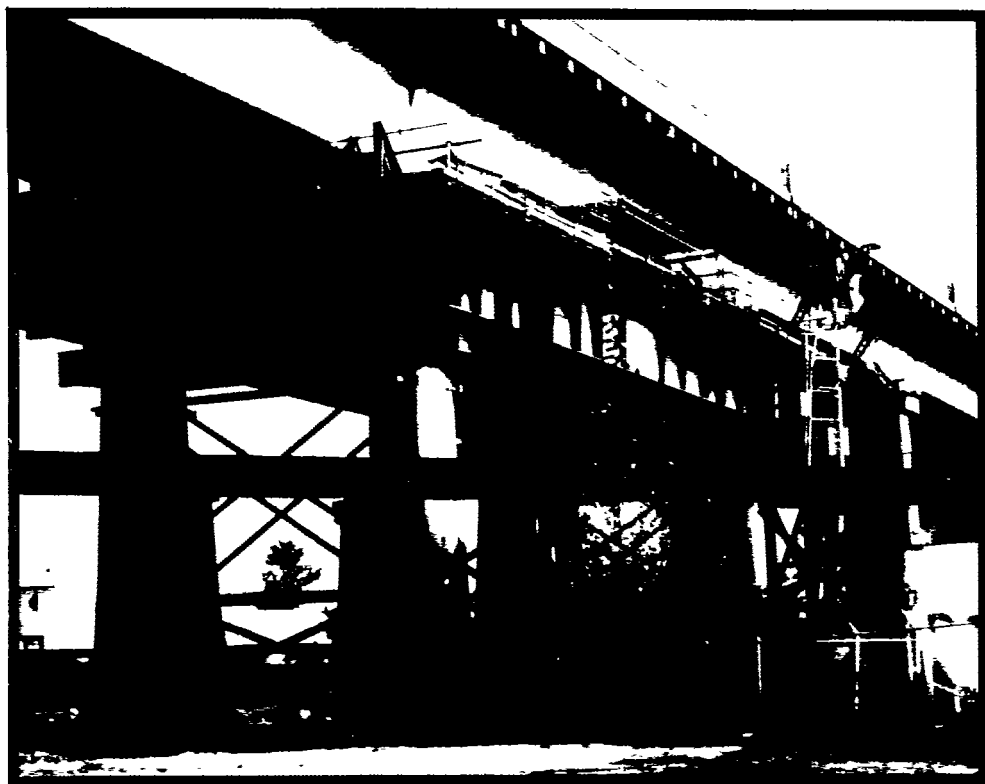


# Evaluation and Rehabilitation of Concrete Structures and Innovations in Design

Proceedings  
ACI International Conference  
Hong Kong, 1991



V. M. Malhotra  
*Editor*

SP-128  
Volume I



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## PREFACE

In December 1991, the American Concrete Institute sponsored an International Conference on Evaluation and Rehabilitation of Concrete Structures and Innovations in Design. This was in keeping with the mission of the Institute, and the specific purpose of the Conference was to disseminate technical information on emerging concrete technology in the Pacific rim region, and to provide technical support for a growing number of ACI International Chapters and members. The Conference was held in Hong Kong, December 2-6, 1991. More than 131 papers from 26 countries were received and reviewed in accordance with the policies of the American Concrete Institute and 90 were accepted for publication. For all those whose submissions could not be included in the symposium volume, the Institute and the Conference Organizing Committee extend their appreciation for their interest and hard work. The accepted papers were published in two volumes, with volume I containing papers dealing with materials and volume II comprised of papers pertaining to design aspects. In addition to the papers which have been published in the proceedings, several others were presented at the Conference, and their authors have been encouraged to publish them in a technical journal of their choice.

Thanks are extended to the members of the review panel who helped review the draft manuscripts. Without their prompt review and constructive comments, it would not have been possible to publish the volumes for distribution at the Conference. The cooperation of the authors in accepting reviewers' suggestions and revising their manuscripts accordingly is greatly appreciated.

The help and assistance of William R. Tolley and his staff for performing the administrative duties associated with this publication are gratefully acknowledged.

V. Mohan Malhotra, Chairman

ACI International Conference on Evaluation and Rehabilitation  
of Concrete Structures and Innovations in Design

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**Dedicated to those members of ACI administrative staff who  
made possible the timely publication of these proceedings.**

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SP 128-1

# A Highly Elastic Surface Coating to Protect Structures Exposed to Coastal/Marine Environments

by R. N. Swamy and S. Tanikawa

**Synopsis:** Concrete is a characteristically porous material, and whatever improvements are made in its formulation and fabrication, micropores and microvoids would always exist on their surfaces. These defects would encourage the transport of aggressive agents into the body of concrete, bearing in mind that cracking, depth and quality of cover concrete and the overall quality of concrete are the three factors that influence the penetration of these agents. Surface coatings on concrete have an important role in preserving the durability of concrete and the steel embedded in it. This paper presents test data on an acrylic based highly elastic surface coating. The strength and elasticity of the coating, its ability to bridge cracks under static and dynamic loadings, its resistance to natural weathering, heat, oxidation, sunshine and rain are discussed. Data are also presented on the adhesion strength of the coating to concrete substrate, and its resistance to carbonation and chloride penetration. Both short and long term test data are presented. It is shown that Aron Wall surface coating can provide long term durability and stability to concrete structures and that it can maintain its integrity, continuity and adhesion to concrete under very aggressive environmental conditions.

**Keywords:** Adhesion; carbonation; chlorides; concrete durability; corrosion; elastic properties; marine atmospheres; protective coatings; temperature; weathering

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### INTRODUCTION

Long experience and a good understanding of material properties has shown us that concrete is a reliable and durable construction material when it is built in sheltered conditions or exposed to normal environments where aggressive elements are not present. Even in moderately aggressive environments, concrete can give long trouble-free service provided care and control are exercised in choosing the right constituents and in mix proportioning, placing, compacting and curing the finished material.

In aggressive environments, particularly where chlorides are present either through the nature of the surroundings or through the use of deicing salts or salt-contaminated constituents, the durability of concrete and of the steel embedded in it can be severely at risk. In many instances the degradation process of concrete is not the result of one factor or one set of aggressive agents. Other factors such as  $\text{CO}_2$  induced carbonation, and atmospheric pollutants also play a very significant part in accelerating the deterioration process. Atmospheric pollutants of sulphur and nitrogen, for example, can reduce pH values to as low as 2.5, create expansive compounds on reacting with cement hydration products, and cause disintegration of concrete. The presence of 1 ppm of  $\text{SO}_2$ , for example, can accelerate the progress of the carbonation front due to  $\text{CO}_2$  alone (1). It can be readily seen that the development of deterioration in concrete is an overall synergistic process, a complex combination of many individual mechanisms, the exact role and effect of each of which is not yet clearly understood.

There are sound and rational reasons as to why concrete exposed to such environmental conditions will suffer a slow, steady but continual and progressive process of deterioration. Where quality control is deficient either in the choice of materials or in any of the several stages involved in construction technology, the cumulative synergistic effects of atmospheric pollutants and other deteriorating agencies can cause unexpected and premature damage to concrete structures often observed at a much earlier age than that predicted by service life models.

Concrete is also a characteristically porous material. It is inherent in the nature of concrete production, that whatever improvements are made in the formulation of the material and the control of its quality, they are not likely to prevent completely the ingress of potentially harmful agents into the material. Micro cracks and micro pores will always exist on the surface of concrete, and provide a path of transport for aggressive elements into the interior of concrete. It is then only a matter of time that chloride ions in particular penetrate into concrete.

The three significant factors that encourage the transport mechanism of these aggressive agents into concrete and affect adversely the durable service life of concrete structures are cracking, cover (both depth of cover and cover quality) and the overall quality of concrete. These three influences have an interactive and interdependent, almost synergistic, effect in controlling the intrusion into concrete of external aggressive agents such as air, water and chloride ions. Atmospheric carbonation (including that arising from the oxides of nitrogen and sulphur) and chloride ions are now known to be the most potentially destructive agents affecting the performance and durability of concrete structures, whilst the depth of cover and its quality are recognised as the most critical factor in determining the electrochemical stability of steel in concrete (2-5).

Environment is thus the one single predominant external factor that can sow the seeds of progressive deterioration in concrete which can critically decide the duration of stable service life of any structure that is exposed to external atmosphere. It so happens that environment is also the one single factor that is beyond the control of human beings.

#### SURFACE COATINGS

There are two possible solutions, not necessarily alternatives, which can help to prevent this unacceptable process of degradation of concrete structures. Surface coatings on the rebar itself can be most effective in substantially delaying or even preventing the risk of corrosion of steel

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embedded in concrete. There are at least two such coatings which have found fairly widespread acceptance, even though they may still have their own limitations, advantages and disadvantages (5-7). Surface or barrier coatings on the concrete have also a long history of application to protect it and the embedded steel from various aggressive agents and external environments. A wide range of vapour barriers, vapour permeable coatings, and surface penetrating sealers have been evaluated in published literature and are available on the market (8-13). Surface coatings, whether on steel or concrete, should however not be at the expense of good quality concrete.

There have, however, been real difficulties in choosing the correct type of coating for a given environment. The availability of a wide range of barrier coatings and the fact that coatings of similar generic types are known to have considerably different diffusion characteristics make the situation very confusing (14, 15). Further, coatings which may have, for example, excellent resistance to chloride intrusion may not be very effective in preventing ingress of  $\text{CO}_2$  (or vice versa) (15, 16). In addition, one has also to recognise that in real environments, structures may be subjected to wetting/drying cycles of freeze-thaw cycles or subjected to expansive reactions or other similar deteriorating processes. There is very little information currently available on the performance characteristics of coatings under such conditions.

Another important factor to be considered is that when protecting concrete, the surface coating has to be able to bridge existing as well as new cracks forming in concrete, and still prevent the penetration of agents that cause deterioration. It is therefore important to evaluate the performance of coatings both in their natural and extended states. Much of the published data on the performance characteristics of surface coatings conveniently ignore this aspect (12-16).

The aim of this paper is to present data on the performance characteristics of a highly elastic acrylic rubber coating, called ARON WALL (AW) coating. Special attention has been given to evaluate the properties of the coating in both its natural and extended states. Data are presented on its crack bridging ability, weathering and heat resistance, adhesion strength, carbonation resistance and resistance to chloride penetration.

#### CHEMISTRY OF ARON WALL

The ARON WALL coating consists of a primer, base coat and top coat with an overall thickness of about 1100 $\mu\text{m}$ . The primer is a specially designed chlorinated (polymer) resin which is dissolved in organic solvents as one component type

and which has a low viscosity of about 0.3 Poise. The base coat is a one component aqueous type which has a high viscosity of 300 Poise. The uniqueness of the base coating is that it mainly uses a soft monomer which has a low glass transition temperature ( $-82^{\circ}\text{C}$ ) which gives it remarkable elastic properties at low temperatures, and enables this elasticity to be sustained for a long time (17). Three types of top coating are possible: an acrylic resin (one component type), acrylic urethane resin (two components type) and acrylic silicone resin (two components type).

## PERFORMANCE CHARACTERISTICS OF ARON WALL

### Elasticity and Crack Bridging Ability

One of the basic engineering requirements of a surface coating is that it should have a high strain capacity, high elasticity and a good tensile strength which will enable it to bridge cracks, accommodate movements and allow without fracturing the opening and closing of cracks. The base coating used in Aron Wall has a very high strain capacity in tension of  $6 \times 10^6$  microstrains, and thus very large extensibility before rupture occurs. The coating is also able to resist tensile stresses in excess of 20 MPa.

The crack bridging ability of the coating was determined by testing the coatings in tension. The coatings were mounted on a slate board  $150 \times 250 \times 5\text{mm}$ : the primer and top coating thicknesses were kept constant at 0.03mm and 0.1mm respectively while the base coating thickness was varied from 0.2mm to 2.5mm. The coating was able to bridge cracks of varying width without rupture, the maximum crack width bridged being over 10mm at a base coating thickness of 2.5mm.

### Fatigue Resistance

Cracks in a concrete structure will in practice open and close depending on the type of applied load and its duration, as well as due to structural movements and the presence of other deteriorating agencies. Cyclic tension tests were therefore conducted on the AW coating with a total thickness of primer + base + top coating of 1.0mm. The details and the sequence of tests are shown in Table 1. The tests were conducted in three ranges of crack width, namely, 0.5-1.0mm, 1.0-2.0mm and 2.0-5.0mm. At each range of crack width, the specimens were subjected to 667 cycles each at  $20^{\circ}\text{C}$ ,  $5^{\circ}\text{C}$  and  $-10^{\circ}\text{C}$  so that at each range of crack width, the test specimens were subjected to a total of 2001 cycles. At the end of the first 2001 cycles, the range of crack width was increased to 1.0-2.0mm, and the 667 cycles repeated at each of the temperatures  $20^{\circ}\text{C}$ ,  $5^{\circ}\text{C}$  and  $-10^{\circ}\text{C}$ . The last 2001 cycles were carried out at a crack width of 2.0-5.0mm. The coating was

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carefully examined at the end of each 667 cycles, and finally at the end of 6003 cycles. The results given in Table 1 show that no cracking occurred in the crack range 0.5-1.0mm at 20°C and 5°C whilst cracks or pin holes occurred in the top coating at -10°C in the 0.5-1.0mm crack range and at all temperatures in the 1.0-2.0mm range. In the crack range of 2.0-5.0mm, the base coating showed signs of cracking at all temperatures. However, in no case did a complete breakdown of the coating occur emphasizing the ability of AW coating to bridge a wide range of crack widths and protect the structure under dynamic loadings.

### Weathering Resistance

In these tests dumb-bell shaped specimens of the base coat membrane, 1mm thick, were left exposed in the open facing south at an angle of 45° for a period of 10 years, and tested in tension at regular intervals. At this orientation, the coatings were exposed to the most severe solar radiation, the annual mean total radiation (in Langley) over a five year period being 132,110.9. The exposure site had also a high variation of temperature, humidity and rainfall. The mean monthly variations over a period of 12 months were:

|                   |                 |
|-------------------|-----------------|
| Temperature       | - 2.5°C to 27°C |
| Relative Humidity | - 60% to 80%    |
| Rainfall          | - 50mm to 200mm |

The results of these exposure tests are shown in Table 2. It can be seen that during an exposure period of 10 years, the tensile strength decreased from about 2.2 to about 1.5 MPa while the elongation decreased from about 315% to about 170%. Even with these losses, the results show the ability of the base coating to maintain both adequate strength and elasticity for a long period of time.

### Resistance to Ozone

The ozone exposure test was carried out on both the base coat (i.e. primer + base coat, about 1030µm thick) and the coating as a whole (i.e. primer + base + top coat of acrylic urethane, about 1100µm total thickness). In addition, the coating was tested both in its natural unextended state and in an extended condition. In the test flexible boards of slate 150x70x3mm were coated with the coating, and the coated test pieces were then subjected to tensile forces in a holding device and cracked at the centre to crack widths of 0.1, 0.3, 1.0 and 3.0mm. The coatings stretching over the cracks were then exposed to an ozone concentration of 75 pphm for 168 hrs at 40°C. All the areas of the test specimens except the coated surface were protected against ozone penetration. After the exposure period the coating surfaces were examined for cracks, blisters, peeling and coating breakages around the cracks. None of the specimens showed any changes or breakages at the end of the test (Table 3).

### Ultraviolet Ray Exposure Test

The test specimens used for this test were similar to those used in the ozone exposure tests. The coatings, stretched over cracks of varying widths and held under tension were exposed to ultraviolet radiation in a special chamber for 1000 hrs. During the test, the specimens were sprayed at regular intervals with water for periods of 18 min to 2 hrs. The coatings were carefully examined again for signs of degradation during the tests and at the end of the test. As shown in Table 3, the Aron Wall coating remained intact without any signs of disruption.

Both the ozone and ultraviolet radiation tests are important tests to ensure the long term stability of rubber and resin based coatings. Such coatings could be damaged by prolonged oxidation, and the accelerated ozone exposure test reported here represents an equivalent life of 4.4 months of continued oxidation attack. Similarly, the ultraviolet radiation test represents about 4 years of continued exposure to sun and rain. Table 3 shows that the Aron Wall developed here will be able to give long term protection to concrete even when exposed to aggressive natural environments of oxidation, sun and rain, without suffering any damage itself.

### Temperature Resistance

In addition to the crack bridging ability under static and dynamic conditions, and the resistance to natural weathering, it is also important to determine the ability of the coating to resist temperature stresses. To establish this both the tensile strength and elongation properties of the base coat were determined for a wide range of temperatures from  $-70^{\circ}\text{C}$  to  $+100^{\circ}\text{C}$ .

In the first series of tests, both the % elongation and tensile strength were determined for a range of temperatures from  $-70^{\circ}\text{C}$  to  $+100^{\circ}\text{C}$  and the results are shown in Table 4. At  $20-30^{\circ}\text{C}$ , the base coating has a tensile strength of 2.1 to 2.3 MPa and a percentage elongation of about 330%. As the temperature decreases, the coating becomes much less elastic but the tensile strength increases substantially. As temperature rises, the tensile strength decreases but the ability to stretch increases. Within the range of  $-30^{\circ}\text{C}$  and  $+60^{\circ}\text{C}$ , which probably represents the extremes of temperature to which almost all concrete structures are exposed, the Aron Wall coating is able to have a % elongation of about 130% to 360% and a tensile strength of 8.5 to 1.7 MPa.

In the second series of tests, base coat specimens were exposed continuously to temperatures of  $80^{\circ}\text{C}$  and  $100^{\circ}\text{C}$  up to 150 days. The results of these tests are presented in Table 5 as percentage degradation in elongation and strength of the base coat. The results show that at  $80^{\circ}\text{C}$  the Aron Wall coating

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is able to retain nearly 90% of its original elongation, whilst the strength increased by 25%. At 100°C, the reduction in elongation was almost 50% at the end of 150 days while strength increase was up to 60%. These results confirm that the coating can remain stable and effective at as high a temperature as 80°C, and that it can withstand temperatures of 100°C for a few days without any damage to the coating and seriously affecting its elasticity properties.

Adhesion Strength

In practice surface coatings on concrete should have excellent adhesion strength to the concrete substrate, not only under normal environmental conditions but also when the structure is exposed to salt laden environments. To examine the adhesion capabilities of Aron Wall coating, two types of tests - one, accelerated salt spray exposure tests, and two, natural marine exposure tests were carried out.

Accelerated exposure tests -- In the accelerated exposure tests concrete prisms 55x65x150mm in size containing a 9mm diameter reinforcing bar in the middle were coated with the Aron Wall coating with an average total coating thickness of 1100µm. The concrete in the prisms was made from a mix of 1.0:2.38:2.91:0.58 (cement:sand:coarse aggregate:w/c ratio), all by weight. Normal portland cement and river aggregates were used. A water reducing agent was added, and the concrete had a slump of 200-210mm.

The test specimens were subjected to two types of salt solution exposure regimes. In the first the specimens were subjected to a salt spray wet-dry cycle. The specimens were initially sprayed with a 3% sodium chloride solution for 12 hrs at 30°C followed by drying for 12 hrs at 50°C. The test was continued for 100 cycles. In the second regime, the specimens were sprayed continuously with 3% NaCl water for 5000 hrs (about 210 days) at 35°C. This test regime aimed to simulate structural deterioration due to salt in hot and humid conditions.

In both the wet-dry cyclic test and the salt spray test, the specimens with Aron Wall surface coatings showed no surface deterioration, and no blisters or cracks appeared in the coatings. The uncoated specimens developed cracks of 0.1mm width in the wet-dry cyclic test, but no cracking occurred in the continuous salt spray test. The average adhesion strength of the coating to the concrete surface was 1.63 MPa and 1.18 MPa respectively after the cyclic test regime and the continuous salt spray. In the cyclic test, failure occurred in the base coat whilst in the continuous spray test, peeling occurred between the top and base coats.

Marine exposure tests -- The specimens for these tests

consisted of reinforced concrete prisms 200x200x300mm with 12-16mm dia bars and 9mm dia stirrups. The main steel had covers of 20mm and 30mm. The concrete mix proportions were the same as that used for the accelerated salt spray tests. Specimens were also made without and with added sodium chloride at the time of mixing. The test specimens were coated with the Aron Wall coating with an overall thickness of about 1000  $\mu\text{m}$ ; companion uncoated specimens were also made for comparison purposes.

The specimens were subjected to two exposure regimes - they were either partially soaked in sea water or exposed to a hot and humid natural weather 2 km from the sea and 2m above ground. The specimens were left exposed for 5 years, and the adhesion strength determined at the end of this exposure period.

The adhesion strength results are shown in Table 6. No blisters, peeling or cracking were observed on the surface of the Aron Wall coating in any of the exposed specimens. Uncoated specimens showed various degrees of cracking, depending on the cover to steel, the type of exposure and amount of chloride added to the concrete. The coating adhesion strength as shown in Table 6 varied from 1.51 to 1.62 MPa in the partially soaked specimens and from 1.66 to 1.71 MPa in the outdoor exposure specimens. The results again emphasize the integrity, excellent adhesion and film continuity of the Aron Wall coating even under severe exposure conditions.

#### Carbonation Depth

The 5 year old uncoated and coated specimens were also tested for penetration of the carbonation front. The carbonated thickness of the concrete was determined by spraying a 1% phenolphthalein ethanol solution containing 10% water. The results of these carbonation tests are shown in Table 7. Uncoated specimens soaked in sea water showed carbonation depths varying from 4.5 to 7.2mm after 5 years exposure whilst specimens exposed outside showed carbonation depths of 7.6 to 8.9mm.

The Aron Wall coating, on the other hand, completely prevented the penetration of carbonation into concrete in both exposure regimes. There was no sign of carbonation in any of the specimens, without or with added sodium chloride. The coating maintained its high resistance to  $\text{CO}_2$  diffusion, and laboratory diffusion test results (17) have been fully vindicated by these field exposure tests.

#### Chloride Penetration

The ability of the Aron Wall coating in preventing the penetration of chloride ions was determined from the test specimens used for the adhesion tests described earlier. The

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specimens 55x65x150mm containing a single 9mm dia bar at the centre were subjected to accelerated tests - a salt spray wet-dry cycle or a continuous salt spray. The test specimens 200x200x300mm prisms reinforced with 12-16mm dia bars and 9mm dia stirrups, were either partially soaked in sea water or exposed to a hot and humid coastal environment for 5 years. The results of the accelerated tests are shown in Fig. 1 and those of the marine exposure tests in Fig. 2.

The chloride penetration profile in the uncoated and coated prisms after the accelerated salt exposure tests show that the coating has virtually prevented the penetration of any chlorides into concrete. The accelerated wet-dry cycles and continuous sprays represent extreme aggressive situations as shown by the penetration of chlorides into the uncoated specimens. The fact that the coating has successfully maintained its impermeability to chlorides under these conditions is a measure of the high quality of the Aron Wall coating.

In the outdoor exposure tests, there was little chloride penetration in both the uncoated and coated specimens after 5 years exposure. Obviously at a distance of 2 km from the sea, there was little water/air borne chlorides to penetrate the concrete. The surface coating was, however, very effective in preventing chloride penetration into the specimens soaked in sea water, as shown in Fig. 2. The uncoated specimens, on the other hand, showed a very distinct chloride profile, following the conventional diffusion law. Figure 2 also shows the chloride profile in the uncoated and coated specimens containing 0.6% and 1% NaCl added during mixing. While the uncoated specimens show high chloride concentrations, particularly, at the location of rebars, the surface coating on the concrete has helped the concrete to achieve a better redistribution of chlorides within the prism itself. In particular, the chloride concentration near the rebars has been prevented. Further, from the quantities shown, it is also clear that the coating has also acted as an effective barrier to the penetration of chloride ions into the concrete from the surrounding sea water.

The results given in Figs. 1 and 2 thus highlight the two distinct roles of Aron Wall surface coating when applied to concrete surfaces. In aggressive environments containing salt laden solutions constantly in contact with concrete, the surface coating can effectively prevent the penetration of chloride ions into concrete. Where the concrete is already contaminated with salt, the coating can additionally help to redistribute the salt more evenly and uniformly within the concrete element, and avoid the harmful concentration of chlorides in the vicinity of rebars, particularly when cover depths to steel are small. In both roles, the Aron Wall surface coating can bring real benefits to enhance the durable service life of concrete structures.

## CONCLUSIONS

The paper presents data on a new surface coating for concrete based on a highly elastic acrylic polymer. Data are presented on the strength and elasticity of the coating, its ability to bridge cracks, adhesion strength to concrete substrates, weathering and heating resistance, resistance to carbonation and resistance to chloride penetration. From the results presented, the following conclusions are drawn.

1. Tension tests show that with a base coating thickness ranging from 0.2mm to 2.5mm, the coating was able to bridge cracks of varying width. The maximum crack width bridged was over 10mm with a base coat thickness of 2.5mm.
2. The coating showed very good fatigue resistance over a range of crack widths from 0.5 to 5.0mm when tested at temperatures from 20°C to -10°C. In no case did a complete breakdown of the coating occur.
3. Exposure to natural weather over a period of ten years showed that the coating was able to maintain adequate strength (minimum 1.5 MPa) and elongation (minimum 170%).
4. Accelerated exposure tests to ozone and ultraviolet radiation of the coating in its natural unextended state and in an extended cracked state showed that the coating remained intact without disruption. The accelerated tests are equivalent to about 4.5 months of continuous oxidation and about 4 years of continuous exposure to sun and rain.
5. In the range of -30°C to +60°C, the Aron Wall coating maintained tensile strengths 1.7 to 8.5 MPa, and elongation of 130% to 160%.
6. The coating remained stable and effective when exposed continuously to a temperature of 80°C up to 150 days. It also withstood temperatures of 100°C continuously for several days without damage and seriously affecting its elasticity properties.
7. The coating showed excellent adhesion to concrete whether it contained chlorides or not in both accelerated salt spray tests (wet-dry cycles and continuous spray) and after 5 years of soaking in sea water and coastal exposure. In all cases no blisters, peeling or cracking were observed, and there was excellent adhesion, film continuity and integrity after exposure to aggressive salt laden environments.
8. Even after 5 years of soaking in sea water and exposure to hot and humid natural weather conditions, there was no sign of any carbonation in any of the specimens without

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or with added sodium chloride in the concrete. The Aron Wall coating had completely prevented the penetration of carbon dioxide into concrete.

9. The coating prevented the penetration into concrete of virtually all chloride ions when subjected to accelerated salt spray tests and five years of soaking in sea water. In chloride contaminated concrete, the coating additionally helped to achieve a more even and uniform distribution of chloride within the coated concrete element.
10. In preventing the penetration of chlorides into concrete, Aron Wall coating has two distinct roles. It prevents the ingress of chlorides from outside, and further, it prevents the chlorides, already in the concrete, from concentrating in the vicinity of embedded rebars.
11. The data presented here show that Aron Wall surface coating will provide excellent protection to concrete and the embedded steel inside whilst maintaining the strength, elasticity and stability of the coating for a long period of time under very aggressive environmental conditions.

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TABLE 1--FATIGUE RESISTANCE TO CRACK OPENING AND CLOSING

| Width of Cracks | Temperature During Test  |     |        |                  |
|-----------------|--------------------------|-----|--------|------------------|
|                 | 20°C                     | 5°C | -10°C  | Number of Cycles |
| 0.5 ≤ 1.0mm     | NO CRACKS                |     | CRACKS | 2001             |
| 1.0 ≤ 2.0mm     | AND PINHOLES IN TOP COAT |     |        | 2001             |
| 2.0 ≤ 5.0mm     | BASE COAT CRACKING       |     |        | 2001             |

TABLE 2--WEATHERING RESISTANCE OF BASE COAT

| Exposure Period<br>Years | Elongation<br>% | Strength<br>MPa |
|--------------------------|-----------------|-----------------|
| 0 (initial)              | 314             | 2.2             |
| 1                        | 259             | 2.1             |
| 3                        | 213             | 2.1             |
| 5                        | 196             | 1.8             |
| 7                        | 177             | 1.7             |
| 10                       | 169             | 1.5             |

TABLE 3--RESISTANCE TO OZONE AND ULTRAVIOLET RADIATION

| Crack Width<br>mm | Ozone              |               | Ultraviolet Radiation |               |
|-------------------|--------------------|---------------|-----------------------|---------------|
|                   | Primer & Base Coat | Total Coating | Primer & Base Coat    | Total Coating |
| 0.1               | NO CRACKS          |               | NO CRACKS             |               |
| 0.3               | NO BLISTERS        |               | NO BLISTERS           |               |
| 1.0               | NO PEELING         |               | NO PEELING            |               |
| 3.0               | NO BREAKAGES       |               | NO BREAKAGES          |               |

TABLE 4--TEMPERATURE RESISTANCE OF BASE COAT

| Temperature<br>°C | Elongation<br>% | Tensile Strength<br>MPa |
|-------------------|-----------------|-------------------------|
| -70               | 17              | 23.5                    |
| -55               | 45              | 18.4                    |
| -40               | 84              | 10.4                    |
| -30               | 126             | 8.5                     |
| -20               | 197             | 7.1                     |
| -10               | 237             | 4.4                     |
| 20                | 326             | 2.3                     |
| 30                | 334             | 2.1                     |
| 40                | 347             | 1.9                     |
| 50                | 353             | 2.1                     |
| 60                | 361             | 1.7                     |
| 70                | 368             | 1.2                     |
| 80                | 381             | 1.1                     |
| 90                | 392             | 1.1                     |
| 100               | 400             | 1.1                     |

TABLE 5--HEAT RESISTANCE OF BASE COAT

| Test Temperature<br>°C | Exposure Period<br>Days | Degradation Rate of Base Coat, % |          |
|------------------------|-------------------------|----------------------------------|----------|
|                        |                         | Elongation                       | Strength |
| 80                     | 0 (Initial)             | 100                              | 100      |
|                        | 7                       | 93                               | 109      |
|                        | 90                      | 94                               | 123      |
|                        | 150                     | 91                               | 125      |
| 100                    | 0 (Initial)             | 100                              | 100      |
|                        | 7                       | 84                               | 145      |
|                        | 90                      | 61                               | 151      |
|                        | 150                     | 49                               | 159      |

TABLE 6--ADHESION OF ARON WALL SURFACE COATING TO CONCRETE AFTER 5 YEAR'S EXPOSURE

| NaCl %                            | Soaking in Sea Water                       |      |      |      |      |      | Outdoor     |         |
|-----------------------------------|--------------------------------------------|------|------|------|------|------|-------------|---------|
|                                   | 0                                          | 0.2  | 0.4  | 0.6  | 0.8  | 1.0  | 0           | 1.0     |
| Appearance<br>Adhesion<br>st. MPa | <u>U N C O A T E D   S P E C I M E N S</u> |      |      |      |      |      |             |         |
|                                   | A L L   C R A C K E D                      |      |      |      |      |      | NO<br>CRACK | CRACKED |
|                                   | <u>C O A T E D   S P E C I M E N S</u>     |      |      |      |      |      |             |         |
|                                   | NO CRACKS ... NO BLISTERS ...              |      |      |      |      |      | NO CHANGE   |         |
|                                   | 1.59                                       | 1.62 | 1.58 | 1.51 | 1.55 | 1.52 | 1.66        | 1.71    |

TABLE 7--CARBONATION DEPTH OF UNCOATED AND COATED SPECIMENS AFTER 5 YEAR'S EXPOSURE

|                           |     | Soaking in Sea Water |     |     |     |     | Outdoor Exp. |     |
|---------------------------|-----|----------------------|-----|-----|-----|-----|--------------|-----|
| NaCl %                    | 0   | 0.2                  | 0.4 | 0.6 | 0.8 | 1.0 | 0            | 1.0 |
| <u>UNCOATED SPECIMENS</u> |     |                      |     |     |     |     |              |     |
| Carbonation Depth mm      | 5.3 | 4.7                  | 4.9 | 7.2 | 4.5 | 6.9 | 7.6          | 8.9 |
| <u>COATED SPECIMENS</u>   |     |                      |     |     |     |     |              |     |
| Carbonation Depth mm      | 0   | 0                    | 0   | 0   | 0   | 0   | 0            | 0   |

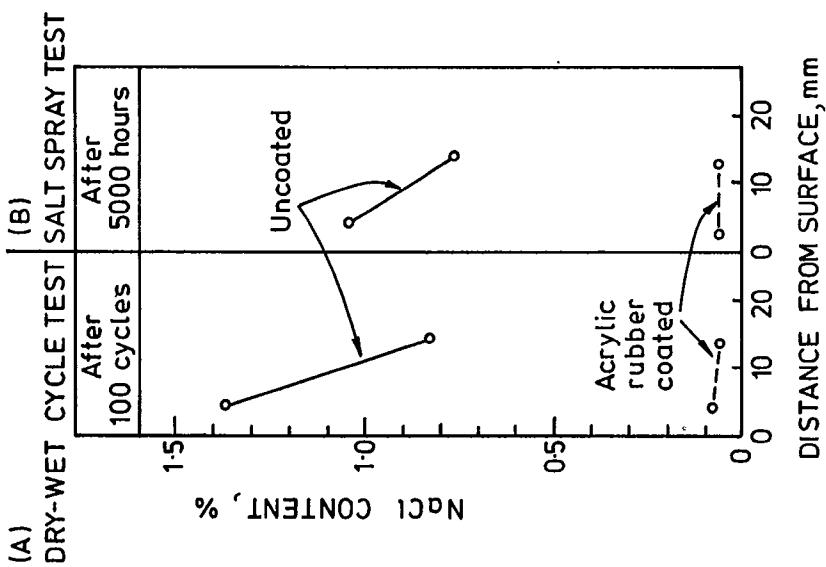


Fig. 1--Distribution of chlorides -- accelerated exposure tests

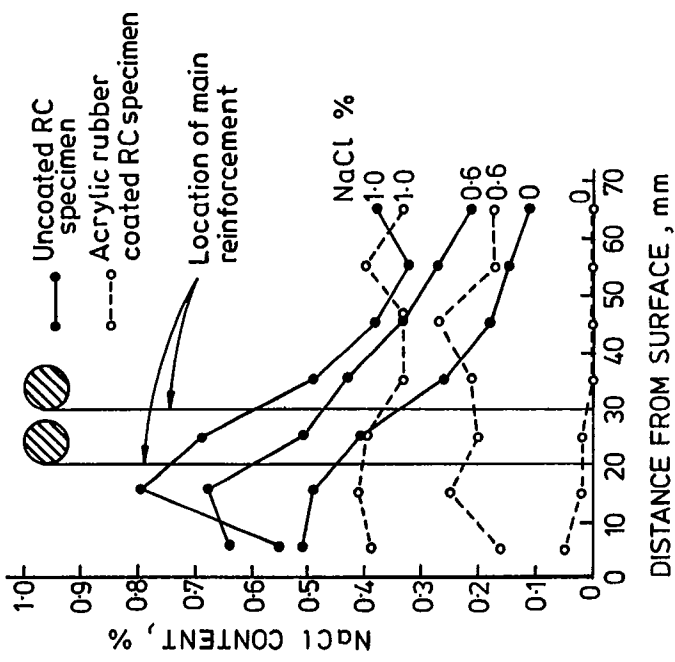


Fig. 2--Distribution of chlorides -- salt water soaking test

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# Concrete Deterioration in High Chloride and Sulfate Environment and Repair Strategies

by H. Saricimen, A. J. Al-Tayyib,  
M. Maslehuddin, and M. Shamim

**Synopsis:** During the mid 1970s there was a boom in the construction industry in Saudi Arabia. In order to meet the housing requirements, and to furnish the infrastructure needed for a growing industry, construction had to be carried out at a pace unprecedented in the country's history. In the absence of guidelines, concrete specifications from other industrialized countries were used. However, when the structures started to show signs of deterioration within a short fraction of their design life, it was realized that specifications developed for temperate conditions cannot be used in this region.

Field and laboratory studies carried out at King Fahd University of Petroleum and Minerals at Dhahran, Saudi Arabia, showed that concrete in this region should not only be designed for strength, but also for durability. Since permeability is one of the most important properties which control the durability of concrete, much emphasis should be given to the production of dense and impermeable concrete.

This paper reports the results of an investigation carried out on a number of reinforced concrete structures exposed to underground and seawater conditions, and which showed serious signs of deterioration within less than ten years after construction. The paper recommends repair procedures for the damaged structures and future practices to extend their service life.

**Keywords:** Canals; chlorides; corrosion; deterioration; harbor structures; permeability; repairs; sulfates; underground structures

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## INTRODUCTION

The environmental conditions in the Arabian Gulf countries are classified as severely corrosive. High sulfate and chloride (salt) content of ground water and soil, high sea salt and SO<sub>2</sub> content, high relative humidity in the atmosphere, and high temperature variation render the environment particularly aggressive for reinforced concrete structures. Table 1 shows the chemical analysis of typical soil and groundwater in Jubail. In addition, substandard quality of aggregate, sand and mixing water also contribute to the poor performance of reinforced concrete structures in this region.

The concrete structures reported in this paper are (1) underground electrical power manholes (UEPM), and (2) sea water cooling canals (SWCC), both situated in Jubail Industrial City in the Eastern Province of Saudi Arabia.

The electrical power distribution system in Jubail Industrial City consists of a number of switching stations interconnected by underground ductbanks throughout the load area. The reinforced concrete manholes which carry 13.8 kV, 34.5 kV, and 115 kV power cables are situated at anode points of the ductbanks, and are used for inspection of the electrical network(1,3).

There are about 3,000 of these underground power manholes in the Jubail Industrial City, spaced on a grid at approximately 200 to 300 m apart. The manholes are generally octagonal in shape and vary in size from 5 to 8 m across. They are as deep as 6 m below the groundwater table(1,3). The construction of manholes was completed between 1977 and 1986. The first major damage of the manholes was observed in 1986, 4 to 5 years after their construction. A survey showed that about 150 of these manholes had serious concrete deterioration, cracking and spalling caused by chloride ion induced rebar corrosion(3). In this investigation more than 24 buried concrete manhole structures were inspected. When this investigation was carried out in 1987, the structures were less than ten years old(1).

The sea water cooling canal system supplies cooling water for about fourteen large petrochemical plants now in operation in Jubail Industrial City. The canal has a capacity of bringing 10 million cubic meters of sea water per day to cool the industrial processes and then return it to the Arabian Gulf. The canal consists of three separate compartments: supply(S), supply/return (S/R) and return (R). Construction of the 10.6 kilometer long system used 760,000 cubic meters of reinforced concrete. The canal was constructed by different contractors during a two-year period (from 1980 to 1982) using concrete supplied from the same batch plants(2,4).

Rust stains and cracks were noticed a few months after the canal was put into service in 1982. The repair techniques used before this investigation included patching, grouting and installation of cover panels. In addition, during the second quarter of 1986, a sacrificial anode system was installed to protect the submerged sections of the canal walls. The objective of the present study was to review delamination damage in the canal system and identify causes of the damage. The structure was less than six years old when this investigation was carried out(2).

#### EXPERIMENTAL TECHNIQUES

The methodology of investigation to evaluate the performance of the concrete structures consisted of field and laboratory studies.

**Field Studies:** Field studies included visual inspection of the structures to obtain a preliminary idea about the form and nature of the deterioration. During the visual inspection, cracking and spalling patterns of the concrete and the severity of the rebar corrosion were recorded. Other measurements included the concrete cover thickness on the reinforcement using an iron depth meter, and the half-cell potential of reinforcing steel bars at a selected grid according to ASTM C 876. Whenever possible, concrete core samples were drilled from the structures for laboratory analysis.

Laboratory Studies: The concrete samples were tested in the laboratory for chloride, sulfate and cement content, and for physical properties. The concrete core samples were cut into 12-mm thick slices at various depths. They were analyzed for acid-soluble chloride and sulfate content. The cement content was determined according to BS-1881 and ASTM C 85. The samples were also tested for physical properties such as: (1) water absorption, unit weight, specific gravity, and volume of permeable voids according to ASTM C 642, and 30-minute water absorption according to BS-1881; (2) ultrasonic pulse velocity according to ASTM C 597; and (3) electrical resistivity of the concrete as retrieved and in saturated surface dry condition after 48 hours of immersion. In addition, the pH of some of the concrete samples were measured to determine the extent of carbonation.

Concrete cores with embedded rebars were tested for corrosion rate using the weight loss and electrochemical techniques. The cores were fully immersed in 2% sodium chloride solution for a week before testing by the electrochemical techniques to determine corrosion rate of the rebars.

## RESULTS AND DISCUSSION

### Case Study 1 - Underground Electrical Power Manholes

The visual inspection of the manholes showed that, in severely deteriorated structures, cracking and spalling of concrete due to rebar corrosion was evident. Water leakage through the ductbank sealant and cable holes, staining and cracking on the walls and duct edges, cracking and spalling at the ductbanks were commonly observed on these structures. Some of the structures exhibited salt deposits, concrete disintegration due to salt effect near the ductbanks, and concrete cracking and spalling on the beams. The walls had water marks, aggregate exposure due to leaching, and microcracks.

According to the physical properties data, Table 2, the concrete quality was good in most of the manhole structures tested. In very few cases, the concrete quality was of medium-to-good quality. The electrical resistivity of as-retrieved concrete was high in some cases, but low in most cases, indicating a risk of high corrosion rates due to galvanic currents in the concrete components of the manholes. The low resistivities measured in the as-retrieved samples may be due to high salt concentration and/or high moisture content probably caused by recent flooding in some of the manholes. The electrical resistivity of concrete cores in a saturated condition was low for all the manhole structures. These results clearly show that intermittent flooding will always increase the risk of high corrosion rates in the manhole structures.

The half-cell potential values, Figure 1, indicated a high probability of active corrosion of the reinforcements in the base slab, lower parts of the walls up to a height of 25 to 50 cm above the base, and the roof slabs around the chimneys in almost all manholes. These are the concrete components of the manholes most likely to come into contact with condensation water, leakage water, flood water and/or ground water in and around the structures. On some of the walls, the half-cell potentials indicated active corrosion of reinforcements up to a height of 230 cm. The reinforcement in the upper 30 to 70 cm of most of the walls was in a state of inactivity or uncertain corrosion.

The chloride ion concentrations in the base slabs and walls of the manholes were several times the allowable threshold concentration (i.e., 0.15% by wt. of cement or 0.03% by wt. of concrete) usually to a depth of 100 mm, as shown in Figure 2a. In most cases, the concentration was also above the threshold value at 75 mm depth, the level of reinforcement. Generally, the chloride ion concentration in the walls increased towards the bottom of the manholes, and decreased with depth from the surface of the concrete components. In the roof slabs and beams, it was below the threshold level at most points.

The sulfate ion concentration was either equal to or below the threshold concentration of 0.6% by wt. of concrete in most of the components of the manholes as shown in Figure 2b.

#### Case Study 2 - Sea Water Cooling Canals

The visual inspection of the three canals in the supply compartment showed damage ranging from rust staining and small cracks to serious cracking and delaminations. Panels installed under all the contracts experienced damage, with damaged areas ranging from as little as 0.1 to about 36 square meters per panel. As shown in Table 3, the total damaged area was the highest (4.2%) in the S/R compartment, and the least (1.6%) in the R compartment.

The physical test results, summarized in Table 4, showed that concrete used in the construction of the canals is of good quality for conventional concrete work but may not be adequate for prolonged and constant exposure to a high chloride environment.

The cover thickness over rebars showed great variation from contractor to contractor. The area with inadequate cover ranged between 9 to 68% over most of the panels. However, the short time required to attain the threshold chloride level indicated that the presence or lack of adequate cover was not a major contributing factor to the early onset of damage (see Figure 5).

Most of the half-cell potential measurements, as shown in Table 5, indicated active corrosion taking place on rebars in

the panels below 1.5 meters from the top of the panels. The bottom half of the supply canal had a sacrificial anode cathodic protection (SACP) system which was installed in June 1986. However, since the panel walls were dry when the half-cell potential survey was carried out, the potential values obtained are not reliable measurements for a system that is being protected.

As shown in Figure 3, the chloride ion concentration had a high value of 1.7% by wt. of cement at the panel surface, decreased with increasing depth from the surface. At a nominal rebar position of 60 mm from the surface, the chloride level was 0.45% by wt. of cement which is well above the threshold level (0.15% by wt. of cement) necessary for corrosion initiation.

Figure 4 shows typical average chloride ion concentration profiles above and below the water level in the supply compartment. Below the water level, the chloride ion concentration was very high within the first 30 mm of concrete cover but decreased rapidly with depth from the surface. At the 60 mm rebar position, the chloride concentration was about 0.35% by wt. of cement. The chloride ion concentration profile above the water level exhibited a similar behavior to that below the water level. The high chloride ion concentration at the surface above the water level may be attributed to wetting/evaporation conditions.

The chloride level at the 60 mm depth after 39.2 months was three times that after 7.5 months in service, as shown in Figure 5. In general, it took less than a year for the chloride concentration at the rebar level in most of the panels to reach the critical level of 0.15% by wt. of cement.

The sulfate content of concrete samples retrieved from the canal walls varied in the range from 0.35 to 0.93% by wt. of concrete as shown in Table 6. In only a few cases did the sulfate concentration exceed the sulfate threshold value of 0.60% by wt. of concrete. Moreover, these concentrations do not raise much concern, as Type V cement was used throughout the construction.

The pH values of the concrete varied in the range of 12.6 to 13.2, which indicated absence of significant carbonation in the concrete.

Four cores with unexposed embedded rebars were tested for corrosion rate. Prior to the testing, the cores were fully immersed in 2% sodium chloride solution for a week, then they were tested using the electrochemical technique. The results are compared in Figure 6, with those obtained by Carl E. Locke, who has soaked the reinforced concrete samples in salt (NaCl) solutions for two months after five days of curing prior to corrosion rate measurement. For comparable chloride content, cores from the sea water canal had higher corrosion rates than the tests carried out by Carl E. Locke(5), indicating poor quality of the concrete in the sea water canal.

## REPAIR AND CORROSION MITIGATION

The rehabilitation program(1,3) for the manholes considered three options: (i) civil repairs of less damaged manholes after removal of deteriorated concrete; (ii) cathodic protection of reinforcement to prevent further damage of the structures; and (iii) use of coatings to stop ingress of moisture, oxygen and chloride ions into the concrete. The civil repair methods included: (a) removal of all delaminated and unsound concrete by chipping and sand blasting; (b) exposure of all steel reinforcements where chloride contamination is present, and cleaning by sand blasting; (c) application of epoxy coatings on the cleaned reinforcement; and (d) repair of the walls to original dimensions with epoxy modified concrete.

Based on the practical and economic viability of the situation, it was decided that the shallower and less congested manholes would be rehabilitated by reconstruction of another manhole inside the existing and deteriorating one. Deeper and more congested manholes were proposed to be repaired and protected by impressed current cathodic protection (ICCP). In view of numerous uncertainties and high installation and maintenance costs, the ICCP system may have little chance of being chosen to mitigate corrosion of reinforcement for the electrical power manholes.

Rust stains were observed on the walls of S/R and R compartments of Canal 2 within three months after hydraulic testing of the canals started (2,4). Six months later, first cracks of the walls were witnessed. The contractor had to repair and overlay about 50% of Canal 2 wall panels with new concrete when substantial areas of less than minimum cover (60 mm) for rebars were identified.

For the panels which had light damage, the damaged concrete was chipped off to a depth behind the reinforcement. The steel reinforcement was cleaned by sand blasting. The wall face was reinforced by injecting grout to original dimensions. For the panels where substantial height and wall area was involved, a complete overlay with wire mesh was cast over the original panel after the mesh was tied to the structural reinforcement for electrical continuity. Concurrently, a program was initiated to investigate the causes of rapid corrosion in order to take precautions to stop it(4).

As a result of the initial investigation, it was decided to try a number of techniques to mitigate corrosion of reinforcement in the dividing walls. These techniques involved: (i) coatings only, (ii) sacrificial anode cathodic protection (SACP) system, and (iii) their combination on immersed and splash zone areas of the walls. On the dry parts of the walls above waterline, coatings and an impressed current cathodic protection (ICCP) system were chosen(4).

The results of the trial programs showed that (i) coatings were effective for providing oxygen starvation and arresting

corrosion of reinforcement on submerged parts of the walls; (ii) Al-Zn-In SACP was capable of mitigating corrosion of submerged portions of the canal walls; (iii) for coatings plus SACP, the coatings were found to prevent sacrificial anode CP current from reaching the reinforcing steel; (iv) it was possible to protect the dry parts of the dividing walls with a durable and effective ICCP system which was compatible with the sacrificial anode system(4).

#### CONCLUSIONS AND RECOMMENDATIONS

The study showed that reinforced concrete structures which may be rated as good-to-very good quality for use in temperate conditions (according to the international specifications) may not be durable enough to withstand the aggressive environment in the Arabian Gulf region.

In such an environment, which contains high levels of chloride and sulfate ions, the permeability of concrete becomes one of the most important parameters governing the durability of reinforced concrete structures.

The chloride ions were more harmful to reinforced concrete durability than sulfate ions in the structures inspected. The chloride ion concentration at the rebar level in the reinforced concrete structures in this region reached the allowable concentration in about a year. This initiated the corrosion of rebars which is followed by cracking, spalling and delamination of concrete.

It is, therefore, recommended that to improve durability, the engineers in this region should follow the specifications diligently and provide low permeability concrete. They should: (i) use high quality and salt-free raw materials; (ii) adopt appropriate mix design parameters, and use pozzolanic admixtures as partial replacement of cement; (iii) pursue the specified curing procedures industriously; and (iv) protect the structures against chloride ion ingress and chloride induced corrosion by using (a) coated rebars, (b) concrete coatings or penetrants, and (c) cathodic protection for the reinforcement.

#### ACKNOWLEDGEMENT

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TABLE 1--ANALYSIS OF SOIL AND GROUNDWATER AROUND THE UNDERGROUND MANHOLES

|                                      | <u>Soil</u>    | <u>Ground Water</u> |
|--------------------------------------|----------------|---------------------|
| pH                                   | 6.3 - 8.3      | 6.2 - 8.0           |
| Chlorides (Cl), ppm                  | 350 - 69,500   | 560 - 105,000       |
| Sulfates (SO <sub>4</sub> ), ppm     | 350 - 44,000   | 1,600 - 11,800      |
| TD Salts, ppm                        | 5,360 - 77,300 | Data not available  |
| Electrical Conductivity (in mhos/cm) | 2.0 - 179      | 1.9 - 310           |

TABLE 2--PHYSICAL PROPERTIES OF CONCRETE FROM  
A TYPICAL MANHOLE STRUCTURE

| <u>Physical Property</u>             | <u>Typical Value</u> |
|--------------------------------------|----------------------|
| Dry Unit Weight (kg/m <sup>3</sup> ) | 2,275                |
| Pulse Velocity (km/s)                | 4.57                 |
| Absorption (%):                      |                      |
| (30 min. immersion)                  | 1.08                 |
| (48 hr. immersion)                   | 2.31                 |
| Volume of permeable voids (%)        | 5.79                 |
| Electrical Resistivity (ohm/cm)      |                      |
| As-Retrieved                         | 20,822               |
| 48 hr immersion                      | 5,316                |

TABLE 3--DAMAGE SUMMARY OF SEA WATER COOLING CANALS

| Canal/Compartment   | Average Time<br>in Service<br>Damage<br>(months) | Total<br>Area<br>(m <sup>2</sup> ) | Damaged<br>Area<br>(m <sup>2</sup> ) | %   |
|---------------------|--------------------------------------------------|------------------------------------|--------------------------------------|-----|
| Supply (S) *        | 35.0                                             | 78,150                             | 1,546                                | 2.0 |
| Supply/Return (S/R) | 38.4                                             | 133,256                            | 5,578                                | 4.2 |
| Return (R)          | 30.5                                             | 73,436                             | 1,199                                | 1.6 |

\* The supply compartment was in service with a sacrificial anode system 3-4 months before the above data were taken.

TABLE 4--RANGE OF PHYSICAL PROPERTIES OF CONCRETE  
MEASURED ON SAMPLES FROM SEA WATER COOLING CANALS

| Physical Property                | Range of Values |
|----------------------------------|-----------------|
| Cement content (%)               | 10.80 - 36.29   |
| Specific Gravity                 |                 |
| Dry                              | 2.20 - 2.34     |
| Apparent                         | 2.60 - 2.68     |
| Unit Weight (kg/m <sup>3</sup> ) |                 |
| Dry                              | 2,202 - 2,286   |
| 48 hr immersion                  | 2,371 - 2,456   |
| Pulse velocity (km/s)            | 3.7 - 4.5       |
| Absorption (%):                  |                 |
| 30 min. immersion                | 1.0 - 3.0       |
| 48 hr. immersion                 | 4.68 - 7.34     |
| Value of Permeable Voids (%)     | 11.36 - 16.67   |
| Electrical Resistivity (ohm/cm): |                 |
| As-retrieved                     | 12990 - 22714   |
| 48 hr. soaking                   | 2955 - 8086     |

TABLE 5--SUMMARY OF MEASURED HALF-CELL POTENTIALS  
(CANAL #2, S-COMPARTMENT, PANEL #135)

| Height from<br>Canal Base (m)       | Half-Cell Potential Readings (-mV) |     |     |     |     |     |     |     |     |
|-------------------------------------|------------------------------------|-----|-----|-----|-----|-----|-----|-----|-----|
|                                     | 0.02                               | 1.0 | 2.0 | 3.0 | 4.0 | 4.5 | 5.0 | 5.5 | 6.5 |
| Distance<br>along<br>Canal Base (m) |                                    |     |     |     |     |     |     |     |     |
| 0.00                                | 822                                | 781 | 705 | 506 | 423 | 357 | 283 | 30  | 6   |
| 0.50                                | 824                                | 777 | 621 | 491 | 421 | 343 | 278 | 47  | 11  |
| 1.00                                | 837                                | 756 | 608 | 487 | 417 | 351 | 280 | 26  | 3   |
| 1.50                                | 849                                | 637 | 605 | 495 | 424 | 340 | 294 | 33  | 14  |
| 2.00                                | 859                                | 737 | 579 | 540 | 408 | 350 | 290 | 22  | 49  |

TABLE 6--VARIATION OF pH AND SULFATE ION CONCENTRATION WITH DEPTH IN CANAL WALLS

| Depth (mm)<br>from wall<br>surface | pH<br>Panel No. 092<br>South Wall |              | Acid Soluble Sulfate Ion Concen-<br>tration - % by wt. of concrete |               |
|------------------------------------|-----------------------------------|--------------|--------------------------------------------------------------------|---------------|
|                                    | Sample<br>#3                      | Sample<br>#4 | Panel No. 092                                                      |               |
|                                    |                                   |              | South<br>Wall                                                      | North<br>Wall |
| 0                                  | 12.7                              | 12.6         | 0.557                                                              | 0.933         |
| 15                                 | 13.2                              | 13.1         | 0.478                                                              | 0.688         |
| 30                                 | 13.2                              | 13.1         | 0.458                                                              | 0.639         |
| 60                                 | 13.1                              | 13.1         | 0.349                                                              | 0.545         |
| 90                                 | 13.1                              | 13.0         | 0.499                                                              | 0.584         |
| 120                                | 13.0                              | 13.0         | 0.483                                                              | 0.505         |

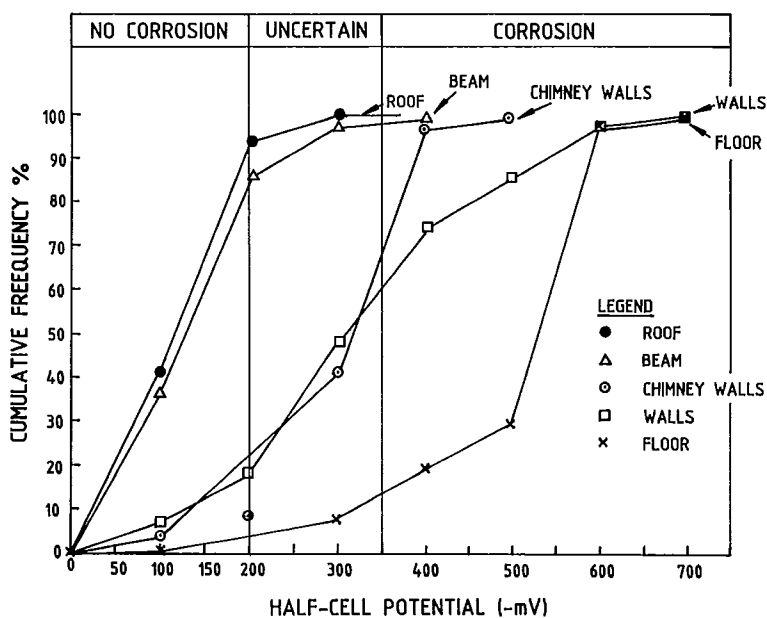


Fig. 1--Half-cell potential cumulative frequency plots for various components of a UEPM

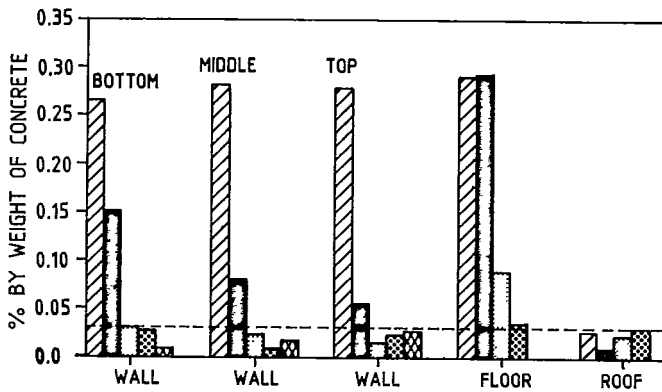


Fig. 2a--Acid soluble chloride ion concentration variation with depth

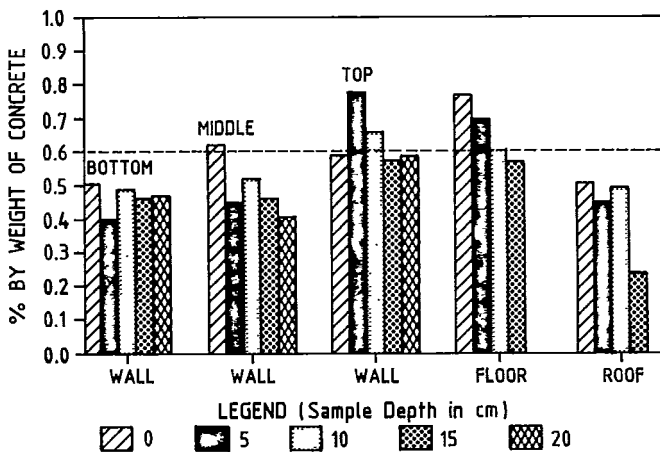


Fig. 2b--Acid soluble sulfate ion concentration variation with depth in bottom, middle and top of the walls, roof and floor of an UEPM

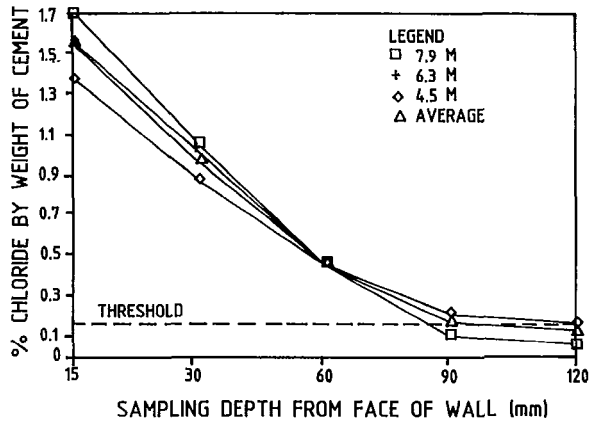


Fig. 3--Average chloride ion concentration profiles of an S-compartment panel of SWCC at various heights from the base slab. The panel was in service for 23 months at a water level of 7.26 m

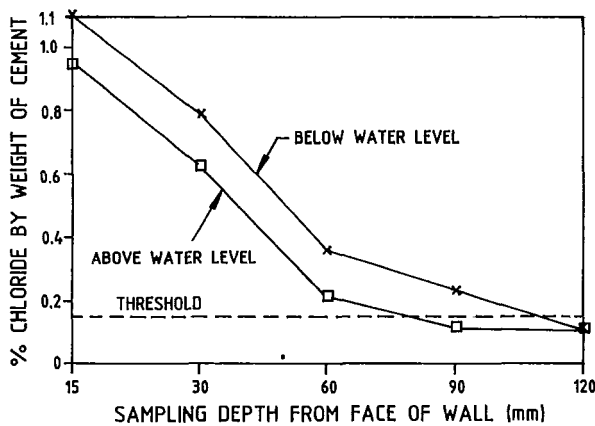


Fig. 4--Average chloride ion concentration profiles above and below the water level in S-compartment of SWCC after 7.5 months in service

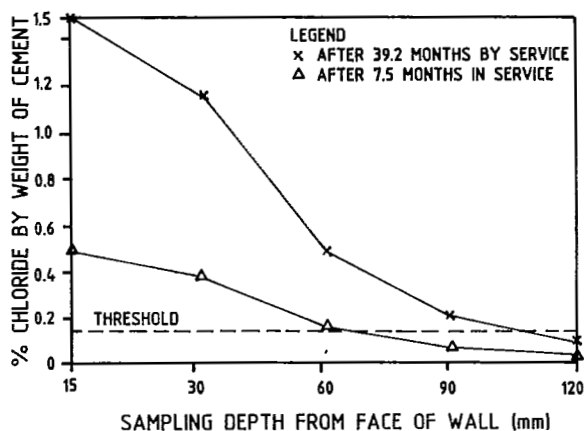


Fig. 5--Average chloride ion concentration profiles of an S-compartment panel for different service durations

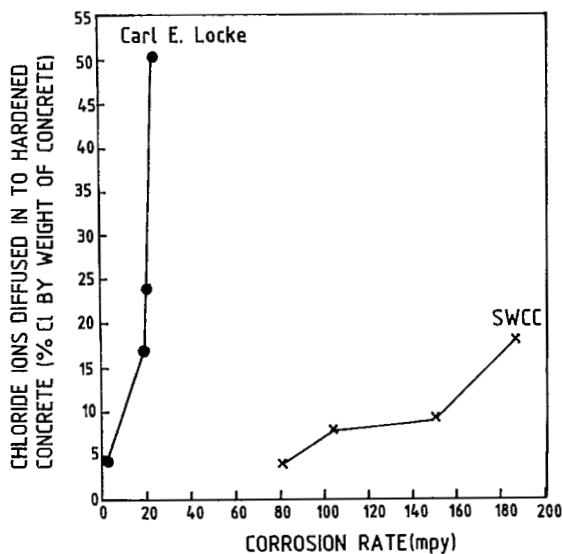


Fig. 6--Corrosion rates of embedded rebars retrieved from SWCC -- Compared with Locke tests (5)

SP 128-3

# Comparative Evaluation of Nondestructive Testers of Hardened Concrete

by K. W. Nasser and P. S. H. Lai

**Synopsis:** This paper describes an investigation which was carried out to determine the within-test variability of various nondestructive test methods and their correlation with the corresponding compressive strength. The nondestructive test methods that were studied were the pin penetration, the ultrasonic pulse velocity and the nuclear density gauge. The tests were performed on solid concrete blocks, cylinders and reinforced concrete slab at different ages.

The within-test variability of the direct readings of the ultrasonic pulse velocity were small compared to the pin penetration and nuclear density gauge. However, when the readings were translated into strength, the within-test variabilities of the standard compression test was about the same as the other methods. The relationship between strength and the pin penetration and pulse velocity readings were good, while the relationship of the density of fresh and hardened concrete with the nuclear gauge readings were satisfactory.

**Keywords:** Compressive strength; concrete blocks; concrete slabs; cylinders; density (mass/volume); hardened concretes; nondestructive tests; tests; variability

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## INTRODUCTION

It is important to determine the quality of concrete and to assess its in-situ strength in order to establish the safety of concrete structures. In practice, the determination of the potential strength of concrete is generally done by means of testing the standard 6x12 in. (150x300 mm) cylinders. Since the conditions under which the test cylinders are made and cured differ from that of the concrete in the structure, the strength of the test cylinders is different from that of the concrete in the structure. Therefore, there is need for reliable testing methods to determine the in-situ strength of concrete. Several nondestructive methods are available in the market (1), but recently, a pin penetration tester (2, 3, 4) and a nuclear density gauge tester (5) were described and advocated as additional useful tools to estimate the early strength of concrete.

In this paper, results obtained by three nondestructive methods, the ultrasonic pulse velocity apparatus (ASTM C 597), the pin penetration tester (ASTM C 803) (2) and the nuclear density gauge (ASTM C 1040) (5) will be presented. Their performance will be evaluated and their results will be compared to the compressive strength obtained from the test of standard cylinders.

## BACKGROUND INFORMATION

The pin penetration tester consists basically of a spring loaded hammer that drives a small pin into the concrete surface which leaves a crater thereof. The pin remains attached to the hammer and the depth of the crater formed during penetration is measured with a dial gauge. It has been established that compressive strength is an inverse linear function of pin penetration readings (2, 3, 4).

The use of a nuclear density gauge on concrete is based upon the principle that the compressive strength of concrete can be determined as a function of the concrete density that can be measured by a nuclear density gauge (5). In the present investigation, the concrete was tested with a nuclear device in two different modes: backscatter and 2-inch (50-mm) direct transmission modes. In the backscatter mode, the tip of the rod which contains the source of radioactive isotope is situated just at the surface of the concrete where the base of the gauge is seated, while in the 2-inch direct transmission mode, the source rod penetrates into the concrete to a depth of 2 inches (50

mm) below the surface of the concrete. The nuclear density gauge does not provide a direct reading of the material's density, but the user can refer to a calibration graph which relates the gauge's output parameter to the material density. The output parameter is called 'count ratio' and is calculated as the ratio of the number of 'counts' produced by the gauge on the material being tested such as concrete, to the number of counts produced by the gauge on a standard specimen. This ratio provides a meaningful density determination over the life of the gauge even though the level of radiation being emitted by the source drops with time. A Troxler model 3440 nuclear density gauge was used in this investigation.

Detailed description and outline of procedures of the pin penetration tester and the nuclear density gauge are found in recent literature (2, 5), while description of the ultrasonic pulse velocity apparatus which has been on the market for a long time, is available from several sources (1, 2).

## TEST PROGRAM

### Concrete Materials and Mix Proportions

This investigation was piggy-backed on research projects that were being carried out in the testing laboratory at the University of Saskatchewan and hence the authors had no input or control over the type of the concrete that was available for testing. The projects had used two similar concrete mixes designated A and B; mix A was a little different from mix B in that it contained accelerating agent of 2.0% by weight of cement. The concrete was obtained from a local ready-mix supplier that used ASTM type I cement in both mixes. Details of the mix proportions are given in Table 1.

### Preparation and Curing of Test Specimens

Three 18x18x6-in. (450x450x150-mm) and three 24x24x4-in. (600x600x100-mm) concrete blocks were cast from mix A, while two 18x18x6-in. and one 24x24x4-in. concrete blocks were cast from mix B. Also, a number of 3x6-in. (75x150-mm) and 6x12-in. (150x300-mm) cylinders were cast from each mix. Mixes A and B were used in two research projects involving testing of a full size stub-girder with a concrete slab on ribbed metal deck and a reinforced concrete beam. The size of the slab was 7.4 ft (2250 mm) wide, 45.1 ft (13750 mm) long and its average thickness was 4 in. (100 mm) and it was reinforced with longitudinal reinforcement and with a double layer of wire mesh. The size of the reinforced concrete beam was 13 in. (330 mm) wide, 20 in. (510 mm) deep and 22 ft (6700 mm) long.

The slab and block forms were filled with concrete, vibrated with an internal vibrator, levelled and trowelled, while the concrete cylinders were cast in cardboard molds and were rodded 25 times in three layers. The slab, the blocks and the cylinders were covered with plastic sheets to minimize moisture loss. After 24 hours, the concrete cylinders were stripped and stored in a water tank at a temperature of 21°C until tested, while the concrete blocks and slab were left in the laboratory at a temperature of 20±2°C and a relative humidity

of 50±5%. After 7 days of curing, the plastic sheets and forms were removed from the concrete blocks.

### **Testing of Concrete Specimens**

The tests were performed on the specimens at different ages: i.e., fresh condition, 1, 2, 3, 7, 14, 28 and 56 days. The size of the concrete blocks that were tested with the nuclear density gauge in the backscatter mode was 18x18x6-in. (450x450x150-mm), while those that were tested in the direct transmission mode was 24x24x4-in. (600x600x100-mm). In order to maintain a clean gauge when making measurements on the surface of fresh concrete, each concrete block was covered with a sheet of 2-mm polyethylene. Also the blocks that were tested with the gauge in the 2-inch (50-mm) direct transmission mode had four holes into each of which the source rod was placed once at a time in a random sequence. The holes were formed and lined with a polyethylene cylindrical insert of 4 in. (100 mm) in length and 0.75 in. (19 mm) in diameter and this size was slightly larger than the diameter of the source rod. Two days after casting, the forms were stripped and the concrete blocks were turned upside down. Since the inside surfaces of the form were clad with arborite, the bottom surface of the block was very smooth and had ready-formed holes cast in it for the source rod to penetrate when testing the hardened concrete blocks. The reinforced concrete slab was tested with the nuclear density gauge in a backscatter mode and the tests were performed at 10 locations spaced evenly across its length.

The pin penetration tests were performed on the circumferential surface of the concrete cylinders as well as on the bottom surface of the concrete blocks. Each cylinder was seated on a semi-circular wooden base which provided support to the cylinder during testing. An average of 6 readings were taken on each specimen.

The pulse velocity test was conducted with a portable digital ultrasonic pulse velocity apparatus. A direct transmission mode was employed with a path length of either 18 or 24 in. (450 or 600 mm) for the blocks and of either 6 or 12 in. (150 or 300 mm) for the cylinders. An indirect transmission mode was used for the reinforced slab. The pulse velocity measurements from each block were the average of two readings, taken in a direction perpendicular to the direction of casting while only one reading was enough for each cylinder in a direction parallel to its casting. The slab was tested in a backscatter mode with a path length of 12 in. (300 mm). The locations where pulse velocity and nuclear density tests were performed on the slab were identical to each other. Four measurements were made at each location.

Concrete cylinders were also used for determination of the compressive strength. The cylinders were capped with sulphur and were tested according to the procedure specified in ASTM C 39.

## **ANALYSIS OF TEST RESULTS**

Statistical analyses were carried out to obtain information on the accuracy of each non-destructive test method on the concrete. The readings

from the non-destructive test results were first converted to compressive strength using appropriate calibration curves (3, 5). The within-test variability for each method is expressed in terms of its coefficient of variation while correlation between compressive strength and the nondestructive test results was obtained by means of regression analysis. Summaries of the calculations were presented in Tables 2 and 3.

As can be seen in Table 1, the difference in the densities of the two mixes and their changes with time were very small in this project and hence, they were not enough to cause significant change in the nuclear gauge readings. Therefore, the conversion of count ratio to 28-day strength was done for the density of fresh concrete only using the 2-in. (50 mm) direct transmission mode. Neville (6) has shown that, for the relatively narrow range of density generally encountered in structural quality concrete (excluding those with light weight aggregates), the relationship between compressive strength and density is linear. Figures 1 and 2 show histograms for the count ratio of the two modes and they are typical of test results for concrete. Tables 2 and 3 also showed the coefficient of variation of the nondestructive test results; their values for the tests on the blocks for the pin penetration and pulse velocity tests were close to that of the compressive strength test results. On the other hand, the corresponding values for the cylinders and the slab were higher. These differences may be attributed to the reinforcing steel in the slab and the wooden base used in conjunction with the cylinders.

A tangible relationship between the nondestructive test results and the corresponding compressive strength was investigated. It was found that a linear relationship of the form  $Y=a+bX$  was good for the pin penetration results, while a power equation in the form of  $Y=aX^b$  was good for the ultrasonic pulse velocities. Data obtained from either cylinders, blocks or slab were correlated both individually and in combination (where applicable). The compressive strength was plotted against the pin penetration and the pulse velocity as shown in Figures 3 and 4 respectively. The coefficient of correlation and constants 'a' and 'b' were given in Tables 4 and 5. In general, both the pin penetration and the pulse velocity tests correlated well with the compressive strength. When all the data from each test method were combined, the coefficients of correlation for pin penetration and pulse velocity were 0.81 and 0.79. If the slab data was excluded from the analysis, the rest of the results would give a coefficient of correlation of 0.87. From this observation, it is evident that the presence of steel reinforcement in the slab did affect the result in this test. Similar observations were also reported by Bungey (7).

## CONCLUSIONS

The following conclusions can be drawn from the results of the three nondestructive test methods of hardened concrete used in this investigation:

(1) After converting the nondestructive test readings on the concrete blocks into estimated compressive strength, the within-test variability of the pin penetration and pulse velocity tests were close to that of the standard compression test.

(2) Linear and power relationships were established to relate the nondestructive test results to the ultimate compressive strength of concrete. The equations were:

(a) pin penetration test

$$C = 8825 - 29.75P \quad r = 0.81$$

(b) ultrasonic pulse velocity test

$$\text{including slab} \quad C = 0.000956V^{5.70} \quad r = 0.79$$

$$\text{excluding slab} \quad C = 0.000243V^{6.21} \quad r = 0.87$$

where:

C=ultimate compressive strength in psi

P=pin penetration in inch  $\times 10^{-3}$

V=ultrasonic pulse velocity ft/sec  $\times 10^3$

(3) The pin penetration test gave the best coefficient of correlation when all the data were analyzed together, while the ultrasonic pulse velocity gave a better value when the results from the reinforced slab were discarded.

(4) The nuclear density test can be used to estimate the density of fresh and hardened concrete.

### ACKNOWLEDGEMENTS

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TABLE 1--SUMMARY OF MIX DESIGN AND CONTROL TEST RESULTS

| Description                          | Mixture designation |      |
|--------------------------------------|---------------------|------|
|                                      | A                   | B    |
| Cement, kg/m <sup>3</sup>            | 310                 | 310  |
| Fine aggregate, kg/m <sup>3</sup>    | 815                 | 800  |
| Coarse aggregate, kg/m <sup>3</sup>  | 1105                | 1100 |
| Water, kg/m <sup>3</sup>             | 130                 | 130  |
| Pozzololith, % of cement             | 2                   | nil  |
| W:C                                  | 0.42                | 0.42 |
| A:C                                  | 6.19                | 6.12 |
| Air content, %                       | 2.9                 | 5.8  |
| Slump, mm                            | 83                  | 114  |
| Unit weight, kg/m <sup>3</sup> fresh | 2400                | 2320 |
| 3-day                                | 2380                | --   |
| 7-day                                | 2360                | 2290 |
| 14-day                               | 2380                | --   |
| 28-day                               | 2370                | 2240 |
| 56-day                               | 2370                | 2230 |

Conversion factors: 1 lb/cu.ft = 16.02 kg/m<sup>3</sup>  
 1 in. = 25.4 mm