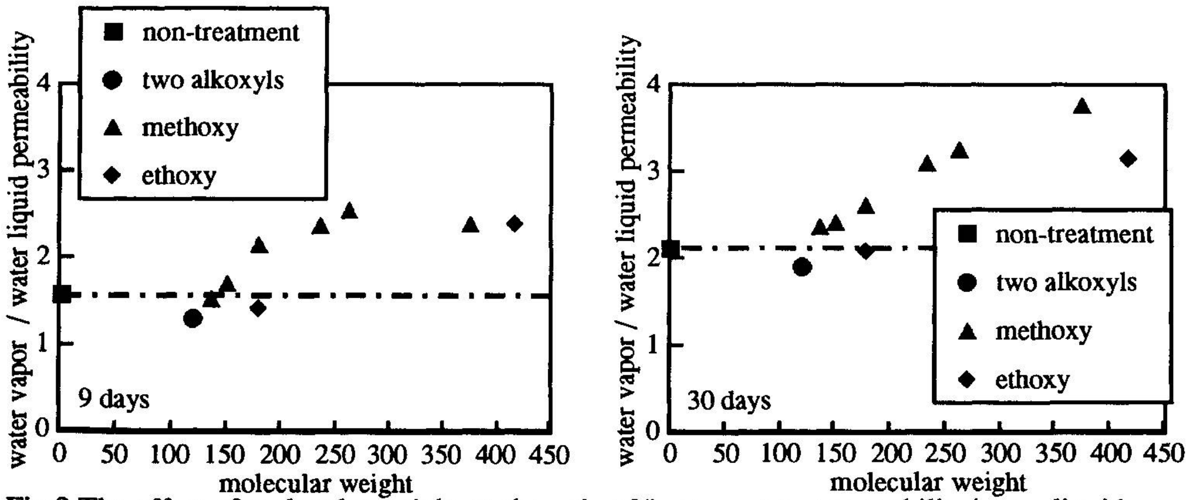

Fig.3 The effect of molecular weight on the ratio of "water vapor permeability/water liquid permeability

3.1.2 Water liquid permeability

Fig.2 shows the relationship between molecular weight and water liquid permeability. In methoxy series, the silanes of larger molecular weight showed the lower water liquid permeability. Since the size of the hydrophilic alkoxyls were the same, this might be due to longer hydrophobic alkyls corresponding to larger molecular weight. The same tendency was observed in ethoxy series as well -

the silanes of larger molecular weight had better resistance against penetration of water.

3.1.3 "Water vapor permeability / Water liquid permeability"

Fig.3 shows the effect of molecular weight on the ratio of "water vapor permeability / water liquid permeability". After 9 days of exposure, while 136, 150 and 178(MTES) showed smaller ratios as compared with that of the non-treated specimen, 262, 234, 374 and 416 of larger molecular weight similarly had good hydrophobicity. Similar results were obtained after 30 days of exposure, that is, when the larger molecular weight resulted in the larger ratio. The silanes which showed large ratio also indicated good hydrophobicity under dry and wet chamber and outdoors conditions. Considering that the actual concrete structures are exposed to dry and wet condition and the relatively small size of the specimens were used in the preliminary experiment, the silanes used in the main experiment was selected mainly on the basis of the results of hydrophobicity in 9 days of exposure. For this reason, the four types of silanes of molecular weight 234, 262, 374 and 416 were selected. Among them, 262 was regarded to have the best hydrophobic property in this series.

3.2 Main experiment

3.2.1 Weight changes

Under all conditions, the weight changes of all specimens impregnated with the silanes were smaller than that of the non-treated specimens. This indicates that the silanes could also control the water content of the specimens used in main experiment. Viewed in terms of weight changes, the silanes proved to be highly effective, especially under the condition of partially immersing in chloride solution.

3.2.2 Effect on expansion caused by alkali-silica reaction

Fig.4 shows the strain under the dry and wet condition. It is observed that non-treated specimens expanded significantly due to the influence of reactive aggregate and chloride. On the other hand, the specimens impregnated with the silanes expanded much less than non-treated specimens. The result indicates that they controlled the water content in the specimens and were quite effective against alkali-silica expansion. However, since the strains of treated specimens were too much larger than those of the specimens without reactive aggregate, it should be noted that unless concrete is mixed properly, the silanes would fail to restrain concrete from excessive expanding in a long term.

3.2.3 Effect on reinforcement corrosion

Fig.5 shows some results of the halfcell potential obtained from the non-reactive specimens in the condition of partially immersing in chloride solution. While the

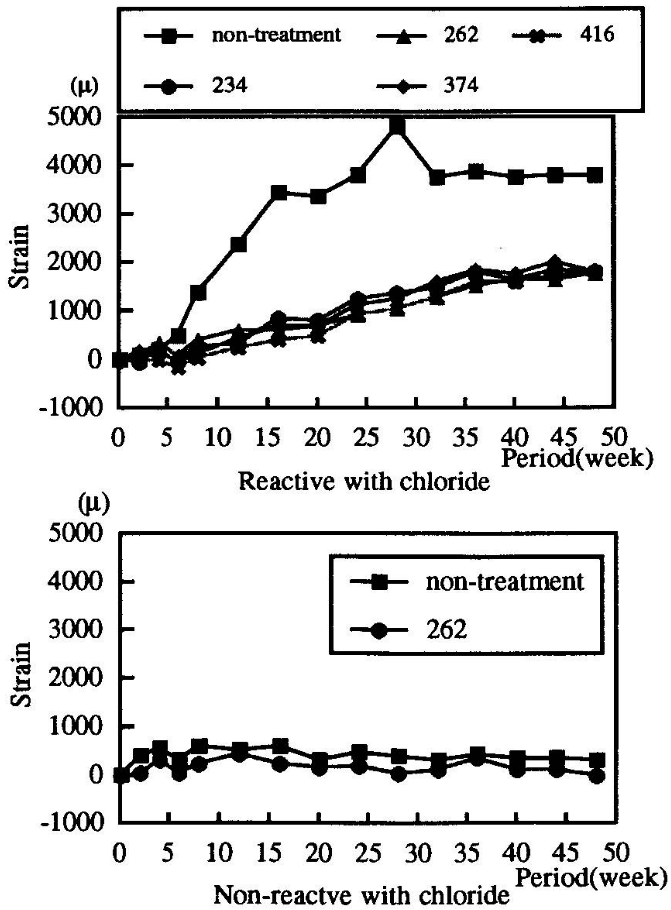

Fig.4 Strain

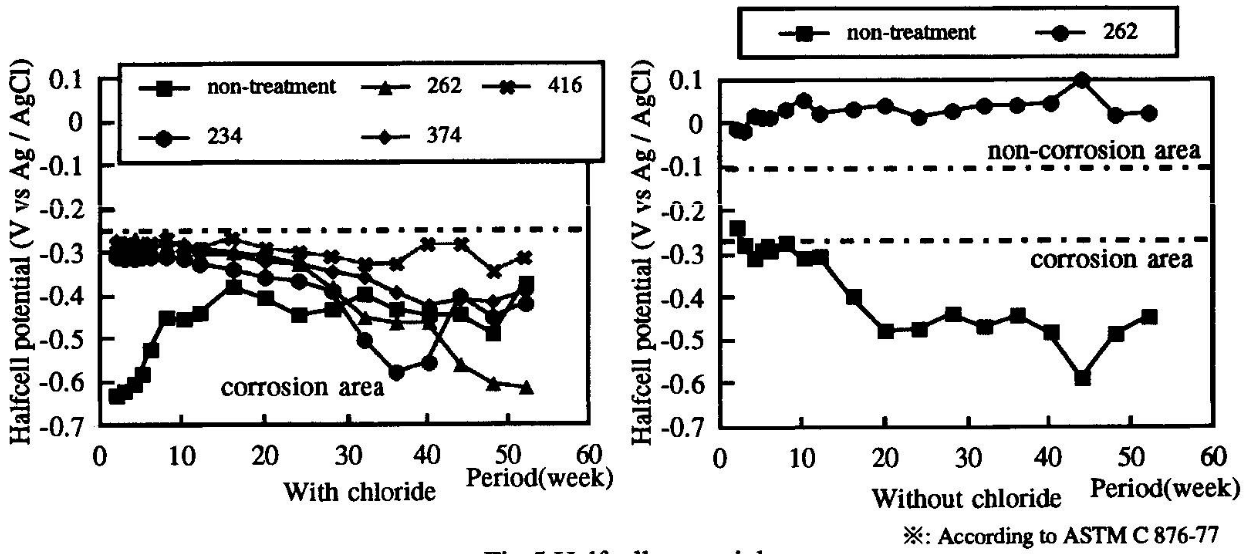

Fig.5 Halfcell potential

halfcell potential of the non-treated specimen without chloride was in the corrosion area, that of the specimen impregnated with 262 was still in the non-corrosion area. This indicates that the silane worked effectively also against corrosion of reinforcement caused by the chloride solution. However,when concrete included much chloride, although the halfcell potential of the impregnated specimens were less negative than that of non-treated specimens in a short term, they gradually became negative with the time passing until some of them were more negative than that of non-treated specimens.

4. Conclusion

The main results obtained in this study are summarized as follows.

1. The silanes of small molecular weight made the large amount of water get out. On the other hand, the silanes of large molecular weight which have long alkyl groups had better resistance against the ingress of water. Among the silanes used in this experimental study, the silane of molecular weight 262 showed the best hydrophobic performance.
2. The silanes worked effectively against the expansion of concrete caused by alkali-silica reaction and the reinforcement corrosion. That is, by impregnating the silanes to concrete, the expansion and the halfcell potential were reduced or even restrained. However, if concrete included an excessive amount of reactive potential and/or chloride, it was difficult to restrain deterioration of concrete in a long term.

5. Acknowledgment

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Extending the Lifespan of Cylindrical Structures

Prolongement de la durée de vie des constructions cylindriques

Erhöhung der Lebensdauer von zylindrischen Konstruktionen

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SUMMARY

Concrete structures frequently show signs of damage such as cracks, caused, e.g. by thermally induced stresses, which usually were not considered adequately by the design provisions. The paper will attempt to summarise conclusions from the information about the most common failures and repair methods of cylindrical reinforced industrial structures such as chimneys, water tanks, silos and cooling towers.

RÉSUMÉ

Les constructions en béton armé laissent souvent apparaître des défauts, telles que fissures engendrées par des tensions amenées par des effets de température que ne sont pas souvent pris en considération lors du calcul du projet. La contribution résume les connaissances sur les défauts et sur les possibilités de réparation des constructions industrielles cylindriques, telles que cheminées, réservoirs d'eau, silos, et tours de refroidissement.

ZUSAMMENFASSUNG

Stahlbetonkonstruktionen zeigen oft Zeichen von Beschädigungen, wie zum Beispiel durch Temperaturspannungen entstandene Risse, die in den bestehenden Normen für die Bemessung nicht ausreichend berücksichtigt wurden. Der vorliegende Beitrag versucht, die Erkenntnisse über die Fehler und Sanierungsmethoden von zylindrischen industriellen Konstruktionen wie z.B. Schornsteine, Wasserbehälter, Silos und Kühltürme zusammenzufassen.

1. INTRODUCTION

Concrete structures in service may be affected by ageing, which may include changes in strength and stiffnes. Some of this ageing effects are benign, others may cause component or system strength to degrade over the time, particularly when the concrete is exposed to an aggressive environment.

Environment stressors may attack the integrity of the concrete and/or steel reinforcement in concert with or independent of operating, environmental, and accidental loads. For concrete strength, the most significant stressors are chemical reactions, freeze-thaw cycling, and temperature effects. For deformed bar reinforcement and prestressing tendons, the possibility of corrosion is the far most important factor.
The most common of all types of problems in r.c. cylindrical structures are the vertical and horizontal cracks of the walls.

Reinforced concrete will crack. In most cases this cracking is not a cause for concern and no treatment is needed. However in some instances, remedial measures may be necessary. Cracks need to be repaired if they reduce:

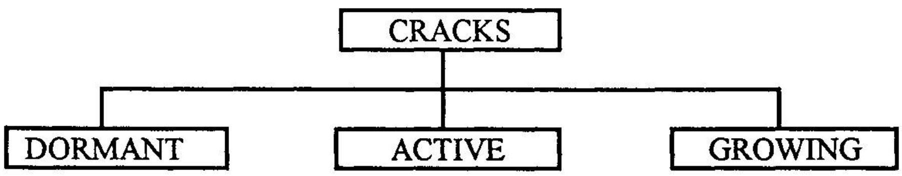
The structural safety, either the load bearing capability or the stability. As long the crack widths are not larger than approximately 0,3 mm and above all, the steel remains in the elastic region, the formation of cracks is considered as a normal phenomenon in reinforced concrete construction [1].

The serviceability. The environmental engineering concrete structures for the containment, treatment, or transmission of water, waste water, or other fluids should be designed and constructed to be essentially watertight. The ability of a structure to retain liquids will be reasonably assured if the crack width is minimised. Cracks up to 0,2 mm can be closed under the conditions which prevail in the field of water tank construction due to autogenous healing [2]. Healing will not occur if the crack is active and is subjected to movement during the healing period [3].

The durability. In the region of cracks carbonation and chlorides tend to penetrate faster towards the reinforcement than in uncracked concrete. The thickness of the concrete cover is of major importance with regard to the influence of cracks. The crack widths (if they are less than 0,4 mm) are less important. Limitation of crack widths for prestressing steel is completely different from that for ordinary reinforcement. Due to the danger of brittle failures, depassivation of the prestressing steel surface must be avoided during the entire lifetime. In most cases decompression must be asked for [4].

A suitable repair counteracts all the deficiencies which are relevant to the use of the structure.

Cracks can be divided into three categories



Dormant cracks are caused by some event in the past which is not expect to recur. They remain constant in width.
Active cracks are cracks which do not remain constant in width but open and close, perhaps as the structure is loaded, perhaps with changes in temperature.
Growing cracks are cracks which are increasing in width because the original reason for their occurrence is continuing.

Before embarking on any treatment, it is important to be aware of why cracks have occurred, otherwise an inappropriate and consequently ineffective repair method can be chosen and may do more harm than good [5].

2. CAUSES OF CRACKS

Cracks in concrete have many causes. White the specific causes of cracking are manifold, the principal causes of cracking in cylindrical industrial structures are thermal stresses and corrosion of reinforcement.

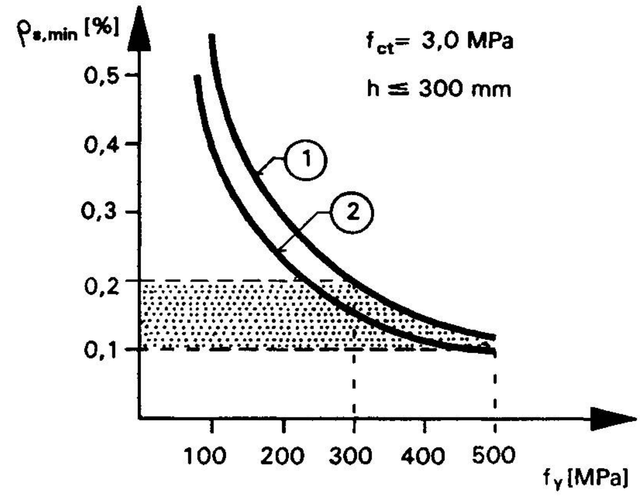
2.1 Thermal stresses

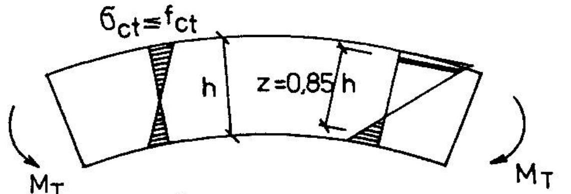
Temperature differences within cylindrical concrete structures may be due to heating up by internal influence as:

- waste gases with a temperature $T_i \leq 300°K$ (chimneys)
- steam, $T_i \leq 35°K$ (cooling towers)
- putrefactive spoilage $T_i = 37 - 55°K$ (sludge digesters)
- stored materials, $T_i \leq 200°K$ (silos)

or external weather influences like frost and solar radiation. As a consequence of the temperature differences dT between the inside and the outside surface, bending moments are activated. Bending moments starting approximately from dT = 15°K, lead to the formation of vertical crack.

Older cylindrical concrete structures, were designed and built frequently with insufficient hoops and that is the reason why, relatively often, vertical separation cracks with the yielding of the reinforcement can be observed. As shown in Figure 1, the minimal degree of reinforcement ρ_{min} depends on the relation of the tensile strength f_{ct} of concrete to the yield point f_y of steel.





$$M_T \approx \frac{f_{ct} \cdot h^2}{\sigma} = M_r \approx \sigma_{sr} . A_s . 0{,}85h$$

$$\sigma_{sr} \approx \frac{f_{ct}}{5\rho_s} \leq f_y$$

(1) $\rho_{s,min} = 0{,}2 \frac{f_{ct}}{f_y}$

EUROCODE 2: $A_{s,min} = k_c . k . f_{ct,eff} . A_{ct} / \sigma_s$

(2) $\rho_{s,min} \approx 0{,}16 \frac{f_{ct}}{f_y}$

Fig.1 The min. degree of reinforcement ρ_{min} versus yield point f_y of steel

One of the major requirements for the structure is to prevent wide vertical separation cracks which subdivide the cylindrical wall into several free-standing segments.

As shown in Fig.2, the cylindrical wall divided by separation cracks behaves with regard to its stresses and deflection much more unfavourably than a monolith structure.

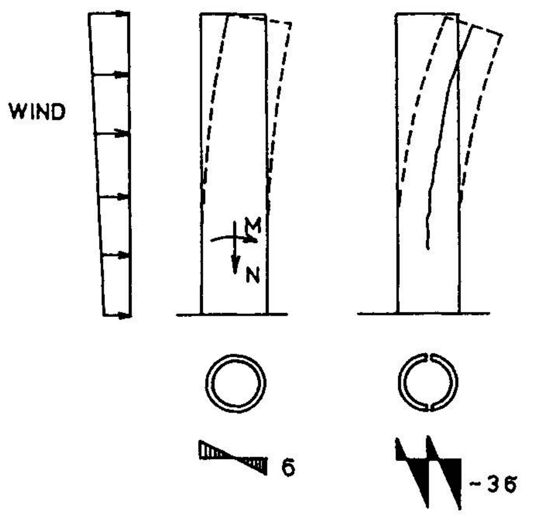


Fig.2 Effects of separation cracks on the load bearing capacity with regard to the wind

2.2 Corrosion of reinforcement

Reinforcing steel may corrode, however, if the alkalinity of the concrete is reduced through carbonation or the passivity of this steel is destroyed by aggressive ions (usually chlorides). Corrosion of the steel produces iron oxides and hydroxides, which have volume much greater than the volume of the original metallic iron. This increase in volume causes high radial bursting stresses around reinforcing bars and results in longitudinal cracks, or spalling of the concrete.
The cracking time of concrete cover can be calculated approximately from following formula [6]:

$$t = 80\frac{c}{d_s . r}$$

where: t is the cracking time (years)
c the thickness of concrete cover (m)
d_s the diameter of the reinforcing bar (m)
r the rate of steel corrosion in concrete (μm/year).

The rate of corrosion at the temperature of 20°K with regard to the relative humidity is presented in Table 1.

Table 1 The rate of corrosion in concrete

RH (%)	Carbonated concrete (μm/year)	Chloride contaminated concrete (μm/year)
99	2	34
95	50	122
90	12	98
85	3	78
80	1	61
75	0,1	47
70		36
65		27
60	0	19
55		14
50		9

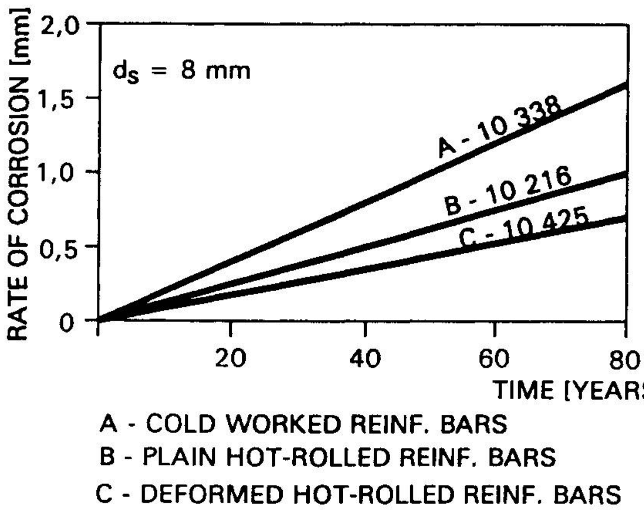


Fig.3 Mean atmospherical corrosion rate as a function of time [7]

Once the concrete cover has been cracked and spalled of, corrosion of the exposed steel will freely proceed like atmospherical corrosion. The mean values for the atmospherical corrosion rate are presented in Fig.3.

The design engineer may set a limit for the minimum cross-sectional area of the main reinforcement steel bars. It may depend on the requirement that the actual stresses in the steel bars can't exceed the yield point.

3. REPAIR METHODS

Following the evaluation of the cracked structure and the determination of the cause of the cracking a suitable repair procedure can be selected. Procedures can be selected to accomplish one or more of the following objectives:

1. Restore the original stiffness;
2. Improve functional performance (e.g. watertightness);
3. Improve appearance of the concrete surface; and/or
4. Improve durability.

Depending on the nature of the damage, one or more repair methods may be selected. For example tensile strength can be restored across a crack by injecting it with epoxy. However, it may be necessary to provide additional strength by adding reinforcement or using post-tensioning. The key methods of crack repair, particularly for cylindrical walls are described in Chapter 3.1 and 3.2.

3.1 A new outer reinforced concrete shell

The obvious solution for cylindrical r.c. structures is to provide the cracked shaft with an additional thin shell (Fig.4). The new outer shell comprises all envisaged repair measures, so that no separate crack pressure - grouting and improvement of concrete surface are necessary[8]. However, unless the cracks are dormant (or the cause of cracking is removed), they will probably recur. In order to restrict the crack width at the outer shell, high amount of reinforcement must be inserted. In view of the increase in dead weight the foundation pression must be verified.

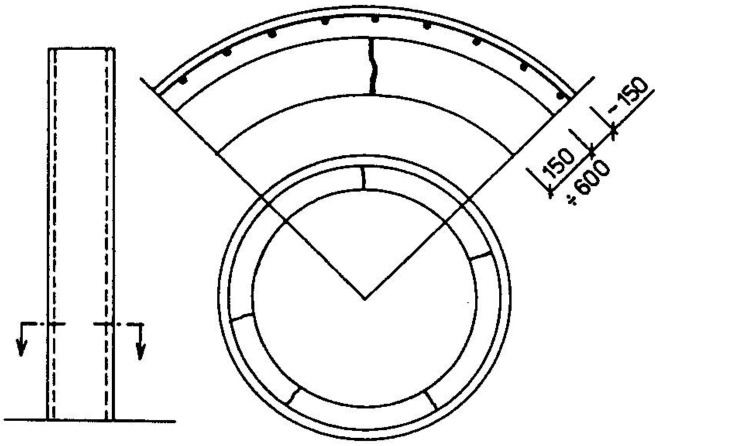


Fig.4 Shaft modification by means of reinforced concrete-jacketing

3.2 Transverse prestressing

Post-tensioning is often the desirable solution when the cracks that have formed must be closed. This technique uses prestressed concrete rings at intervals of about 10 m (Fig.5). They are post - tensioned by means of single unbounded tendons [9]. The local transverse prestressing must be restricted in view of additional vertical and tangential stresses which can lead to a additional crack formation. More uniform circumferential precompression can be achieved with external prestressing tendons at intervals about 2 m (Fig.6). This stressing technique is particularly beneficial when the wall is insufficiently reinforced at the inner surface.

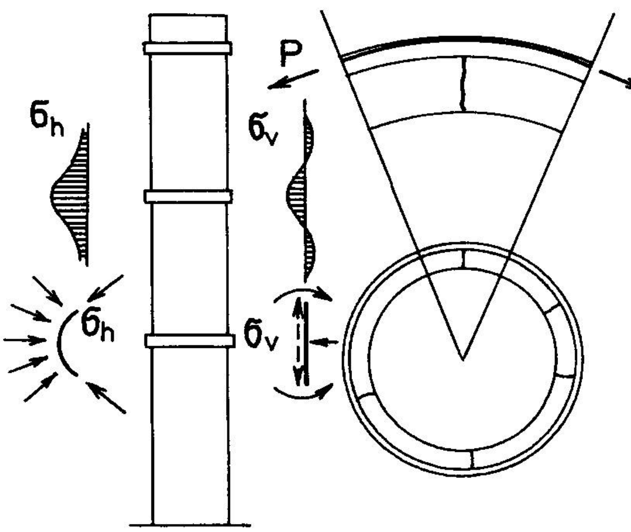


Fig.5 Shaft modification by means of prestressed concrete-rings

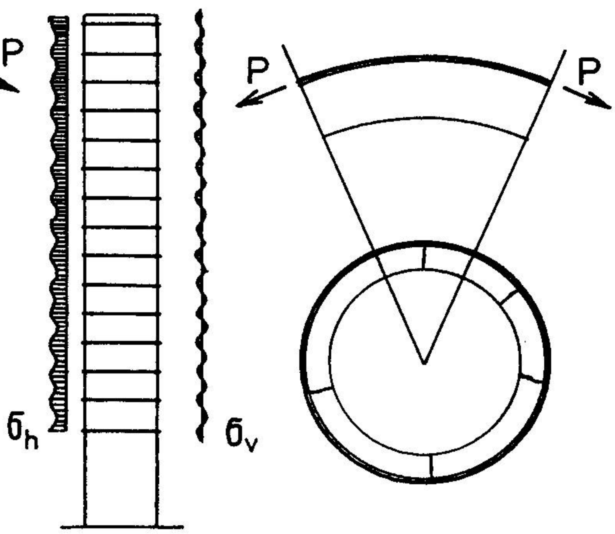


Fig.6 Shaft modification by means of prestressed external tendons

4. CONCLUSIONS

Primary causes of cracks and available actions to extend the functional life of cylindrical r.c. industrial structures are discussed. It has been shown that:

1. Temperature gradient between the inside and outside surface is the most likely cause for the development of vertical cracks and corrosion of reinforcement is the most likely cause for spalling of the concrete cover,
2. Some of these problems are due deficiencies in the previous or present standards, while others are due to the designers failure to comprehend fully the requirements of the standard. Still other problems result from the fact that many structures under inspection were not reinforced as designed,
3. The selection of successful repair techniques should consider the causes of cracking, whether the cracks are dormant, active or growing, and the need for repair of structures whose designed lifetime is over, while they are still in function, and
4. Application of external prestressing for restoring the orginal stiffness, improving the functional performance and durability is a very economical procedure and may be used on the cylindrical industrial structures.

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