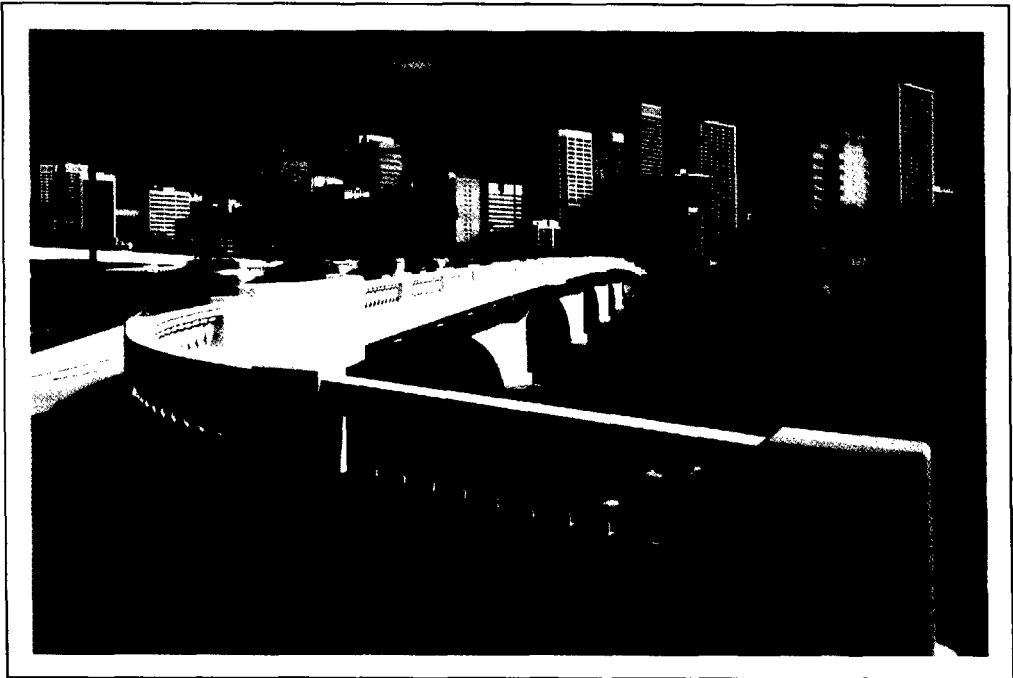


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**ADVANCES IN
CONCRETE
TECHNOLOGY**

**PROGRÈS DANS LE
DOMAINE DE LA
TECHNOLOGIE DU BÉTON**

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Editor

V. M. Malhotra

SP - 179

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PREFACE

The Canada Centre for Mineral and Energy Technology (CANMET) of Natural Resources Canada, Ottawa, has played a significant role in Canada for over 35 years in the broad area of concrete technology. In recent years, CANMET has become involved increasingly in research and development dealing with the supplementary cementing materials, high-performance normal-weight and light-weight concretes and alkali-aggregate reactions. As a part of CANMET's technology transfer program, an international symposium on Advances in Concrete Technology was sponsored jointly with the American Concrete Institute and other organizations, and held in Athens in May 1992. In June 1995, CANMET, in association with the American Concrete Institute and other organizations in Canada and the U.S.A. sponsored the Second CANMET/ACI International Symposium on Advances in Concrete Technology in Las Vegas, U.S.A. For the Athens symposium, the CANMET publication *Advances in Concrete Technology* constituted the proceedings of the symposium; for the Las Vegas symposium, the proceedings were published by the American Concrete Institute as ACI SP-154.

In August 1997, CANMET, in association with the American Concrete Institute and other organizations in Canada and New Zealand, sponsored the Third CANMET/ACI International Symposium on Advances in Concrete Technology in Auckland, New Zealand. The main purpose of the symposium was to bring together representatives from industry, universities, and government agencies to present the latest information in the subject area of the symposium, and to explore new areas of needed research and development. Thirty-three refereed papers from 15 countries were presented. In addition to the refereed papers, more than 20 other papers were presented and distributed at the symposium. The proceedings were published as ACI SP-171.

In June 1998, CANMET, in association with the American Concrete Institute, the Japan Concrete Institute, and several other organizations in Canada and Japan, sponsored the Fourth CANMET/ACI/JCI International Conference on Recent Advances in Concrete Technology in Tokushima, Japan. More than 80 papers from 20 countries were received and reviewed in accordance with the policies of the American Concrete Institute. Sixty-two refereed papers were accepted for presentation at the conference and for publication in ACI SP-179. In addition to the refereed

papers, more than 30 other papers were presented and distributed at the symposium.

Thanks are extended to the members of the CANMET/ACI review panel which met in Honolulu, Hawaii in August 1997 to review the papers. Without their prompt review and constructive comments, it would not have been possible to publish this ACI special publication for distribution at the symposium in Tokushima. The cooperation of the authors in accepting reviewers' suggestions, and in revising their manuscripts accordingly, is greatly appreciated. Particular thanks are extended to H.S. Wilson, Consultant, Ottawa, and G.D. Brearly and M. Venturino, both of CANMET, for their help in the processing of the draft manuscripts.

The help of Messrs. A. Bilodeau (Chairman), B. Fournier, and R. Chevrier (Members) of the Slide Review Panel is greatly appreciated.

H. Beavers, Managing Editor, ACI and her staff deserve very special mention for their assistance in getting this publication ready on time for distribution in Tokushima, Japan.

V.M. Malhotra, P. Eng.

Editor

Chairman, CANMET/ACI/JCI Fourth International Conference
on Recent Advances in Concrete Technology

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Some Durability Considerations in the Design of the Confederation Bridge

by W.S. Langley, G. Forbes and
E. Tromposch

Synopsis: Transportation to and from Prince Edward Island, Canada's smallest province, has been by ferry for the past century. The cost to operate the ferry system became an increasing burden for the Canadian government and a proposal call was solicited in 1987 to the private sector to construct and operate an alternate system to the Province of Prince Edward Island. A proposal was accepted for a private consortium to build a 12.9 km bridge from the mainland to Prince Edward Island. The bridge was completed in May, 1997.

A precast concrete, post-tensioned segmental box girder structure was selected for the site. A requirement of the Government of Canada was that the design and construction of the bridge provide a structure with a design life of 100 years. The bridge is located in a harsh marine environment, with some 100 annual cycles of freezing and thawing. Ice floes which originate in Northern waters pass through the Northumberland Strait in the winter and early spring months. Water temperatures vary from about -2 °C in the winter months to +18 °C in summer. The salinity of the water in the Northumberland Strait is approximately 3.5%.

This paper presents some of the durability concerns which were considered during the design and construction of the bridge and describes how these concerns were addressed.

Keywords: abrasion; aggregates; alkalis; chlorides; curing; durability; fly ash; freeze thaw durability; scaling; silica fume; sulphates.

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INTRODUCTION

The Confederation Bridge completed in May 1997, across the Northumberland Strait, is a complex multi-span concrete box girder structure 12.9 km in length. It consists of multiple fixed frames separated by expansion joints and hinges so as to accommodate the movements associated with loading, creep, shrinkage and thermal effects. The structure is designed for a service life of 100 years in a harsh marine environment. Many design and construction features are specific to this structure. The piers for the main girders are founded in upwards of 35 meters of water and in current velocities approaching 2 meters per second, thus, accessibility for monitoring and repair of defects, if required, would be extremely difficult.

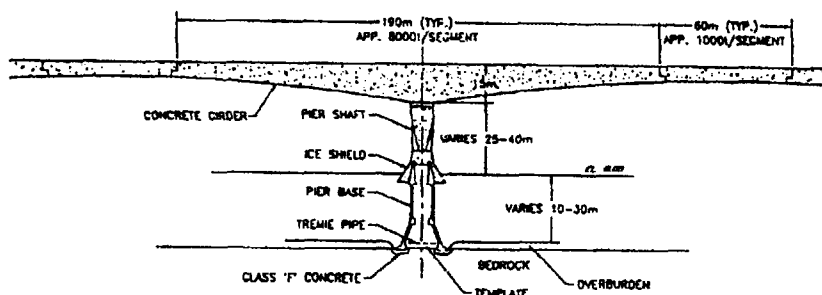
The approach section on Prince Edward Island (Canada's smallest province) consists of six spans of 93 meters each, founded on sandstone bedrock with shear keys to resist the lateral loads of wind, waves and ice. The west approach piers for the 14 spans of 93 meters each, are founded on vertical and battered, concrete filled pipe piles. The pier shafts were constructed of match cast segments which were post tensioned vertically in place. The bridge has a deck width of 11 meters which allows for one traffic lane and one wide shoulder in each direction. The bridge has a typical clearance of 49 meters above sea level with the exception of a navigation span which is 60 meters above water. The main bridge girders were segmentally constructed as well, using a balanced cantilever method.

The main bridge consists of 44 spans of 250 meters each. The piers for the main spans were cast on land in the Prince Edward Island staging facility, moved to a jetty by an innovative hydraulic slider technique, and then transported to the site by a heavy lift marine vessel. These piers were located on three previously installed concrete positioning pads, and further supported by tremie concrete between the pier base and bedrock.

The main span girders were match cast and post tensioned at the staging facility and then transported to the site. The spans were constructed using two types of components: a 190 meter cantilever unit and a 60 meter drop-in span. The completed cantilever units weigh upwards of 8000 tonnes. The cantilever girder depth varied from 14.5 meters at the piers to 4.5 meters at the cantilever tip.

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The location of the Confederation Bridge is shown in Fig. 1 and a typical pier foundation, pier shaft, main span girder and drop-in section is shown in the sketch below.



The Confederation Bridge crosses the Northumberland Strait from the mainland of Canada to Canada's smallest province, Prince Edward Island. The placid summer waters of the strait, reverts to a cold hostile marine environment in winter and early spring, as the Gulf ice moves through the strait. This places a high demand on the quality of concrete materials, production and construction. Development of ice in the strait is of little consequence compared to the affects of tide and wind driven ridged and rafted ice. The size and configuration of the bridge pier foundations are largely dependent on the imposed ice loads. The design utilizes an ice breaking cone at water level to reduce the imposed loads by causing the ice to bend upward and break in flexure. This cone or ice shield is either clad in cathodically protected steel or constructed of very high strength concrete for improved abrasion resistance.

DURABILITY OF CONCRETE

High durability is not an intrinsic property of concrete, but is achieved by a judicious choice of materials, accurate proportioning, thorough mixing, proper placing, adequate curing and protection. The main force of deterioration was considered to be that of chloride ingress and subsequent corrosion of the reinforcing steel. Other potential deterioration forces considered were alkali-aggregate reaction, sulphate attack, freezing and thawing, ice abrasion, salt scaling and cracking due to thermal gradients if these were not controlled within tolerable limits.

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The approach taken to address the durability concerns are addressed in the following subsections.

Concrete Materials

Cement - A low alkali, silica fume blended cement was used throughout all concrete manufacture. This particular cement was manufactured to strict specifications for this particular project. Approximately 130,000 tonnes of the cement were utilized.

The low alkali cement, blended with 7.5 percent silica fume was required to contain a tricalcium aluminate content between six and ten percent. This range was considered appropriate for the marine environment such that neither depassivation of the reinforcement steel or potential for sulphate attack should occur.

Fly Ash - A low calcium fly ash (ASTM Class F) was used in most of the structural concrete at 15 percent replacement by mass of the blended cement. The fly ash was found to have a beneficial affect on heat development, permeability, concrete workability and air void stability. The fly ash replacement was in the order of 29 percent in the more massive sections of the structural components, so as to minimize the potential for thermal cracking.

The properties of the silica fume blended portland cement and fly ash are shown in Table 1.

Aggregate - Aggregate comprises some 70 percent by volume of high performance concrete; thus, the long term stability of the fine and coarse aggregate is a significant factor, particularly for the 100 year design life.

Twelve sources of sand and gravel were identified in Atlantic Canada and Quebec as potential sources of concrete aggregate for the Confederation Bridge. Extensive petrographic examinations were performed so as to identify potential long term problems such as alkali-aggregate reaction, presence of pyrite, poor crystal structure, high porosity, etc. Coarse aggregate, processed from a fine grained granitic intrusion was selected for the coarse aggregate and a natural quartz sand was selected for the fine aggregate. These materials were processed, washed and classified at the source and the coarse aggregate was further washed on site after shipment and immediately prior to batching concrete.

The results of the alkali-aggregate reactivity tests are shown in Fig. 2 and 3. The expansion measured on the aggregate was well below the acceptable limit of 0.1 percent in the accelerated mortar bar test and 0.04 percent in the concrete prism test at 24 months.

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Concrete Mixtures

Approximately 360,000 cubic meters of concrete was used in the construction of the Confederation Bridge. The concrete for the various phases of construction could be divided into four basic classes as follows:

Class A	Structural concrete for main girders, drop-in spans, pier shafts, pier bases and abutments.
Class A100	Exposed ice shield concrete.
Class C	Approach pier foundation and mass concrete (concrete sections greater than one meter in thickness).
Class F	Tremie structural concrete (antiwashout admixture used in moving water).

Each of the classes of concrete (with the exception of the tremie concrete) was available with either a 20 mm or 10 mm nominal maximum size of coarse aggregate, and their usage depended on the congestion of reinforcement and clear space in the formwork.

Class A Concrete - The requirements of the Class A concrete were as follows:

Minimum cement content	450 kg/m ³
Maximum water/cementitious materials	0.34
Maximum Rapid Chloride Permeability	1000 coul.
Maximum Water Permeability @ 28 days (CANMET Method)	10 ⁻¹⁴ cm/sec
Maximum air void spacing factor	260 µm
Average air void spacing factor	< 230 µm
Design strength @ 91 days	60 MPa

The mixture proportions of the Class A concrete are shown in Table 2. It may be noted that the requirements for low permeability and low water-cementitious materials ratio override the requirements for strength. Table 3 provides physical properties of the Class A concrete as developed in the laboratory.

Class A 100 concrete - A special high performance concrete was developed for the pier shaft ice shields. The ice which moves through the Northumberland Strait during the late winter and early spring contains abrasive sediments picked up during grounding of the ice floes.

Table 2 shows the concrete mixture proportions for the Class A100 concrete.

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The concrete used in the ice shields had a very low water cementing materials ratio of 0.25, which provided a Rapid Chloride Permeability of 125 coulombs at 91 days. The average compressive strength of the A100 concrete mixture used in the ice shield was in excess of 80 MPa at 91 days. The abrasion wear was approximately 60 percent of that of the 80 MPa concrete (Class A). Fig. 4 shows the results of the surface abrasion test for various concretes using the ball bearing method as outlined in ASTM.

Class C Concrete - The requirement to maintain a thermal differential of less than 20 °C within the concrete and a maximum temperature in mass concrete of less than 70 °C was controlled by the use of a low calcium fly ash, substituted for 30 percent by mass of the total cementitious materials. The fly ash was batched as a separate ingredient at the batch plant. The tertiary blend of low alkali portland cement, silica fume and fly ash gave excellent performance and imparted a very low permeability to the concrete.

The requirements for Class C concrete were as follows:

Minimum cement content	300 kg/m ³
Maximum fly ash content	130 kg/m ³
Maximum water/cementitious materials	0.36
Maximum air void spacing factor	260 µm
Average air void spacing factor	< 230 µm
Minimum compressive strength @ 91 days	40 MPa

Table 2 shows the mixture proportions for the Class C concrete.

Class F concrete - Class F concrete was used for tremie placement in shear keys, tremie seals, and beneath the precast pier foundations as a support for the bridge piers on the sandstone bedrock. The tremie concrete was placed in a variety of conditions dictated by the environmental conditions in the Northumberland Strait. While some of the pier bases were placed in relatively quiet water, others were placed in water currents upwards of two meters per second. Design criteria for the Class F concrete has been adopted largely from the Japanese standards and codes. The tremie concrete was placed in upwards of 35 meters of water, under diver surveillance.

The following requirements were applicable to the Class F concrete:

Minimum compressive strength	35 MPa @ 28 days
Slump flow	475 - 550 mm
Air content	4 - 7 %
Washout CRD test	10 % maximum
Slump (without antiwashout)	180 ± mm

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The slump flow test is the measurement of the flow in mm of the diameter of the concrete after performing the standard slump test.

Table 2 shows the concrete mixture proportions for Class F concrete in quiet water without antiwashout admixtures and for use in flowing water with antiwashout admixtures.

Air Content and Slump

Air Content - The requirements for air content, spacing factor and specific surface were as outlined in CSA A23.1-1994. The requirements were as follows:

Total air content		5 - 8 percent (plastic concrete)
Spacing factor	Average	230 μm
	Maximum	260 μm
Specific surface		$\geq 23.6 \text{ mm}^{-1}$

The total air content of 5 - 8 percent was used as a guide only and concrete acceptance was based on either a satisfactory air void system in the hardened concrete or a RDF in excess of 70 percent after 500 cycles in the ASTM C666 Procedure A test.

The structural concrete had a specified slump of $180 \pm 40 \text{ mm}$. The pumpline configuration varied widely to accommodate all the various placing scenarios. The length of pumpline and free fall in the downline had a marked influence on air loss through the pump and resulting air void spacing factor.

Table 4 shows typical results of air measurements after mixing, prior to pumping, after pumping and after vibration in the formwork in mock-up tests. The tests are not indicative of the concrete in the structure but indicate the necessity to carefully plan the placement methodology.

Mixing time also had an influence on the quality of the air void system and Fig. 4 shows the relationship between mixing time and air void spacing factor for a concrete mixture with 7.5 percent air.

Freezing and thawing tests on field samples cut from mock-up structural elements, indicated that the high performance concrete would pass the ASTM C666 test with a spacing factor in excess of 300 μm . A spacing factor up to 300 μm was accepted in areas where critical saturation was unlikely to occur. This would exclude the tidal zone or splash zone.

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Chloride Ingress

Corrosion of the reinforcement steel is recognized as the primary factor leading to deterioration of air entrained concrete. The chloride ion destroys the natural passivating layer of iron oxide that forms around the embedded reinforcement and then acts as a catalyst in the steel corrosion process. The rate at which corrosion continues is governed by the access to water and oxygen, electrical resistivity of the concrete and other geometric factors. There is no general agreement of the level of chloride ion required to be present, such that corrosion will be initiated. The stated limits generally vary from 0.4 to 0.5 percent by mass of cement. Many current codes define the upper chloride threshold level in terms of cement content and 0.4 percent of the cement mass is commonly stated.

The corrosion threshold for prestressing steel is considered to be less than that for reinforcing steel, and the critical levels are often stated as 0.1 to 0.2 percent by mass of cement.

A very low permeability concrete is assumed to be the best defense against the ingress of corrosive ions. This assumes a high quality covercrete. The relative permeability of the concrete was determined by the Rapid Chloride Permeability Test, steady state diffusivity testing, water permeability tests and chloride ponding tests. There was a good general agreement in these tests in that all tests values were very low.

In predicting the service life based on a threshold level of chloride ion in concentration at the reinforcement, Fick's Second Law of Diffusion was utilized. The time to threshold corrosion levels were predicted based on a diffusion coefficient of $4.8 \times 10^{-12} \text{ m}^2/\text{sec}$, a chloride concentration of 2100 ppm (measured in the Northumberland Strait) and a cover thickness of 75 mm. Based on these assumptions, a threshold level of chlorides will not be present for 60 years.

The predicted years to corrosion is considered to be conservative since:

- a) the Cl^- binding capacity of silica fume has not been considered
- b) the chloride diffusivity of concrete is known to improve with age - actual measurements have shown an improvement by a factor of 10.
- c) the prediction was performed for cement only concrete
- d) the high resistivity measured in the actual concrete (500 ohm-m) should extend the time to corrosion by 20 to 30 years.

Concrete will still have useful life after the threshold level of chlorides are reached. Timely maintenance will ensure continued service.

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Alkali-Aggregate Reactivity

Many of the aggregate sources in Eastern Canada are alkali-silica reactive. The low calcium fly ash used in the Confederation Bridge concrete has been used for approximately 12 years as a method to ameliorate alkali-silica reactivity. This particular source of fly ash has been researched thoroughly since the 1980's and has proven to be a reliable source.

Silica fume also ameliorates the expansions which accompanies alkali-silica reaction (in the short term at least). The quantity of the silica fume in the blend of low alkali portland cement, silica fume and fly ash (5.3 % to 6.4 %) would not be sufficient to control expansion with the use of a reactive aggregate. The important nature of this structure and the design for a long service life of 100 years dictated that a non-reactive aggregate be used as well as a low alkali cement and supplementary cementing materials.

Sulphate Attack

Normal portland cement (CSA Type 10) commonly used in marine construction in Eastern Canada contains a C_3A content of 11 to 13 percent. Historically, this cement has performed well in the relatively cold waters of the Atlantic ocean, and there is no recorded serious sulphate damage to marine structures, even though some of these structures are in excess of 70 years old. It is believed that by specifying a C_3A content of from 6 to 10 percent and with the low permeability of the concrete, that sulphate attack will not be a concern in the 100 year service life. The usage of low calcium fly ash further reduces the potential for sulphate deterioration.

Freezing and Thawing Resistance

Concrete is generally considered to be resistant to cyclic freezing and thawing in a saturated condition, if the air void spacing factors are less than about $230\ \mu\text{m}$ and the specific surface greater than $23.5\ \text{mm}^{-1}$. The spacing factor is the more important and may be the controlling factor in durability. Concrete in the Confederation Bridge which is considered to be in a highly susceptible environment for freezing and thawing damage to occur, includes the ice shields in the tidal zone, the elevation to which wave run-up occurs on the pier shafts and the bridge deck, including the barrier walls.

Flowing concrete (slump 180 mm to 240 mm) containing silica fume, a high cement factor and superplasticizer is difficult to place while maintaining proper

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air void characteristics. Air losses occur during extended mixing and transport, pumping, placing and vibration. There is a tendency for the air void spacing factor to increase from the time of mixing to the end of placement.

The pump line configuration appears to be a major factor in air losses and changes in the air void parameters. It is postulated that very small air bubbles go into solution in the concrete mixture water during pumping and further that significant air is lost through vacuum extraction in pump down lines. The height of the pump boom must be kept at a minimum to avoid undue air losses.

Table 4 shows the air losses and changes which can occur in the air content and air void system from the time of mixing to the time of placement.

Some researchers have indicated that the air void spacing factor may be increased for high strength concrete of low water cementing materials ratio. There is insufficient data related to field performance of high strength concrete in an aggressive marine environment to consider an increased air void spacing factor at this time for other than portions of the structure which is unlikely to be critically saturated.

Ice Abrasion

During the winter and early spring, ice floes move through the Northumberland Strait, and make contact with the pier ice shields. The ice shields are sloped 52° so that the moving ice will move up the slope, fail in flexure and pass around the piers. This reduces both the horizontal ice pressures on the piers and reduces surface abrasion. The zone of contact between the ice and the ice shield is approximately from elevation -1.3 m to +3.0 m.

Some of the moving ice carries embedded sand and gravel picked up during grounding of the ice on the shoreline. The embedment of granular material causes additional abrasive action on the concrete.

An air entrained concrete mixture (mixture A 100 described earlier) with a compressive strength of 80 MPa and water cementitious materials ratio of 0.25 was used in construction of the ice shields. Abrasion resistance is known to be highly dependent on compressive strength. The interior of the formwork for the ice shield concrete was covered with a controlled permeability form liner so as to provide a dense and smooth surface on the concrete.

Fig. 5 shows a comparison of 60 and 80 MPa concrete with and without the form liner as well as the abrasion resistance of normal structural concrete with a compressive strength of 35 MPa. The high quality concrete in the ice shield should provide serviceability for the design life.

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Salt Scaling

Salt scaling due to saline ocean waters has not been identified as a serious durability consideration in marine structures in Eastern Canada. The duration of wetting and drying cycles in the tidal zone does not promote crystal growth of salts.

The bridge deck is covered with an acrylic modified asphaltic water proofing membrane and asphalt paving. The barrier wall is constructed of high quality slipformed concrete of low water cementitious materials ratio. The barrier wall concrete had air void spacing factors in the order of 180 μm which is well below the CSA requirements of 230 μm .

Thermal Cracking

Cracks induced by thermal gradients are undesirable as they provide an avenue for chlorides to reach the reinforcement and also cause stress risers which may induce other forms of cracking. Thermal cracks may be in the form of clear through cracks or micro-cracks within the concrete and normally appear at the surface.

The concrete specifications for the Confederation Bridge limited the maximum permissible concrete temperature at the time of placement, maximum temperature rise within the concrete element and maximum permissible temperature gradient. The permissible temperatures and gradients are shown in Table 5.

The temperature and thermal gradients can be controlled by one or a combination of:

- using a cement with a heat generation of less than 350 kJ/kg measured at 7 days
- using a low water cementitious materials ratio
- using chipped ice as a replacement for mixture water
- using supplementary cementing materials as a partial replacement for portland cement
- cooling the coarse aggregate by washing immediately prior to mixing concrete
- adjusting the ventilation in the hoarding as required
- not using any form of elevated external heat for curing

The main components of the bridge were modeled to determine the potential for cracking to occur, and all major components were instrumented with thermocouples in strategic locations. If the temperature gradients exceeded the

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specification limits, the concrete was subjected to petrographic examination for micro-cracking.

Throughout the project, thermal cracking was not a significant problem. The few cracks that did occur either sealed autogenously or were injected with a low viscosity polymer so as to completely fill the cracks.

CONCRETE CURING

Moist curing of concrete is generally accepted as the most efficient and desirable method of curing concrete for long term durability and strength development and particularly for silica fume concrete. Some of the bridge components were in excess of 30 meters high and constructed in the winter months. Due to practical and safety reasons moist curing was not applicable. The contract documents addressed curing with several options as follows:

- water curing
- membrane curing
- in-form (steel formwork) curing for five days
- a combination of the above

Extensive testing was performed to evaluate the effectiveness of various curing methods and curing membranes. The rapid chloride permeability test, steady state chloride diffusivity tests and absorption was used to evaluate curing methods. Long term studies are still in progress on the evaluation of curing methods (an extension of the studies performed during construction), however, the following conclusions appear reasonable at this time:

- Curing compounds which meet the minimum requirements of the applicable ASTM standards (moisture loss less than 555 g/m² in 72 hours at 38 °C) will not prevent some desiccation at the concrete surface. Water based white pigmented or clear curing compound with a high solids content ($\geq 35\%$) is effective in reducing desiccation potential.
- The results of the rapid chloride permeability and chloride diffusivity tests indicate similar values for the cover concrete whether the concrete is moist cured or membrane cured.
- Rapid chloride permeability testing shows that there is insignificant difference in the long term results regardless of the duration of in-form curing.
- High performance concrete may be affected more by the temperature of curing than by the presence or absence of moisture.
- The application of a membrane is beneficial to reduce moisture loss, regardless of the duration of in-form curing.

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- Fig. 6 to 9 show the results of the rapid chloride permeability tests on mock-up samples cured in the formwork for periods of from one day to five days and at temperatures which varied from 5 °C to 50 °C for the duration of in-form curing. Fig. 10 shows the rapid chloride permeability values for concrete continuously moist cured at 23 °C until the time of testing after one day in the form.

CONCLUDING REMARKS

The construction of the Confederation Bridge commenced a new era in the transportation of peoples and goods across the Northumberland Strait. The 12.9 Km bridge replaced a ferry system which operated for in excess of 100 years.

Due to the high cost of construction and the difficulty in executing significant repairs, the Confederation Bridge was designed for a service life of 100 years. This is the first structure of this nature constructed in Canada with a design service life of 100 years. The challenge of the design and construction for the service life was based on:

- a structural design such that most of the construction could be performed as a land based operation
- the use of high performance concrete with low water cementitious materials ratio and low permeability
- the use of mock-ups to establish materials performance and provide important data on which to base the works procedures
- a Quality Management Plan based on proven techniques and ISO standards
- research and evaluation of materials for concrete production, curing, repairs, etc.
- a greater than normal concrete cover thickness as shown in Table 6
- a limitation of 0.15mm permissible crack width (wider cracks were injected with a polymeric sealer - unless they sealed autogenously)

TABLE 1—CHEMICAL AND PHYSICAL PROPERTIES OF SILICA FUME BLENDED CEMENT AND CLASS F FLY ASH

	Blended Cement Type 10SF	Low Calcium Fly Ash ASTM Class 'F'
<u>Physical Properties</u>		
Fineness		
Passing 45 μm	89.5	84
Blaine Surface Area	443 m^2/kg	351 m^2/kg
<u>Vicat Setting Time</u>		
Initial	118 min.	
<u>Compressive Strength</u>		
3 Day	23.9 MPa	
7 Day	32.1 MPa	
28 Day	48.9 MPa	
<u>Strength Activity Index</u>		
Water Requirement		88 %
Activity Index		94 % @ 28 Days
<u>Autoclave Expansion</u>	0 %	0.003 %
<u>Chemical Analysis</u>		
Silicon Dioxide (SiO_2)	27.1 %	45.2 %
Alumina Oxide (Al_2O_3)	4.7 %	20.7 %
Ferric Oxide (Fe_2O_3)	3.0 %	24.8 %
Calcium Oxide (CaO)	59.5 %	1.63 %
Magnesium Oxide (MgO)	1.1 %	0.97 %
Sulphur Trioxide (SO_3)	2.35 %	0.65 %
Sodium Oxide (Na_2O)	0.45 eq. %	0.59 %
Potassium Oxide (K_2O)		2.40 %
Loss on Ignition	1.0 %	2.66 %
Insoluble Residue	0.45 %	
<u>Bogue Potential</u>		
C_3A	7.4 %	

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TABLE 2—CONCRETE MIXTURE PROPORTIONS FOR THE VARIOUS STRUCTURAL ELEMENTS

Concrete Mixture Component	Quantity of Materials, kg/m ³			
	Class 'A'	Class 'A100'	Class 'C'	Class 'F'
			Antiwashout	No Antiwashout
Low Alkali Silica Fume Cement	430	520	307	400
Low Calcium Fly Ash	45	60	133	40
Classified Natural Sand	705	562	745	912
Stone				
20 mm - 10 mm	565	550	580	520
10 mm - 5 mm	465	550	475	345
Water	145	142	159	165
Admixtures, ml/m³				
Air entraining Agent	180	350	90	40
Water reducing Agent	1800	1600	-	3440
Set Retarder	-	580	-	280
Superplasticizer	3200	6000	1100	9200
Antiwashout Agent	-	-	-	4000

TABLE 3—PHYSICAL PROPERTIES OF THE CLASS 'A' CONCRETE

Property	Numerical Value
Slump	
Prior to Addition of Superplasticizer	40
After Addition of Superplasticizer	200
Air Content %	
Spacing Factor, μm	153
Specific Surface, mm^{-1}	25.8
Linear Coeff. Of Expansion, per $^{\circ}\text{C}$	8.3×10^{-6}
Compressive Strength, MPa	
24 hours	34.9
3 days	52.2
7 days	62.2
28 days	81.9
Modulus of Elasticity (E), GPa	
28 days	40.0
91 days	41.0
Diffusivity, $\text{m}^2/\text{sec.}$	5.8×10^{-13}
Rapid Chloride Permeability, coulombs	155 at 91 days

TABLE 4—SUMMARY OF AVERAGE AIR CONTENTS FOR VARIOUS POINTS OF SAMPLING

Concrete Mixture	Air Content, percent			Properties After Vibration
	At Plant	Before	After Pump	
Class 'C'	7.2	6.0	3.1	$3.6^1, 4.9^2, 219^3$
	5.5	5.7	4.4	$3.7^1, 4.6^2, 277^3$
	4.0	4.0	3.0	$1.9^1, 2.9^2, 422^3$
Class 'CD' (10mm)	8.5	8.6	4.5	$4.0^1, 4.9^2, 241^3$
	5.5	5.1	3.4	$2.8^1, 3.6^2, 434^3$
	4.7	4.7	4.0	$3.1^1, 3.0^2, 400^3$

Notes: ¹ denotes plastic air content in formwork

² denotes air content on the hardened concrete

³ denotes the air void spacing factor in μm in the hardened concrete

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TABLE 5—PERMISSIBLE TEMPERATURES TO CONTROL THERMAL CRACKING

Parameter	Permissible Maximum Value
Cement Heat Development	350 kJ/kg at 7 days
Placement Temperature	
Sections Greater ≥ 2 Meters	18 °C
Other Concrete, Ambient ≤ 10 °C	25 °C
Other Concrete, Ambient > 10 °C	20 °C
Temperature in Hardened Concrete	70 °C
Thermal Differential	20 °C

Note: Sources of external heat was not permitted to accelerate cement hydration

TABLE 6—REINFORCEMENT COVER THICKNESS

Location	Specification	ACI 357	CSA S6
<u>Superstructure</u>			
Top Reinforcement	50 + mm		50 mm
Barrier Wall	50 mm		50 mm
Girder Inside Face	35 mm		30 - 40 mm
Post-tensioning Ducts	100 mm	90 mm	
Splash Zone	100 mm	65 mm	
<u>Substructure</u>			
Substructure	100 mm	50 mm	
Post-tensioning Ducts	125 mm	75 mm	

Fig. 1—Location of the Confederation Bridge

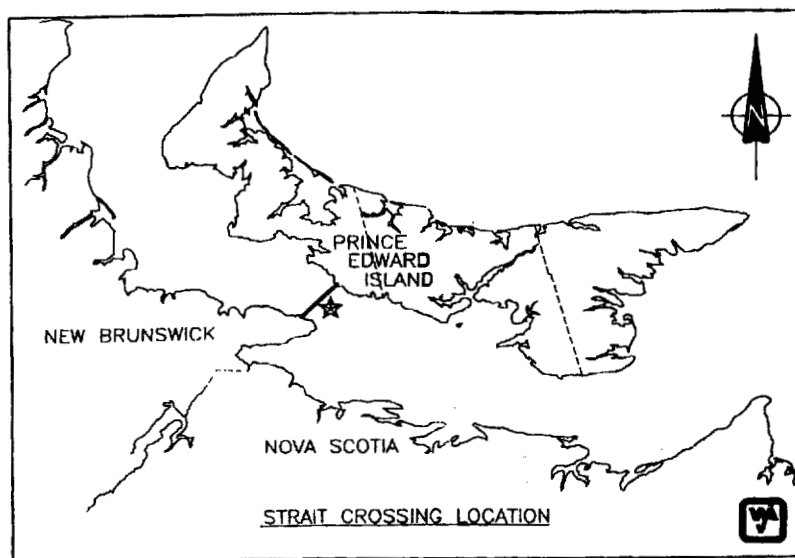


Fig. 2—Results of the accelerated mortar bar tests on sand and stone

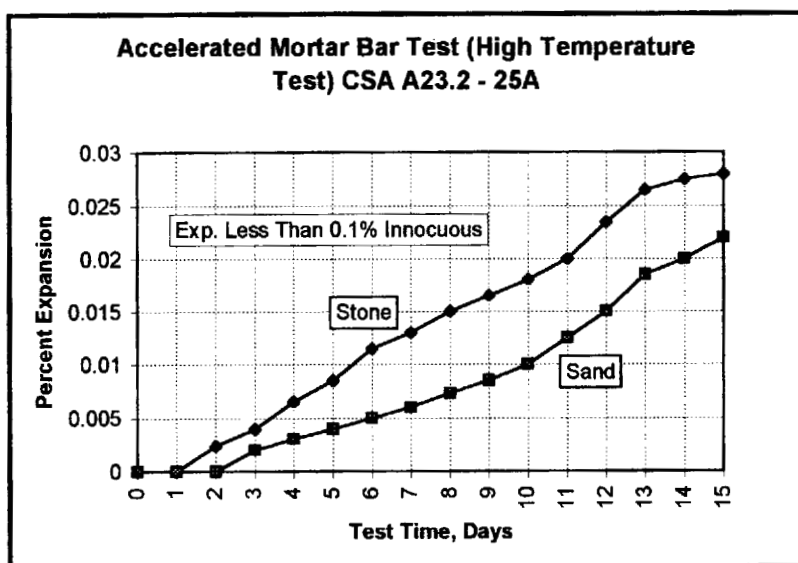


Fig. 3—Results of the concrete prism tests on sand and stone

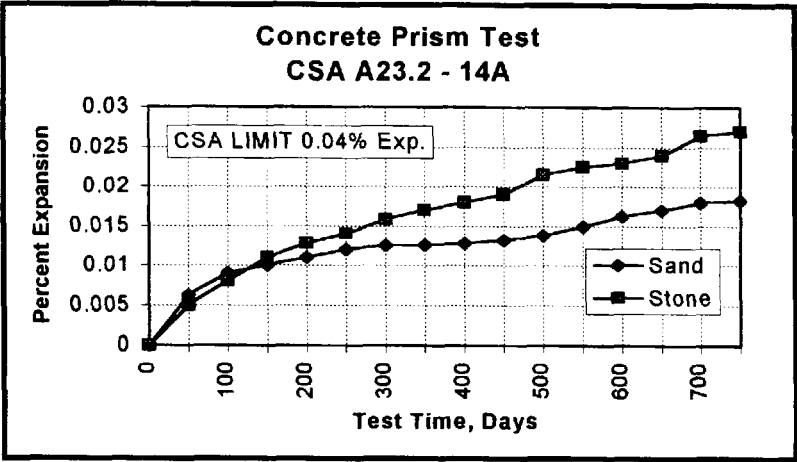


Fig. 4—Influence of concrete mixing time on the air void spacing factor

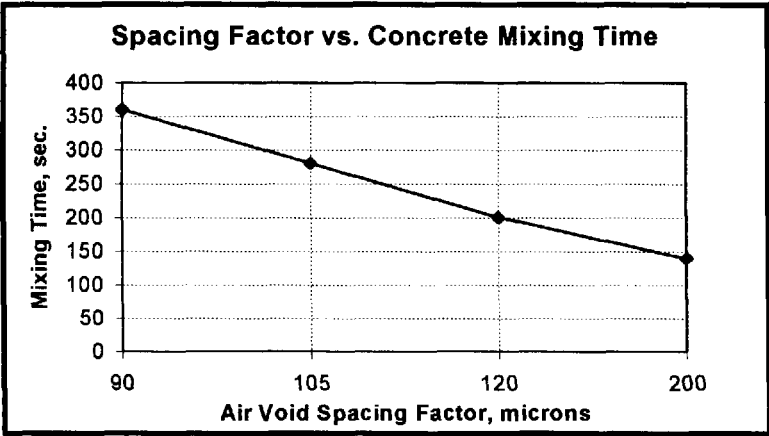


Fig. 5—Surface abrasion test results with and without a form liner

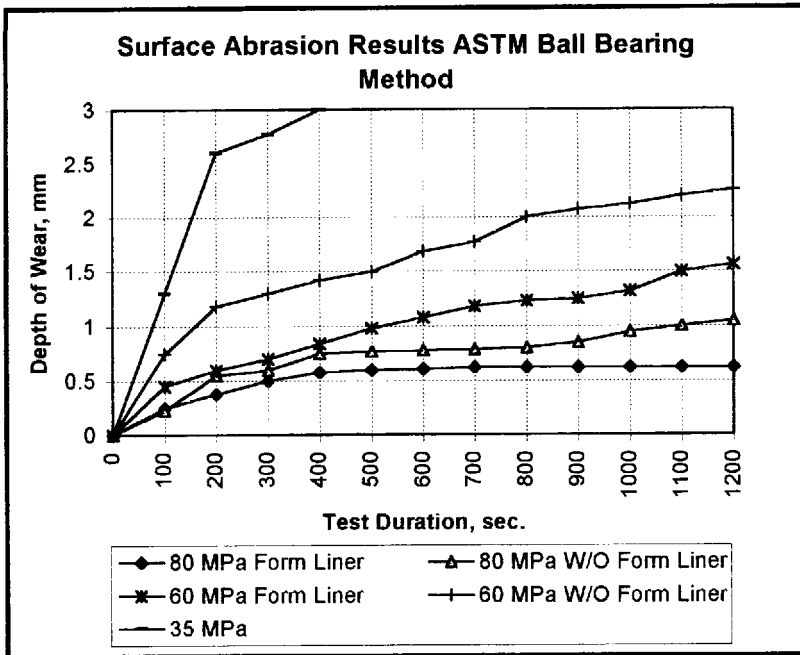


Fig. 6—RCP of concrete cured in form 1 day, subsequently air dried

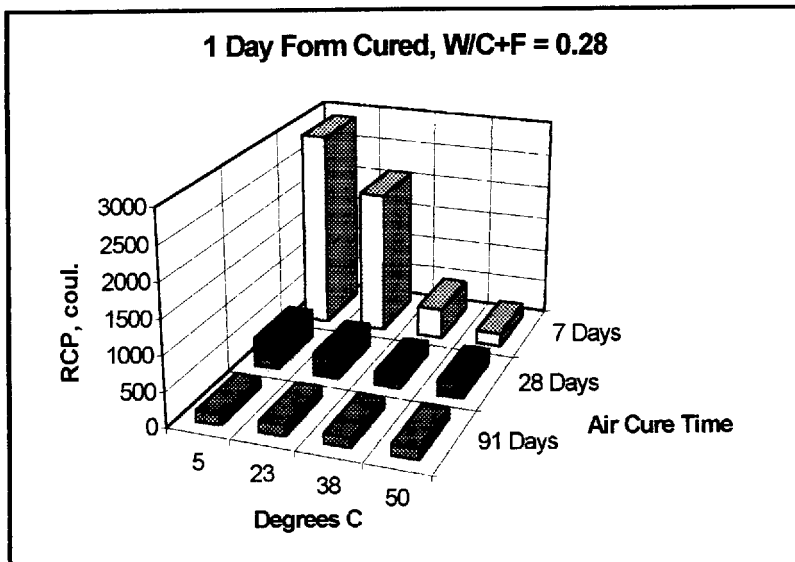


Fig. 7—RCP of concrete cured in form 2 days, subsequently air dried

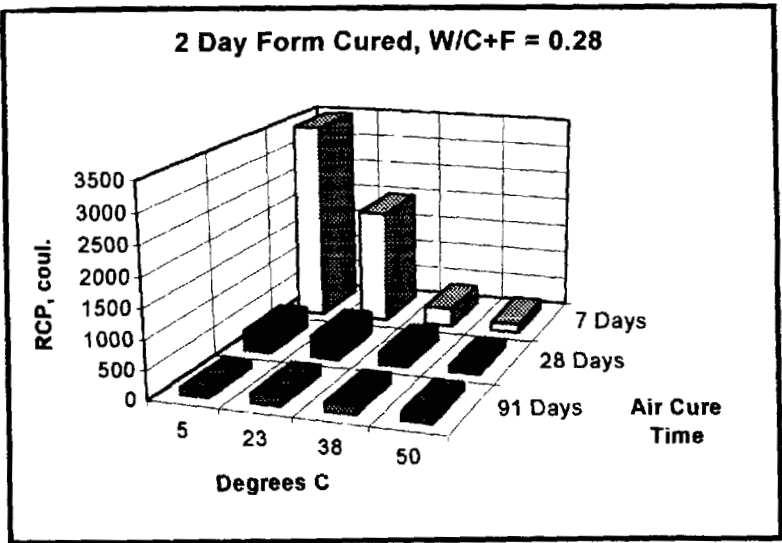


Fig. 8—RCP for concrete cured in form 3 days, subsequently air dried

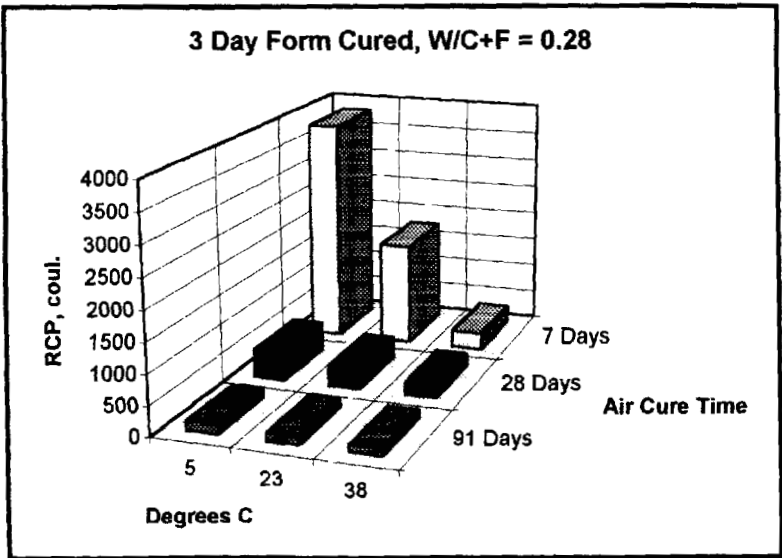


Fig. 9—RCP for concrete cured in for 5 days, subsequently air dried

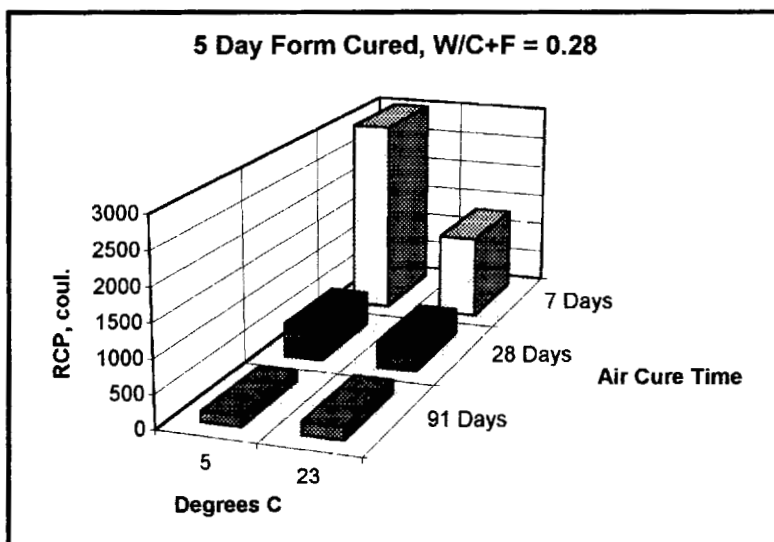
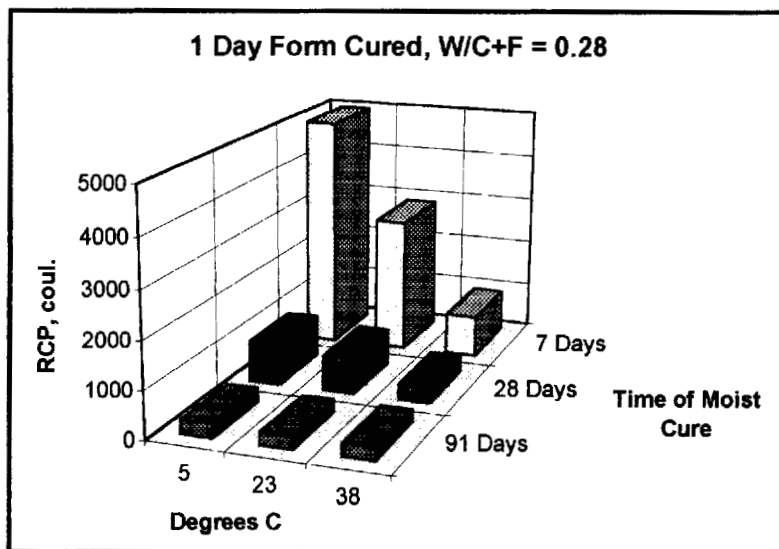


Fig. 10—RCP for concrete cured in form 1 day, subsequently moist cured



SP 179-2

High Volume Wastes Concrete (HVWC)

by H. Uchikawa, S. Hanehara and H.
Hirao

Synopsis: High volume waste concrete (HVWC) containing hundreds of kg of waste-derived materials in unit volume of concrete as raw materials was prepared to examine the workability, strength development, hydration of cement, composition and structure, and dissolution of harmful elements from hardened concrete to increase the amounts of waste-derived material that can be used for manufacturing concrete. The waste-derived materials tested in this experiment were incineration ash of urban refuse and sintered coal ash as the substitution for fine aggregates, and sintered sewage sludge and glass cullet as the substitution for coarse aggregates.

It was determined that HVWC could keep good workability without segregation and developed higher strength than ordinary concrete even if the amounts of waste-derived aggregates in concrete exceeded 600kg/m^3 .

Increase in combined water in hardened HVWC and the production of cement hydrates including C-S-H with age was normal and the influence of trace elements contained in waste-derived aggregates on the cement hydration was negligible.

Decrease in the amounts of Ca(OH)_2 and increase in C-S-H which was estimated from the pore volume of 3 to 6 nm in diameter was recognized in later age in the case when blastfurnace slag or fly ash was used as a binder.

Non-uniformity in distribution of aggregate, large pore and microgroove between aggregate and cement paste which might occur by the use of large amounts of waste-derived aggregate was not observed.

There was no remarkable difference in type and quantity of elements between HVWC and ordinary concrete dissolved from them.

From the results described above, it is considered that the use of HVWC is a very promising technique to safely consume large amounts of wastes.

Keywords: compressive strength; concrete; hydration; pore size distribution

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INTRODUCTION

Although the application of large quantities of wastes in the concretes industry is the needs of the times (1), the wastes used as the raw materials for cement and concrete are at present as little as 10 to 100 kg per ton of finished cement and 10 to 200 kg per cubic meter of concrete, respectively. As for the substitution of waste-derived materials for aggregate, some papers have reported that the fluidity and strength development of concrete are reduced with the increase of the fly ash partially substituted for the aggregate (2)(3).

This paper outlines the development of high volume wastes concrete (HVWC) containing as much as hundreds of kilograms or more of wastes per cubic meter of concrete by using a combination of several types of waste-derived materials and properly adjusting the particle size distribution aiming at the usage of a large quantity of the waste-derived materials not only as the blending component for cement but also the substitute for the aggregate for concrete.

SAMPLE AND EXPERIMENTAL METHOD

Materials Used for Experiment and Mixture Proportion of Concrete

Incineration ash of urban refuse, sintered coal ash, sintered sewage sludge

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and glass cullet as well as blastfurnace slag and fly ash which have been widely used were used as the waste-derived materials. The chemical compositions, specific gravities, fineness modulus (FM), specific surface areas and crushing value of the materials used for the experiment are listed in Table 1. Their particle size distributions are illustrated in Fig.1. As listed in Table 2, the mixture proportion of HVWC was fixed as follows: unit amount of water is 185 kg/m³ of the upper limit specified in Japanese Architectural Standard Specification (JASS) No.5, and the unit amount of binders including portland cement, fly ash and blastfurnace slag are 200, 100 and 200 kg/m³, respectively. The water • binder ratio was fixed at 37%. The waste-derived materials except blastfurnace slag and fly ash were used as the substitute for the aggregate. Two types of HVWC including HVWC(I) prepared by using waste-derived material (1519 kg/m³) as all the fine and coarse aggregates and HVWC(II) prepared by using that (631 kg/m³) as only fine aggregate were used for investigation. Reference concrete with a mixture using natural aggregate was also prepared. Since the target air content was 4.5%, a polycarbonic acid-based AE water reducing admixture and AE intensifier were used. The AE water reducing admixture was added so as to nearly equalize the slump and slump flow values of HVWC(I) and (II) and reference concrete to each other. For easily understanding the differences of HVWC(I) and (II) from ordinary concrete and the above-mentioned reference concrete, ordinary concrete was also prepared at the W/C of 0.6 using 300 kg of cement/m³, Hamaoka sand and Iwase gravel for comparison as listed in Table 2. The size of the test cylinders were 10 cm in diameter and 20 cm high. They were removed from the mold 24 hours after placing and cured at 20°C in water till the specified age.

Experimental Method

The slump and strength of concrete were determined by the test methods specified in JIS A 1101 and A 1108, respectively. The samples used for determining the composition of hardened concrete and investigating its structure were prepared by cutting out pieces of approximately 5mm square from the hardened concrete with a diamond cutter, terminating the hydration reaction with acetone and D-drying. The qualitative and quantitative analyses of the hydrate were made by differential scanning calorimeter (DSC). The structure of hardened concrete was observed by scanning electron microscope (SEM) and electron probe microanalyser (EPMA). The pore size distribution was determined with a mercury intrusion porosimeter (4). The quantities of elements dissolved from the waste-derived materials and hardened HVWC were determined by the method specified in Notification No.13 of Environment Agency using inductively coupled plasma analysis (ICP). The strength of aggregate used for the experiment was measured according to BS 812 in such a way that 50g of the aggregate with the particle diameter of 0.6 to 2.5 mm was sieved through a screen with an opening of 0.3 mm

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before and after applying a load of 50N/mm^2 to it, and then the ratio of the weight passing through the sieve -crushing value- was obtained.

RESULTS AND DISCUSSION

Properties of Fresh HVWC

The slumps and air contents of HVWC are listed in Table 2. The fluidity of HVWC(II) using the waste-derived material as only fine aggregate was better than that of HVWC(I) when using it as both fine and coarse aggregates. The specified fluidity can be, therefore, attained using a smaller quantity of the AE water-reducing admixture in HVWC(II). Since the incineration ash of urban refuse and the sintered coal ash contain less fine particles than the natural fine aggregate, HVWC were short of particles with the intermediate diameter between cement and aggregate, whereupon the materials were apt to be separated though the slump was easily developed. Even though the waste-derived materials were used, the fluidity was not so much reduced and the materials were not extremely separated in concrete as well as premature stiffening which may have occurred due to the dissolution of the unfavorable minor and trace elements were not observed.

Compressive Strength of HVWC

Fig.2 reveals that the compressive strengths of any types of concrete at the age of 3 days range from 10.6 to 12.2N/mm^2 except 17.4N/mm^2 of the reference concrete. Although the difference between the compressive strengths by mixture proportion and raw materials was small, as mentioned above, it began to increase at the age of 7 days and the strengths of HVWC(II) and (I), reference concrete and ordinary concrete at the age of 91 days were as large as 41.2 , 26.0 , 59.8 and 33.2N/mm^2 , respectively. The compressive strength of HVWC(I) hardly increased at and after the age of 7 days, hence the difference from that of HVWC(II) which increased with the advance of the age. The strength of HVWC(II) remarkably increased even at and after the age of 7 days, exceeding by 8.0N/mm^2 attained by ordinary concrete at the age of 91 days. The strength of the reference concrete was higher than those of the others from the early age, exceeding that of ordinary concrete by 26.6N/mm^2 at the age of 91 days. The shape of glass cullet used as the substitute for coarse aggregate in HVWC(I) was flatter and its surface was smoother than the natural crushed stone from Iwase and the crushing value (41.5%) of it is inferior to that (28.7%) of Iwase crushed stone as shown in Table 1. It is, therefore, considered that the strength-developing properties of hardened

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concrete is hindered by a synergistic effect of the increased load stress, low bond strength to cement paste and its low strength. The crushing values of the incineration ash of urban refuse and sintered coal ash substitutionally used as fine aggregate to HVWC ranging from 40.5 to 64.0% was lower than that (23.7%) of natural land sand from Hamaoka. From these results, it is considered that the strength of the paste of reference concrete was higher than that of the other concretes from early age. Large difference between the compressive strengths of HVWC(I) and (II), and ordinary concrete after the age of 7 days were attributed to the difference of strength of aggregates used because the cement paste part developed higher strength than that of aggregates.

Composition and Structure of Hardened HVWC

Contents of $\text{Ca}(\text{OH})_2$ and combined water--The contents of $\text{Ca}(\text{OH})_2$ and combined water per mass of binder in hardened concrete at the age of 91 days are illustrated in Fig.3. The contents of $\text{Ca}(\text{OH})_2$ in HVWC(I) and (II) and the reference concrete were 5.5, 5.7 and 5.9%, respectively, which was relatively equal, while that in ordinary concrete for reference was as little higher as 14.0%. The contents of combined water in HVWC(I) and (II), the reference concrete and ordinary concrete were 22.4, 23.1, 23.8 and 31.5%, respectively. The higher contents of $\text{Ca}(\text{OH})_2$ and combined water of ordinary concrete than that of the other concretes may have been caused by higher W/C. Combined water and $\text{Ca}(\text{OH})_2$ content in HVWC(I) and (II), and reference concrete were relatively the same. Considering these results together with C-S-H produced as estimated from the pore volume of 3-6 nm in diameter described later, influence of the trace elements dissolved from the waste-derived aggregates on the hydration of cement is considered to be negligible.

Structure of hardened concrete--Optical micrographs of hardened HVWC(I), back scattered electron images and secondary electron images of polished cross section of hardened concrete at the age of 91 days are illustrated in Fig.4, 5, respectively. Fig.4 reveals that each raw material was macroscopically evenly distributed in the structure and that coarse pores and spaces between the wastes and cement paste were not observed. Fig.5 illustrating the back scattered electron images reveals that the thickness of the layer of C-S-H formed around the unhydrated cement particles in HVWC was almost the same as those of the reference concrete and ordinary concrete. It is, therefore, inferred from this that the hydration reaction of cement is hardly affected by the minor and trace elements which may be dissolved from the waste-derived materials. The transition zone made of crystals of $\text{Ca}(\text{OH})_2$ deposited around the aggregate in ordinary concrete was observed, while the transition zone was hardly produced in HVWC(I) and (II) and the reference concrete because the W/C is low and the quantity of

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Ca(OH)_2 deposited is small.

Fig.5 illustrating the secondary electron images reveals that the hardened structure of the reference concrete and HVWC were almost the same though the morphology of C-S-H in the reference concrete was slightly clearer than that in HVWC. The reference concrete and HVWC have dense structures made of fine crystals 100 nm in length or under of C-S-H on the whole, while ordinary concrete has a coarse structure made of C-S-H about 300 nm in length. This could be due to the W/C of ordinary concrete being higher than those of HVWC and the reference concrete. That figure also reveals that the particle size and morphology of C-S-H in HVWC using the waste-derived materials were equal to those of the reference concrete using natural aggregate and the structure of the cement paste part was dense.

Pore structure--The pore size distribution in hardened concrete at the age of 91 days is illustrated in Fig.6. The total pore volume in HVWC(I) and (II), the reference concrete and ordinary concrete are 15.0, 12.8, 10.3 and 10.5%, respectively. The volume of coarse pore $1\mu\text{m}$ or larger in HVWC(I) and (II) were as large as 4.5 and 2.4%, respectively. It is considered that the coarse pores are pores originally existing in the waste-derived materials used as the aggregate remaining unfilled with cement hydrate. The pore volume of 50 nm to $1\mu\text{m}$ in diameter composing the transition zone(5) in ordinary concrete was as large as 5.1%, while those in HVWC(I) and (II) and the reference concrete were 1.11, 0.97 and 0.68%, respectively. The volume of pore 3 to 6nm in diameter corresponding to the distance between the layers of C-S-H in HVWC(I) and (II) and the reference concrete are 4.8, 4.8 and 5.0%, respectively, while that in ordinary concrete is 2.1%. This suggests that HVWC and the reference concrete have large quantities of C-S-H by proceeding the pozzolanic reactions with blastfurnace slag and fly ash contained in concrete, hence the structures of paste at the advanced age are densified.

The above-mentioned results indicate that there are no remarkable differences in the composition and structure of the cement paste between HVWC using the waste-derived materials and those in the reference concrete. It is estimated that the compressive strength of concrete prepared using the low-strength waste-derived aggregate is lower than that of the reference concrete prepared using natural aggregate, but the required durability is secured because the structure of the hardened concrete is dense. This will be judged by the long-term test results of concrete.

Dissolution of Elements from Hardened Concrete

The quantities of elements dissolved from HVWC(I) and ordinary concrete

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are listed in Table 3 together with the environmental criteria and emission criteria of water for each element. There was little difference in the types and quantities of the elements between HVWC(I) and ordinary concrete. The quantities of elements dissolved from the hardened concrete met the emission criteria of water. Harmful elements and heavy metals are fixed to hardened concrete as insoluble hydroxides or by adsorption or as solid solution(6)(7) . The harmful elements were hardly dissolved from HVWC using a large quantities of waste-derived materials as the aggregate. Maybe this is because not only those elements are hardly dissolved from the waste-derived aggregate but also they are fixed to hardened concrete through the routes mentioned above.

SUMMARY AND CONCLUSIONS

- 1) High volume waste concrete (HVWC) was prepared on an experimental basis substituting waste-derived materials as much as hundreds of kilograms per cubic meter of concrete for the aggregate by adjusting the particle size distribution by means of combining several types of wastes. And the fluidity, strength development, hydration reaction, structure of hardened concrete, dissolution of elements from hardened concrete and others of HVWC were investigated. The results indicate that some types of wastes are possible to be safely used as aggregate for concrete in large quantities.
- 2) Even though in the case when more than 600 kg of the waste-derived material per cubic meter of concrete are used as the aggregate by combining several types of waste-derived materials with different particle sizes, HVWC can secure the fluidity of the fresh concrete without segregating raw materials. Although the compressive strength of HVWC is lower than that of the concrete using the natural aggregate, because the waste-derived aggregate has low strength, flat shaped and smooth surface, 40N/mm² or higher compressive strength of HVWC can be obtained at the age of 91 days.
- 3) The hydration reaction of cement proceeds normally in HVWC without being affected by the dissolution of minor and trace elements contained in the waste-derived material. The production of C-S-H can be increased and the deposition of Ca(OH)₂ can be reduced by using slag and fly ash. It is, therefore, estimated that the hardened structure of HVWC does not so much differ from that and the durability of it is not inferior to that of the concrete using the natural aggregate.
- 4) The type and quantitie of harmful elements dissolved from hardened HVWC are almost the same as those of ordinary concrete and they meets the emission criteria of water.

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TABLE 1—CHEMICAL COMPOSITION, SPECIFIC GRAVITY, FINENESS MODULUS, SPECIFIC SURFACE AREA AND CRUSHING VALUE OF MATERIALS USED

		Chemical composition (%)										Specific gravity	Fineness modulus	Blaine* (cm ² /g)	Crushing value (%)
		SiO ₂	Al ₂ O ₃	Fe ₂ O ₃	CaO	MgO	SO ₃	Na ₂ O	K ₂ O	P ₂ O ₅	Total				
Binder	Normal portland cement	21.5	5.4	3.2	64.5	0.6	2.0	0.3	0.5	—	98.0	3.16	—	3,200	—
	Fly ash (FA)	62.6	18.6	8.5	1.8	1.3	0.9	0.3	1.1	—	95.1	2.26	—	4,150	—
	Blast furnace slag (SL)	32.8	15.0	0.3	42.6	5.4	2.2	0.1	0.2	—	98.6	2.91	—	4,110	—
Aggregate	Fine	79.1	15.0	3.7	1.9	0.1	0.0	0.0	0.0	—	99.8	2.59	2.7	—	23.7
	Coarse	75.3	11.4	2.9	2.1	1.2	0.0	2.4	3.3	—	98.6	2.62	6.8	—	28.7
Aggregate from waste materials	Incineration ash (IA)	50.0	15.6	4.1	16.2	2.3	2.0	7.47	1.4	0.52	99.6	2.61	3.6	—	40.5
	Sintered coal ash (CA)	52.5	24.0	5.1	7.0	1.8	0.2	1.0	1.0	—	91.6	2.05	4.0	—	64.0
	Sintered sewage (SS) sludge	32.1	14.1	4.5	34.4	2.4	2.0	0.8	0.51	8.0	98.8	2.92	6.0	—	41.7
	Coarse Glass cullet (GC)	69.3	5.7	0.5	9.6	0.6	0.0	11.7	1.1	0.0	98.5	2.50	6.3	—	41.5

*: Blaine specific surface area

TABLE 2—MIXTURE PROPORTIONS OF CONCRETE

	Cr ⁶⁺	Cu	Zn	As	Cd	Pb
HVWC(I)	0.05	0.12	ND	ND	0.001	0.02
Ordinary concrete	0.02	0.01	ND	ND	0.001	0.008
Emission criteria of water	0.5	3	5	0.1	0.1	0.1
Environmental criteria of water	0.05			0.01	0.01	0.01

TABLE 3—ELEMENTS DISSOLVED FROM HARDENED CONCRETE AND EMISSION AND ENVIRONMENTAL CRITERIA OF WATER (PPM)

	W/B (%)	s/a (%)	Unit amount (kg/m ³)										Admixture (to binder, %)	Unit amount of wastes (kg/m ³)	Slump (cm)	Flow (cm)	Air (%)	
			Binder			Fine aggregate			Coarse aggregate									
			Cement	FA	SL	Sand	LA	CA	Gravel	SS	GC	*1 FM						*2 FM
HVWC(I)	37.0	43.0	200	100	200	—	500	131	3.7	—	296	592	6.2	0.8	1.6	24.0	65	6.5
HVWC(II)	37.0	43.0	200	100	200	—	500	131	3.7	897	—	—	6.8	0.3	0.8	24.0	58	3.7
Reference concrete	37.0	43.0	200	100	200	661	—	—	2.7	897	—	—	6.8	0.4	0.4	23.5	—	1.5
Ordinary concrete	60.0	41.5	300	—	—	738	—	—	2.7	1059	—	—	6.8	0.1	—	24.5	—	4.4

*1: Fineness modulus of aggregate, *2: AE water reducing agent, *3: AE intensifier

Fig. 1—Particle size distribution of aggregate from waste materials

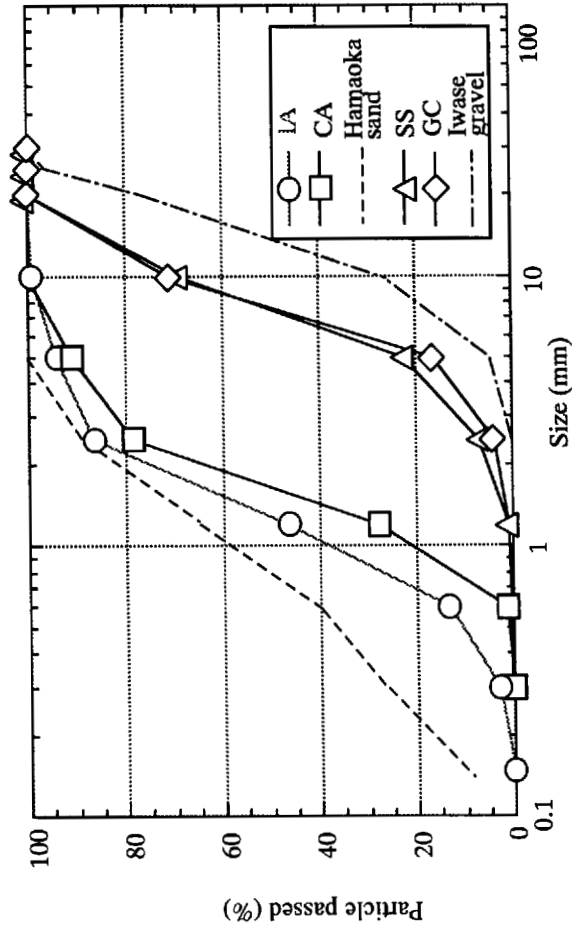
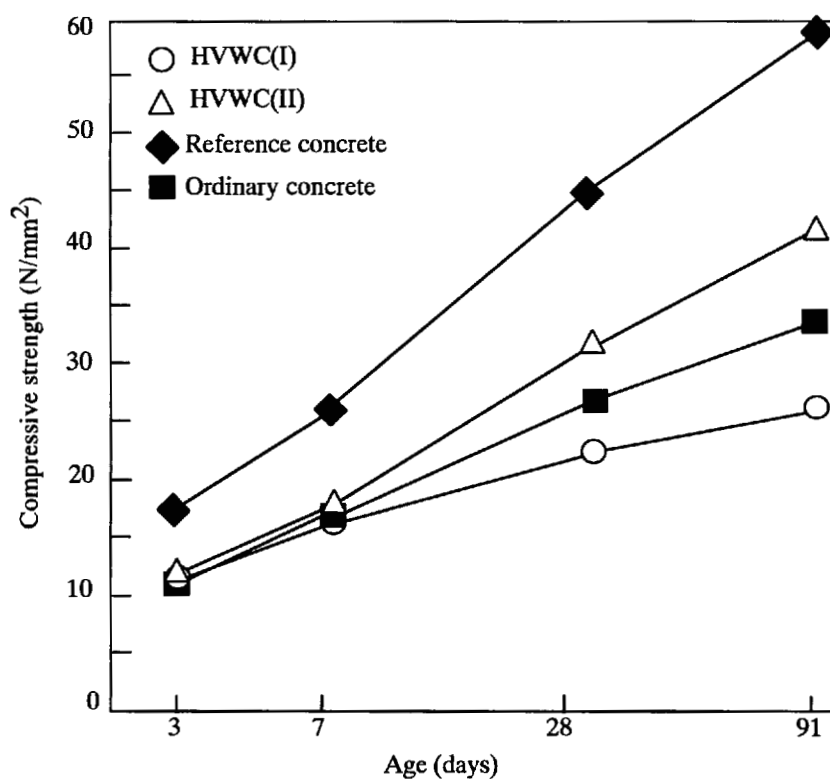


Fig. 2—Compressive strength of hardened concrete



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Fig. 3—Amount of combined water and Ca(OH)_2 per mass of binder in hardened concrete (20°C, 91 days)

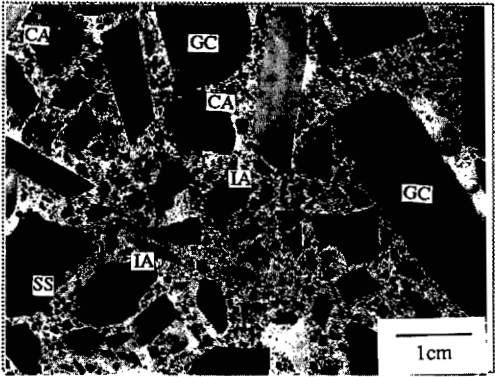
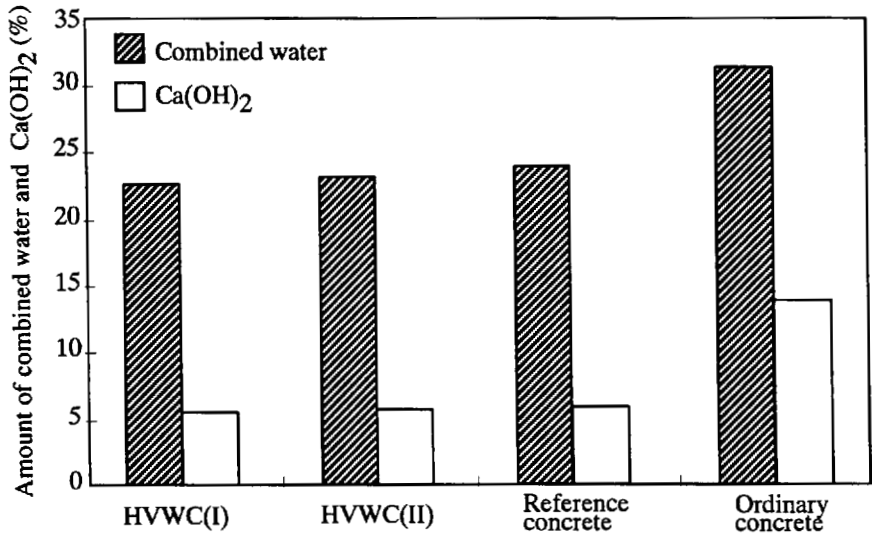
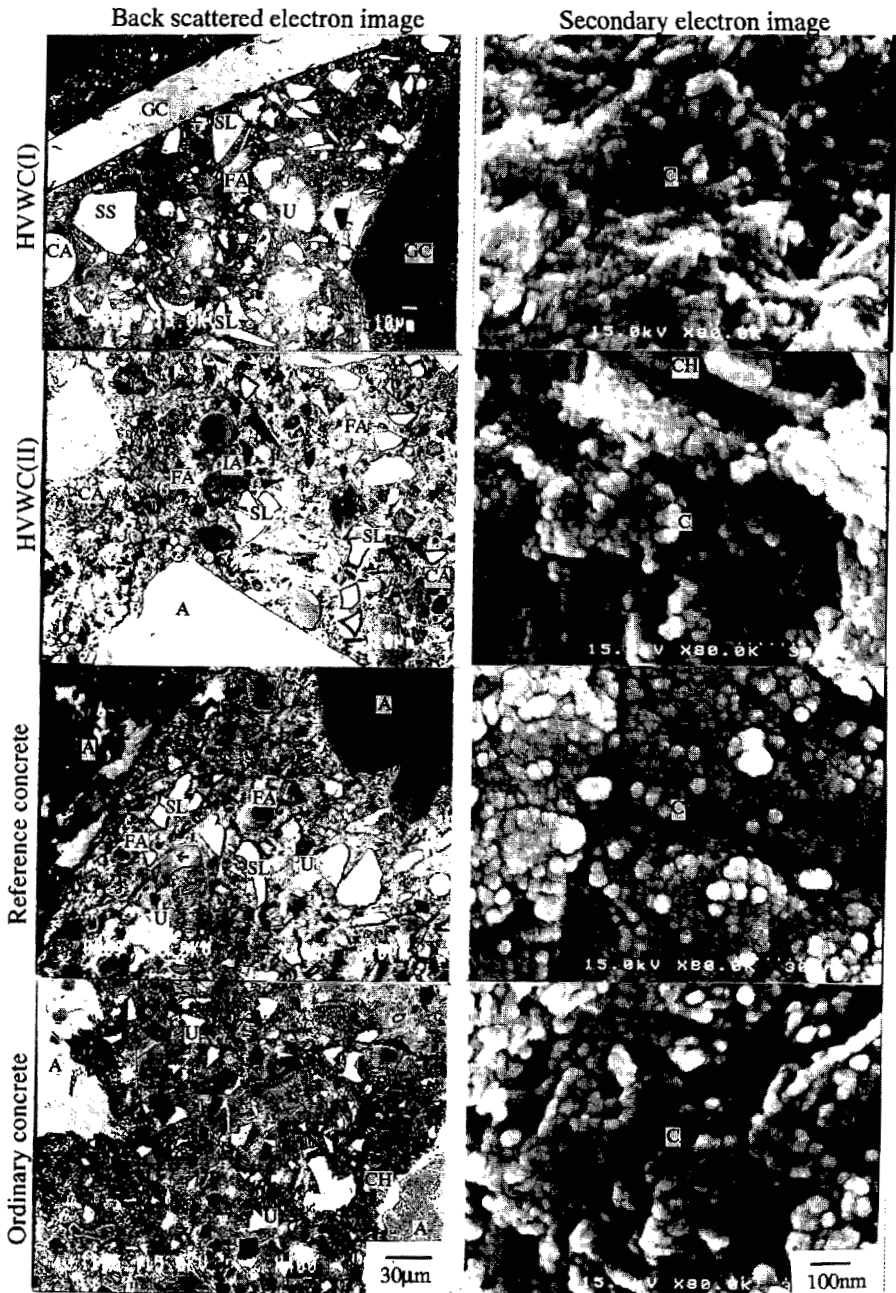


Fig. 4—Pore size distribution of hardened concrete (20°C, 91 days)

Fig. 5—Optical micrograph of HVWC(I) (20°C, 91 days)



A: Aggregate, CH: Calcium hydrate, C: C-S-H

Fig. 6—Back scattered electron image and secondary electron image of hardened concrete (20 °C, 91 days)

