HIGH PERFORMANCE CONCRETE

FOR

HIGHWAY BRIDGES IN NOVA SCOTIA

by
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LIST OF SYMBOLS AND ABBREVIATIONS

AASHTO	American Association of State Highways and Transportation Officials
CPCA	Canadian Portland Cement Associon
CSA	Canadian Standards Association
d	discount rate
e	escallation rate
Ec	modulus of elasticity (MPa)
f'c	concrete compressive strength (MPa)
HPC	high-performance concrete
n	number of years
NSC	normal-strength concrete
NSDoT	Nova Scotia Department of Transportation and Public Works
PC	present cost ($)
PV	present value ($)
SHRP	Strategic Highway Research Program of the United States Federal Highway Administration
SS	specific surface of air voids

ACKNOWLEDGEMENTS

The opportunity to be involved in a significant engineering project as part of a Masters program is gratefully acknowledged. Dalhousie University DalTech (formerly the Technical University of Nova Scotia) was invited to be a part of the development of new materials for highway bridge construction in Nova Scotia because of the Faculty's reputation for the practical application of research in concrete technology. The University's involvement is largely the result of my supervisor Jean-Francois Trottier's recognition by the local construction industry. The thesis enabled me to assist in examining various engineering issues related to the development of new technology, including materials science, mechanics, economics, construction methodology and project coordination. Again, Dr. Trottier is to be acknowedged for making this possible, and for guiding and supervising my efforts with insight, humour and patience.

The project's implementation team included project direction and coordination through the Nova Scotia Department of Transportation and Public Works' (NSDoT's) Ray Snair and Gary Pyke, industry input coordinated by the Canadian Portland Cement Association's Bill Dooley, materials design and testing by DalTech's Jean-Francois Trottier and Jacques Whitford's Wib Langley and Gordon Leaman, construction engineering by NSDoT's Brian Russell, and design by NSDoT's John Salah, all assisted by Denis Mitchell's and John Bickley's expertise through Concrete Canada. NSDoT's Peter Adams provided the valuable maintenance data for this project. The bridge was built by Alva Construction with prestressed girders by Strescon and ready-mix concrete from Casey Concrete. Materials testing during construction was by Jacques Whitford and Associates.

ABSTRACT

A highway overpass structure was built in Nova Scotia utilizing high-performance concrete (HPC) to improve the durability of the deck and other components and to reduce the cost of prestressed girders. The bridge was put into service in December, 1997. An economic assessment, carried out as part of this project, indicated that the maintenance costs of conventional structures are currently in the order of 20% of initial construction costs. The use of HPC is expected to reduce maintenance costs sharply. Construction costs are also considered to be less than for a conventional design. The overall life-cycle cost of this structure may be reduced by as much as 20-25% through the use of HPC.

The introduction of a new technology involved careful planning by a project team to develop interest and support from all involved groups, and to develop a strong understanding of the relationships between quality, price and risk. Various design options were studied from which the conventional design and two alternative HPC concepts were presented for discussion. The final selection was then made with a view to balancing the advance in technology with the success and economy of the project. These design options and related construction and maintenance costs are described, together with the process of structural design, materials selection, tendering and construction. The testing program to develop the concrete mixture design is described in detail.

1. INTRODUCTION

What skills can engineers offer in responding to community needs for transportation? Are significant advances in quality and economy still possible, or is the technology substantially mature? How do we compare alternative systems and design options?

The reduction in government structure and various changes in the way public services are delivered in the 1990's provides engineers with an opportunity to reassess how to lead in tackling broad issues in which technology contributes to the solutions. Or engineers may just provide technical support as required to assist other groups in the development of systems, and risk failing to gain the recognition for innovation that is needed to promote research and development.

Highway bridges are a significant component of Nova Scotia's infrastructure. The majority of these bridges are small or medium spans and a common perception is that the related technology is mature and does not offer much opportunity for major innovation. However, research into concrete materials over the past two decades has led to a significant advance in design and construction methods and the potential to reduce the capital and life-cycle costs of bridges, while maintaining or improving performance to the user.

Recent advances through concrete research include

- a greater understanding of the microstructure, chemical composition and mechanical behaviour of cement pastes and concrete,
- the rheology, setting and curing processes of concrete,

- the influence of silica fume, fly ash and other pozzolans, and
- the development of admixtures which change flow characteristics, setting times, air entrainment, adhesion and permeability.

The effect of these advances, particularly in the use of pozzolans, is to enable hydraulic concretes to be produced reliably which are an order of magnitude greater in strength, or durability, or workability, than was the case twenty years ago. Research and development in the areas of materials and structural mechanics are to the point that design and construction with high performance concretes are being codified for general use by the engineering community. Various high-rise buildings, bridges and other structures have been built ahead of these codes.

A number of short-span bridges using high-performance concrete (HPC) have been built in North America since 1994 [eg. Fed. Highway Admin., 1997]: the improved mechanical properties are used to reduce the quantity of prestressed concrete and to reduce permeability, thereby improving durability.

A high performance concrete bridge project was initiated in Nova Scotia in 1996 with a view to reducing maintenance costs and addressing long-term quality issues.

This thesis describes the results of research and development of HPC materials and its use in the construction industry to date. The materials design, economics and project development of a prototype bridge are described in detail.

2. PROTOTYPE H.P.C. BRIDGE PROJECT DEVELOPMENT

A general summary of the project is included in [Fletcher, 1997].

In 1996 the Nova Scotia Department of Transportation and Public Works (NSDoT) initiated a study into the potential application of HPC in highway structures. It was recognized early that introducing a new technology involves careful planning to develop the interest and support from all involved groups, and to develop a strong understanding by all the stakeholders of the relationships between quality, financial price and risk.

A multi-disciplined approach was necessary for the project's development. A steering committee was formed to guide the project through bridge selection, conceptual design, final design and construction specification. Initially, members included:

- Ray Snair, now retired, who initiated the project within NSDoT,
- Bill Dooley, the Canadian Portlaned Cement Association's (CPCA's) regional director for Atlantic Canada, who chaired the committee,
- Gary Pyke, NSDoT's concrete materials specialist,
- John Salah, design engineer with NSDoT,
- Jean-Francois Trottier of DalTech, who led the materials design,
- Wib Langley, consultant in concrete materials design,
- and myself.

The bridge's construction was the result of strong collaboration on structural and materials research and design, maintenance engineering, economics, construction issues and contract management.

Meetings were held in the summer and fall of 1996 in which the literature on completed projects and current technology was reviewed. Potential objectives for the study were identified and the construction of a prototype bridge was proposed to evaluate various aspects of the technology, including:

- durability of concrete structural elements and wearing surfaces under local conditions of salting and freeze/thaw cycles,
- construction methods, structural and materials design, and
- price and quality assurance issues.

The prototype bridge was selected from various bridges that were scheduled to be built in 1997. The West River East Side Bridge was chosen because it has more than one span, the spans are reasonably long for precast concrete construction, it is reasonably close to the Halifax area and it is in a part of the Province in which the aggregates used by ready-mix concrete suppliers is predictable and suitable.

The committee was expanded to include Brian Russell, construction engineer with NSDoT in November, 1996. A meeting was held on Nov. 28, 1996 at which a presentation was made by Denis Mitchell on behalf of Concrete Canada on structural design and materials developments. Contributions were also made through phone discussion and informal meetings by Jim Francis, a bridge engineer with NSDoT, Peter

Adams, NSDoT's bridge maintenance manager, and Gordon Leaman of Jacques Whitford, involved in materials design.

At that point in time it was clear there was a potential to reduce the cost of the precast bridge girders by using high-strength concrete, either by reducing the size of the girders or by eliminating one of the four girders in each span. The concrete strength required to have only three girders, 60-70 MPa, was also considered to be suitable for improving durability.

Two alternatives were considered in the design of the deck for the HPC bridge:

a] use traditional components and compare the durability of the HPC deck to a 'normal strength concrete' (NSC) deck, or

b] maximize the exposure of the HPC deck through elimination of the waterproofing membrane and asphalt wearing surface.

In both cases the design objective was that HPC elements are not to require repair during the structure's design life.

Conceptual structural designs and proposed concrete design parameters were developed. Capital costs were estimated for the conventional design and the two HPC options: actual costs of recent conventional designs were reviewed and the relative complexity and learning required to use HPC were considered. Maintenance costs were projected for both options from which the life cycle cost estimates were prepared in the form of net present worth. Finally, the design was selected with a view to balancing the advance in technology with a need for demonstrable success of the project, in terms of both quality and economy. The exposed concrete deck (Option [b] above) was recommended because of its economic potential and because the exposed deck can be more easily assessed for deterioration. Also, micro-surfacing is possible in the event of deterioration of the exposed deck.

The project was authorized on February 17th, 1997 following a presentation to NSDoT of the expected reductions in both initial and life-cycle costs and the limited risk of any compromise in quality. A concrete materials testing and design program was authorized shortly thereafter. Design parameters for the concrete mixture were confirmed on Feb. 25th, 1997. The project schedules (proposed and actual) are included in Appendix A. The structural design and specification of the bridge was completed in April, 1997, as was the preliminary design of the concrete mixture.

A single mixture design was developed for both the precast bridge girders and the cast-in-place concrete elements. Three phases of testing were proposed to develop the concrete mixture design:

1) preliminary tests (compressive strength, air void and rapid chloride permeability tests and evaluation of plastic properties) with six mixture designs in which the ratio of water-to-cementitious materials were varied,
2) advanced testing on a selected mixture, including salt scaling, flexural fatigue endurance, creep and diffusivity tests in addition to a repeat of the preliminary tests, and

3) confirmation tests, similar in scope to the preliminary tests, for a trial mixture which was to be prepared and pumped by the contractor.

Concrete materials specifications were developed by NSDoT with support from DalTech, Jacques Whitford and Associates, CPCA and Concrete Canada. Information meetings were also held with the local ready-mix concrete and roadbuilders associations.

The bridge was tendered in July, 1997 as part of a larger contract for highway construction. A half-day workshop was held during the tender period to introduce the technology to potential bidders: contractors were able to hear what experience exists in Canada and elsewhere, what NSDoT's objectives were for the project and what items in the specifications were different from more familiar projects.

FIGURE 2.1: HPC bridge contributers attending the contractors' workshop, July 1997.

L/R: John Bickley, Gordon Leaman, Mark Pertus, Gary Pyke, John Salah, Al Gibson (Atl. Prov. Ready-Mix Concr. Assoc.), Jim Francis, Bill Dooley, J-F Trottier and myself. Missing are: Ray Snair, Peter Adams, Wib Langley, Brian Russell and Denis Mitchell.

3. BRIDGE DESIGN CONSIDERATIONS

3.1 General parameters

The prototype bridge is a two-lane secondary road over a new section of divided highway. It is part of a realignment and widening of a section of the TransCanada Highway in Pictou County, Nova Scotia, and is called the West River East Side Road Bridge. The bridge is in two spans with a total length of 70.7 m and width of 8.85 m. Supports are skewed by 25.5 degrees from the perpendicular axis. Design loading is CS600 (CSA S6-88).

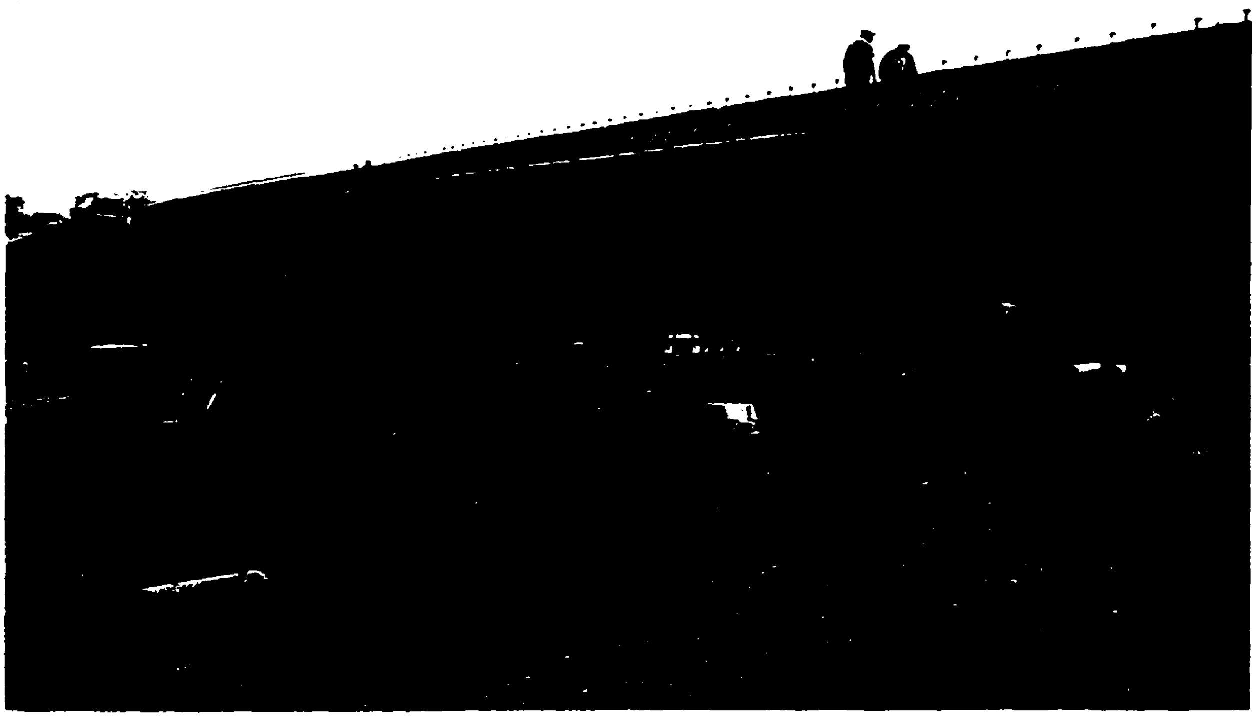


FIGURE 3.1: West River East Side Bridge under construction, Nov. 1997.

3.2 Structural design

Bridges in Nova Scotia are commonly built from precast concrete or steel girders and concrete decks with epoxy-coated reinforcing, about 50 mm of concrete cover, waterproofing membranes and asphalt concrete wearing surfaces. The bridges are expected to have a useful life of 80-100 years, though changing traffic patterns and design loads tend to reduce the life for which the original designs are valid to, say, 50 years.

3.2.1 Girder design

The West River East Side Bridge is a two-span structure using free-bearing end supports and continuity at the central support for live loading.

With prestressed components, the bending strength is a function of prestressing force, concrete compressive capacity and lever arm. Increasing the compressive strength of concrete and the prestress load enables longer spans and shallower, lighter and/or fewer sections. The advantage to increasing concrete strength may ultimately be limited by the ability to locate and stress the tendons, or quality assurance in design and materials, or the cost of concrete and turn-around time in the precasting shop.

For this project, four AASHTO bulb T sections are required for the bridge cross-section if standard concrete design strengths are used (30 MPa at release and 35 MPa at

28 days). The most effective result of increasing the concrete strength was found to be the use of three AASHTO bulb T's instead of four, with concrete design strengths of 54 MPa at release and 65 MPa at 28 days.

A total of 42 strands of 16 mm tendons are required for each girder, compared with 48 strands of 13 mm tendons for the reference design [Salah, 1997].

3.2.2 Deck design

The principal design objectives for the HPC concrete deck were to control the rate of diffusion of chloride ions from the surface of the deck to the deck reinforcement, to limit the potential size of any corrosion cell at the reinforcement and to limit the amount of wear at the surface. The HPC specifications were designed to limit corrosion over the design life of the bridge without the use of epoxy coating for reinforcement or a waterproofing membrane. The concrete cover over the top reinforcing of the deck was specified to be a minimum of 65 mm, which allowed 15 mm for the surface profile and construction tolerances. Other specifications included:

- rapid chloride permeability, per ASTM C1202: less than 600 coulombs at 91 days
- air-void spacing factor: 0.26 mm maximum; 0.23 mm average, and
- special provisions to control shrinkage cracking.

With these specifications it was considered possible that the deck reinforcement would be adequately protected to minimize corrosion without the use of epoxy coating of the reinforcement or a waterproofing coat of the deck. Mechanical wear of the concrete surface was not considered to be an issue for the traffic loading expected for the bridge.

As detailed below, there is a significant potential for saving life-cycle costs by eliminating the waterproofing and asphalt. The effectiveness of the HPC will be determined as the protype bridge is exposed over time. The epoxy-coating for reinforcing was eliminated for the prototype bridge which will enable chloride penetration to be detected more readily, but the cost saving is minimal and epoxy-coated reinforcing is recommended for general construction with HPC until there are successful data from this and other prototype structures.

The reference design included a deck thickness of 200 mm with a minimum cover of 50 mm to epoxy-coated reinforcing, a waterproofing membrane and an asphalt concrete wearing surface with an initial thickness of 50 mm. The deck was designed to span 2.4 m between girders. It was found that, using HPC and additional reinforcement, the standard depth to reinforcing could be maintained for the span of 3.0 m between girders. Deletion of the waterproofing and asphalt also reduced the applied dead weight, enabling less reinforcement to be used. However, an additional thickness of 15 mm was used for the exposed concrete design to provide for the finished profile of the deck and tolerances in the placement of reinforcing steel.

3.2.3 Substructure and piers

The same concrete mixture was specified for the substructure and piers as for the deck: this enabled everyone involved in the project to gain experience with the material before the more critical components were built. Also, durability is an issue for all exposed concrete surfaces, particularly the abutments and beams, and HPC is recommended for the columns for its improved compressive strength characteristics.

4. BRIDGE MAINTENANCE ISSUES

The maintenance of bridges is considered to be a significant issue to NSDoT. Ongoing improvements to design details, materials and quality control during construction are helping to mitigate future maintenance costs, but it is expected that bridge maintenance will continue to be a significant part of NSDoT's budget.

A review of existing maintenance records [Adams, 1996] found that asphalt concrete wearing surfaces, waterproofing and joint fillers frequently need to be replaced after 8-12 years of service; expansion joints may be replaced after 10-30 years and concrete deck delaminations may be repaired after 15-30 years. All deck repairs include the replacement of waterproofing and asphalt and related traffic control; concrete rehabilitation may include the repair of delaminated concrete in the deck, girders, beams, columns and/or abutments, plus the replacement of expansion joints, waterproofing and asphalt and related traffic control.

The impact of maintenance on life-cycle costs was assessed using a schedule of deck repair every twelve years and rehabilitation every thirty-five years, which accounts for ongoing improvements in design details and construction quality. As detailed in the economic analysis below and Appendix F, the equivalent present value of maintaining a conventional bridge is in the order of 20% of the initial construction cost, or 16% of the total life-cycle cost, and at least 75% of this cost relates to maintenance of the concrete deck and wearing surface.

Many of the causes of bridge deterioration and repair relate to the materials specifications for concrete and asphalt. Bridge decks which use a waterproof membrane and asphalt wearing surface are susceptible to loss of protection for the concrete due to wear or failure of the asphalt layer.

FIGURE 4.1: Highway 102 Bridge, July, 1997

If repair or replacement is not carried out when required, the concrete may be exposed to severe chloride attack in combination with freezing and thawing and variation in moisture content. Decks are particularly susceptible to attack at expansion joints (Fig. 4.2), where salt-water intrusion may damage the precast girders and supporting beams below.

FIGURE 4.2: Highway 102 Exit 9 Overpass, July, 1997

Exposed concrete decks were used in Nova Scotia until the 1970's, but went out of favour because delaminations in the concrete decks were causing an unacceptable level of maintenance (Figs. 4.3 and 4.4). The condition was aggrevated by relatively porous concrete mixtures, freezing and thawing cycles and, in many cases, alkali-aggregate reaction.

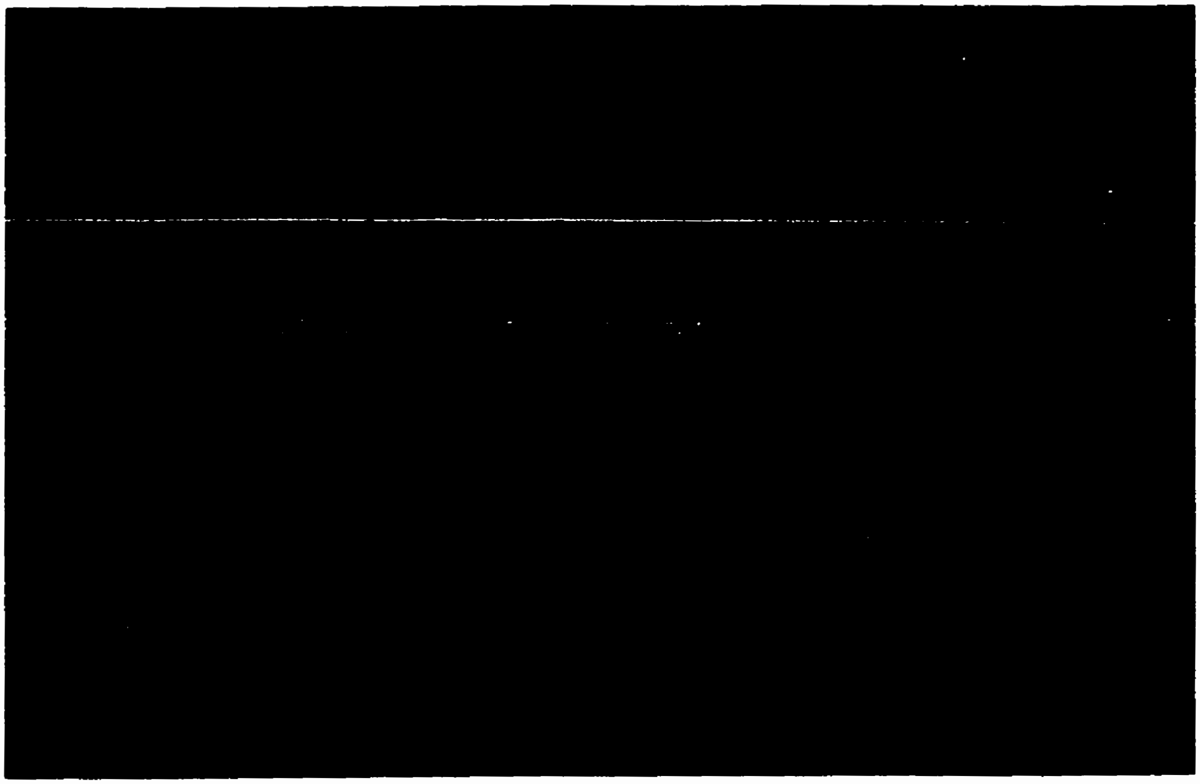


FIGURE 4.3: Highway 102 Exit 8 Overpass, July, 1997 (a)

- showing effects of chloride diffusion and corrosion of reinforcing on exposed concrete deck and along the curb

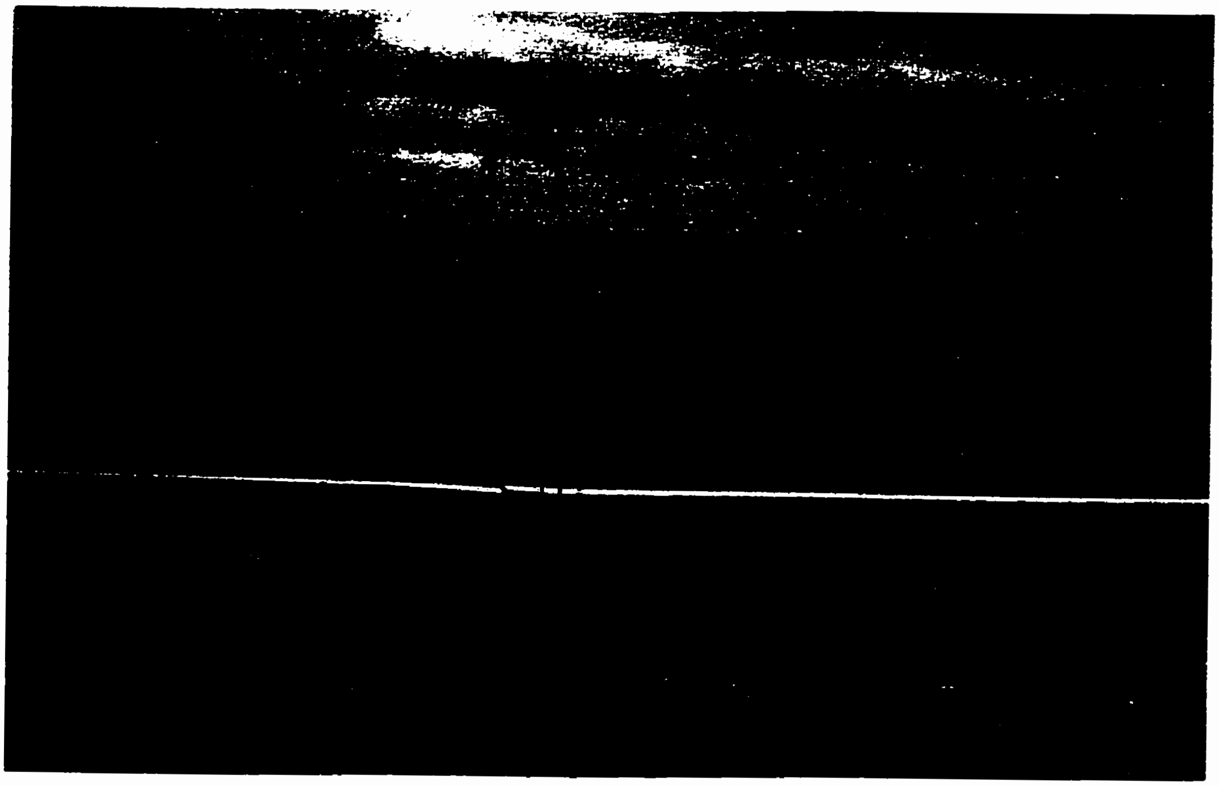


FIGURE 4.4: Highway 102 Exit 8 Overpass, July, 1997 (b)

- showing many of the typical ingredients of deterioration:
  - surface abrasion,
  - diffusion,
  - corrosion,
  - mechanical wear, probably by snow-plows, and
  - alkali-aggregate attack

Southern and central Nova Scotia is notorious for alkali-aggregate reactivity: Fig. 4.5 describes the condition of a bridge abutment that is typical for many older structures which are now in need of replacement. Fig. 4.6 is an abutment on the Centennial Highway in Halifax; it is about 30 years old and was being patched at the time of the photograph, as were the curbs and large portions of the deck the end-walls. The deterioration is the combined effect of alkali-aggregate reaction, chloride attack, freezing and thawing cycles and possibly other factors.

FIGURE 4.5: Railway bridge, Halifax, NS, July, 1997

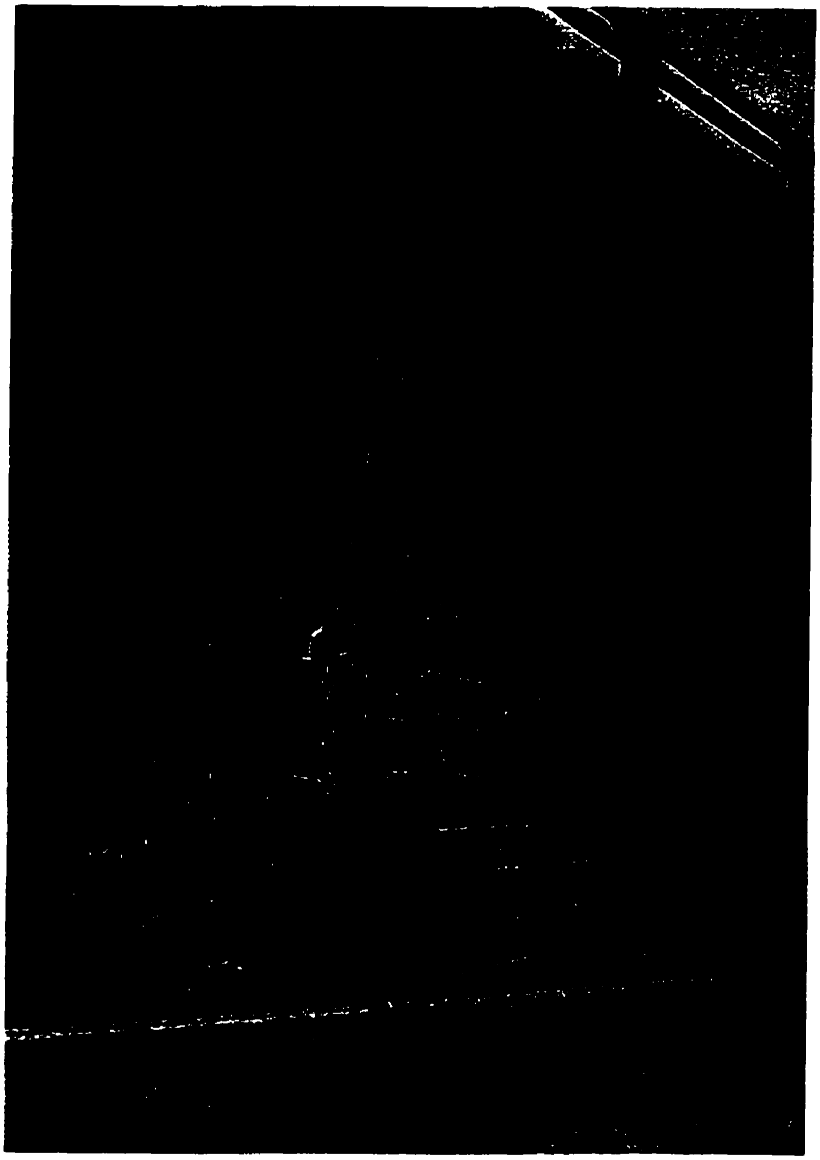


FIGURE 4.6: Bridge abutment, Centennial Highway, Halifax, NS, July, 1997

5. CEMENT AND CONCRETE CONSIDERATIONS

Some aspects of the chemistry and mechanical properties of hydraulic cement and concrete are detailed in Appendix a. The appendix describes the influence of aggregate strength and gradation, type and quantity of cementitious material and water content on the strength and stiffness of concrete; also the effect of pozzolans (silica fume and fly ash) on both durability and strength.

The success factors for concrete in the construction industry include convenience, economy, adaptability, strength, and durability. The raw materials have to be commonly available and economically manufactured. Practical, reproducible methods are required for mixing, placing and curing. Dimensional stability, compressive strength and durability are required of the final concrete product, and these qualities must be consistent. All these factors are interrelated and frequently an optimum result is the balance of conflicting objectives.

5.1 Dimensional stability

Dimensional stability relates to shrinkage, elastic stiffness and creep:

a] Shrinkage creates deformations and stresses in finished concrete that are significant from a design and performance viewpoint. Types of shrinkage include [Neville, 1987]:

- plastic shrinkage during setting of the plastic mix ($1000x10^{-6}$ to $2500x10^{-6}$, increasing with increased cement content),

- autogenous shrinkage due to hydration ($50x10^{-6}$ to $100x10^{-6}$),

- drying shrinkage: $0\text{-}800x10^{-6}$, due to drying out of capillary pores. Drying shrinkage includes a reversible portion in the order of 40-70% of the total drying shrinkage,

- carbonation shrinkage: $0\text{-}800x10^{-6}$, due to reaction between carbon dioxide gas and the hydrated cement, and

- wetting expansion: the reversible portion of drying shrinkage.

b] Initial deflection under load is approximately linear at low levels of loading but increases non-linearly with the size and duration of loading. A secant modulus is used to represent initial deflection under sustained design stresses (typically 20-30 GPa for traditional concretes), resulting in strains in the order of $500x10^{-6}$ at 50% of f_c'. The secant modulus of concrete is greater than that for cement paste and less than that for aggregates. Both the deformation under load and the Poison Ratio of concrete increase dramatically with loading to greater than $0.3f_c'$ because of the growth of bond cracks at the interface of cement and aggregate particles.

c] Creep, the increase in strain under sustained constant stress, is typically 1-3 times greater than the initial deformation under load. Creep is a long-term process: characteristically, it takes 14 days for the first 25%, 3 months for 50% and a year for 75% of the full long-term creep to develop [Neville, 1987].

The extent of creep deformation depends on the degree of hydration before and after application of the load: the long-term creep of concrete loaded at 28 days after mixing may be in the order of double that for concrete loaded one year after mixing. The ambient humidity and shape of specimen are also significant, with long-term creep in submerged conditions (100% relative humidity) characteristically one-third that for 50% humidity. Creep is also dependent on the magnitude of load, with larger creep values experienced when the stress exceeds 50% of f_c'. Removal of loading results in some instantaneous recovery, some creep recovery and some residual deformation.

Long-term deflection is frequently a limiting criteria in the design of beams and slabs. It also influences the buckling behaviour of eccentrically loaded columns and causes loss of prestress in prestressed reinforcement.

5.2 Compressive strength

The compressive strength of concrete is usually less than that of the paste or the aggregate. Where laterally unrestrained, strains and fractures along discontinuities in the matrix cause lateral tensile stresses which ultimately cause bursting of the concrete section. Lateral restraint of the concrete results in much higher compressive load resistance, with failure due to crushing.

The compressive strength of concrete is the major determinant of the strength of columns, arches, prestressed structures and other elements subjected to compression; it has a lesser influence on the strength of members subjected to bending.

5.3 Durability

Durability is the resistance of concrete components to environmental influences that, over time, damage the concrete.

Physical influences on durability include frost action, mechanical wear, temperature changes, changes in volume of the concrete constituents and variations in loading. Chemical influences include attack by de-icing and/or sea-water salts, attack by sulphates, chlorides, organic acids and carbon dioxide, leaching, long-term changes in composition of the cement gel and alkali-aggregate reaction.

The durability issues focussed on for the development of HPC included:

a] Freezing and thawing of concrete in a saturated condition, which results in pore water and vapour migration and freezing and thawing of water in the cement/pore matrix and aggregate particles. Expansion and contraction of concrete occurs which may disrupt the microstructure of the the paste, cause localized cracking of the paste and aggregate and/or damage of concrete structures at their restraints. Pore water solutions are also altered and solute migration occurs.

Materials design for resistance to freeze/thaw attack includes:

- maintaining very low permeability of the cement paste for the life of the structure, which requires a mixture design that has few, small pores, prevents leaching and is dimensionally stable,
- suitable aggregate selection and drying to prevent damage when frozen, and
- a well-distributed system of air-voids, with particular attention to air-void spacing.

b] Resistance to abrasion is of concern in the event that concrete bridge decks are exposed to traffic. Abrasive resistance is strongly correlated to concrete strength, low permeability and resistance to freeze/thaw attack. Methods of placing, finishing and curing concrete and of designing the physical details of the concrete elements also have a strong influence on the success of maintaining a resistive surface.

c] Sulphate attack is a potential form of chemical attack in which sulphates in acids or sodium and/or potassium solutions react with components of the cement gel and free calcium hydroxide. Calcium sulphate and sulphoaluminate deposits are formed and are characterized by cracking (due to an increase in volume) and white formations. Magnesium sulphate also reacts with the gel very slowly if the pH remains above 10.5; however, if leaching reduces the pH then a non-binding deposit of magnesium silicate is formed which further reduces the strength of concrete [Lea, 1970].

d] Chlorides: gypsum and ettringite (calcium sulphate and calcium sulphoaluminate) are more soluble in the presence of chloride ions than sulphate ions: calcium formations tend to leach out of concrete in the presence of chloride solutions, eg. road salt, resulting in an increase in porosity and a potential for galvanic pitting of reinforcement.

Other forms of chemical attack are detailed in Appendix B. In all cases, the concrete's permeability, its chemical composition and air entrainment are critical to the rate of deterioration.

6. HIGH-PERFORMANCE CONCRETE

Initially, the development of high-performance concretes was generally to improve compressive strength. However, the use of superplasticizers, low water content and low permeability have led to advanced placement techniques and greatly improved durability features.

"High strength concrete" is a type of HPC. The Portland Cement Association [Farny, 1994] describe high strength concrete as having a strength in excess of 50 MPa and considers the ultimate limit for 90-day strength to be in the order of 200 MPa (29 Ksi).

Concrete strengths in the range of 70-150 MPa have been developed and used in:

a] large offshore structures and bridges in Europe and Canada,
b] high-rise buildings in North America,
c] general prefabrication (high early strength), and
d] short and medium span bridges in North America [eg. Farny, 1994].

Strengths above 150 MPa have also been used in prefabricated struts, eg. in reticulated 3-D structures in Europe and at Sherbrooke, Quebec [Aitcin, 1997].

The mixing and placing of HPC may be similar to ordinary-strength concrete, with plasticizers added at the batching plant and/or in increments at the site. The use of HPC requires close cooperation between the engineer, the concrete producer, the construction contractor and the testing agency [Farny, 1994, Ryell, 1993].

6.1 Mixture design issues

Characteristic mixture proportions include:

a] a cementitious content of 360-600 kg/m^3 (600-1000 lb/yd),

b] a water/cementitious ratio of 0.22-0.35,

c] the use of pozzolans (fly ash, blast-furnace slag and/or silica fume), and

d] as much as 10 L/m^3 of superplasticizer.

The cement in HPC forms an amorphous microstructure with less porosity than traditional concrete, particularly at the aggregate/paste interface. This results in more stress being transferred through the aggregate and greater durability and strength. The reduced porosity of the cement paste is achieved through controlling the particle size distribution of the cement and admixtures to maximize the composite density, and by limiting the water and sulphate contents.

Pozzolans, including fly ash, silica fume and ground blast-furnace slag, are recommended as a mineral admixture for high strength concrete, and silica fume is recommended for strengths above 100 MPa [Farny, 1994]. Fly ash and blast-furnace slag may be considered as supplementary ingredients in HPC and are used primarily to reduce the superplasticizer dosage and possibly eliminate the need for retarders; fly ash also contributes to the long-term strength of concrete without increasing the peak heat of hydration significantly. Silica fume, with an average particle size of 0.1 um, reduces the porosity of the cement paste and also reacts with lime to contribute to gel strength.

Research in Japan [Utica, 1991] indicates that a "classified fly ash" can also be used to improve workability and reduce the cost of 70 MPa concrete. Also, 60 MPa concrete has been developed for building construction without fly ash, silica fume or blast-furnace slag by using a water/cement ratio of 0.285 and superplasticizers [Tachibana, 1994].

Coarse and fine aggregates may be optimized in relation to the strength of the paste and the cement/aggregate bond. The perfect aggregate would be crushed, clean, cubical in shape and conditioned to a "saturated, surface dry" condition. The bond is more critical for coarse aggregates, while the shape and grading are more significant for fine aggregates. For example, in some Norwegian offshore construction, the sand is classified into eight fractions which are then blended for concrete manufacture [Collins, 1993].

Entrained air reduces the density and strength of the cement paste: a strength reduction of 5% was observed in this project for an increase in air content of 1%. However, at strengths above 150 MPa, the need for entrained air may be eliminated by the reduced significance of pores capable of retaining freezable water.

6.2 Examples of applications - general

In Canada, HPC has been used in the Confederation Bridge [eg. Dunaszegi, 1996], the Hibernia offshore structure [eg. Collins, 1993] and high-rise buildings [Farny, 1994]. In offshore construction, the benefits of HPC include:

a] durability (low permeability),

b] structural - reduced cross-sections for prestressed elements, platform cells and arches, which are principally in compression, and

c] constructibility - high slump concrete with good cohesion is required where reinforcement is congested.

In the Toronto area, 70-85 MPa concrete has been used in Scotia Plaza, BCE Place and the Adelaide Centre, using cementitious blast-furnace slag and silica fume in conjunction with Type 10 Portland Cement [Ryell, 1993]. In Halifax, HPC has been used in the Park Lane and Purdy's Wharf II developments [Langley, 1989]. The advantage in using HPC is economic, with savings in concrete mass and volume, in construction formwork, and in retail space lost to structure. Examples in the United States include high-rise structures utilizing large-diameter steel-encased 'super-columns' and conventional rigid-frame structures [eg. Moreno, 1990]; however, the economics of using HPC in rigid-frame high-rise structures are considered to be not attractive at present.

6.3 Examples of applications - short and medium span bridges

HPC has been used for precast and for in-situ concrete girders and for exposed bridge decks in Quebec [eg. Mitchell, 1993] and Ontario (Fig. 3.1); [Bickley, 1996]. In the United States, the Strategic Highway Research Program (SHRP) has assisted in the development of a number of HPC bridges and related products utilizing HPC in Texas, Alabama, Georgia, North Carolina, Colorado, Nebraska, Ohio, Virginia, Washington and New Hampshire [Federal Highway Administration, 1997; Ralls, 1994, 1996; Roller, 1995]. Advantages in using HPC include:

a] durability - resistance to freeze-thaw attack, chloride attack and scaling because of low porosity and permeability and higher cracking loads,

b] structural - increased flexural and compressive strength; reduced elastic shortening and creep under prestress; reduced number and/or size of girders, and

c] constructibility - reduced time in prestress beds; reduced transfer length and "harping" of prestress tendons.

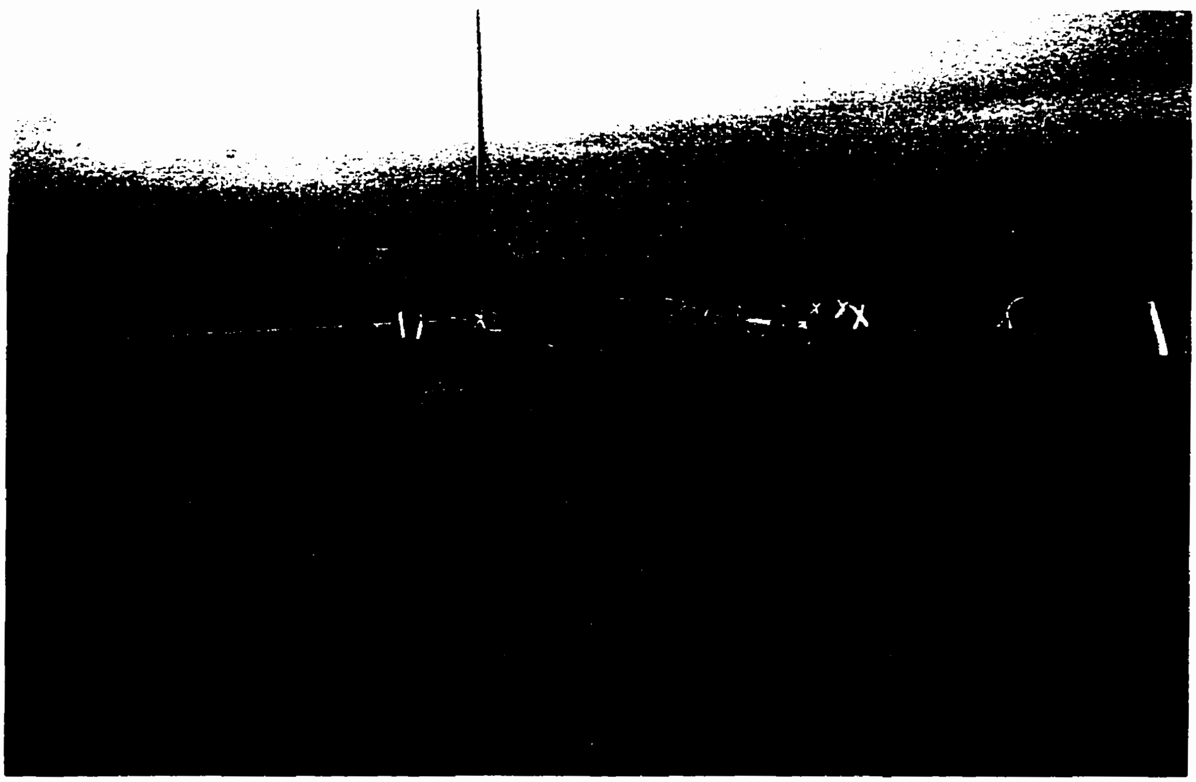


FIGURE 6.1: HPC bridge, Highway 407, Ontario, Oct. 2nd, 1997

7.0 MIXTURE DESIGN AND TESTING PROGRAM

7.1 Scope

The Technical University of Nova Scotia (now DalTech) was commissioned to provide the concrete mixture design for the HPC bridge construction specification. Terms of reference included:

- aggregate sampling,
- initial trial mixture testing,
- advanced testing and
- ready-mix trials.

The initial trial mixture testing included the design, production and preliminary testing of six mixtures in the laboratory from which a concrete mixture was selected for detailed investigation. Advanced testing was carried out on the selected design mixture to confirm its mechanical properties and to evaluate durablity characteristics. While the ready-mix trials described in the terms of reference were not carried out, construction methods and materials testing results were reviewed during construction and are included in construction notes below.

The testing program was carried out in Jacques Whitford's laboratories, Dartmouth, NS, with the exception of salt scaling and flexural fatigue endurance tests which were carried out at DalTech.

7.2 Structural and material design objectives

It was determined that the design loading could be carried by three bulb-T sections in each span and that locally-available prestressed beds were capable of providing the required prestress force. A concrete compression strength, f'c, of 65 MPa was required for the precast concrete at 28 days.

Concrete that was to be cast in place was designed primarily for durability; a nominal compression strength of 60 MPa at 28 days was selected for design. The concrete deck is the most critical component from a durability viewpoint: it was designed to be exposed without repair for 80 years. The materials design objectives for durability of the HPC mixture included a rapid chloride permeability of less than 600 Coulombs at 91 days and air-void spacing factors of 0.26 mm (max.) and 0.23 mm (average).

A single mixture was designed to meet the criteria of both the precast and the deck elements. The mixture was also used for all other concrete work at the site, which enabled the contractor to become familiar with the material before placing the deck.

The concrete mixture design was to be specified as the base option for contractors; alternative concrete mixtures could be accepted on review of appropriate test data by the contractor.

7.3 Materials

7.3.1 Coarse aggregate

The testing program used a granitic natural stone, commonly available to potential ready-mix concrete suppliers in the area. The aggregate was from Will-Kare's Folly Lake quarry and was collected from Fundy Ready-Mix. A nominal aggregate size of 20 mm was used for the testing program, though construction included the specified 20 mm aggregate for cast-in-place concrete and 13 mm crushed aggregate in precast units.

Testing was in accordance with CSA A23.2 (1994):

- average bulk specific gravity (saturated surface dry): 2.79; absorption 0.75%
- petrographic number (NSDoT method): 108
- aggregate soundness - average loss (5 cycles, $MgSO_4$): 1.87%
- LA abrasion test - loss at 500 revolutions: 15.47%

TABLE 7.1: Sieve Analysis - Coarse Aggregate

sieve opening (mm)	percent passing - measured	percent passing - specification
28	100	100
20	98.0	90-100
14	71.0	-
10	40.7	25-60
5	2.7	0-10
2.5	1.1	0-5

7.3.2 Fine aggregate

Sand also from a Will-Kare quarry and was obtained from Fundy Ready-Mix. The sand had an average fineness modulus of 2.78, with 2.3% passing the 80 um sieve. Again, testing was in accordance with CSA A23.2 (1994):

- average bulk specific gravity (sat. surface dry): 2.702; absorption 1.33%
- aggregate soundness - average loss (5 cycles, $MgSO_4$): 5.61%
- aggregate soundness - average loss (micro-deval test): 7.81%

TABLE 7.2: Sieve Analysis - Concrete Sand

sieve opening (mm)	**percent passing - measured; sample (a)**	**percent passing - measured; sample (b)**	**percent passing - specification**
5	100	100	95-100
2.5	85.4	86.2	80-100
1.25	62.7	63.0	50-90
.63	38.9	39.8	25-65
.315	18.0	19.9	10-35
.160	7.5	18.3	0-25
.080	2.4	2.2	0-1.7

7.3.3 Cement/silica fume

Type 10 low-alkali silica fume cement was used for all mixtures, supplied by Lafarge Canada. This blended cement includes approximately 8% silica fume.

7.3.4 Fly ash

Class F fly ash was used, supplied by Shaw Resources, Nova Scotia. A detailed analysis is included in Appendix B.

7.3.5 Retardant/water reducing admixture

Grace Construction Products' "Daratard 17" was used for the testing program to simulate possible field conditions for delayed setting of the concrete, and as a water-reducing agent during the initial batching of the concrete. "WRDA-82" was also specified for water-reduction without delay in setting. (Retardants are typically sodium or calcium triethanolamine salts of hydrogenated adipic or gluconic acids.)

7.3.6 Air-entraining admixture

The air entraining agent was added with the initial batching to correspond to batching conditions for construction. It was found that additions were required both before and after the addition of superplasticizer in some cases to obtain the required air content. Grace's "Darex II" was used for all preliminary mixtures and some advanced testing. MasterBuilder's "MicroAir" was also used for advanced testing.

7.3.7 Superplasticizer

Grace's "WRDA19" was added as required to obtain a slump of approximately 200 mm. (Superplasticizers are deflocculation agents - see Appendix a.)

7.4 Laboratory mixing and testing procedures

Concrete was batched in the laboratory in an Eirich 150 L counter-current mixer (Fig. 7.1). Actual batches ranged from 100 L to 140 L in size. The coarse and fine aggregates and water were cooled before batching as required to maintain a concrete temperature of 15 - 20 ^{0}C while mixing. The batching process included:

- the dry ingredients were mixed;
- water, water-reducing admixture and air-entraining admixture were added;
- the batch was mixed;
- initial slump and air content were measured;
- superplasticizer was added;
- the batch was mixed;
- slump and air content were measured;
- air-entraining admixture and/or was added as required;
- mixing, measurement and additions were repeated as required;
- final slump, air content and temperature measurements were recorded, and
- test samples were moulded.

Cylinders used for compressive strength testing were 100 mm in diameter, while cylinders used for testing modulus of elasticity had a diameter of 150 mm. The ends of all cylinders were ground before testing (Fig. 7.2). Testing included pairs of compressive strength measurements at 1, 3, 7, 28 and 91 days (Fig. 7.3) and modulus of elasticity measurements at 7 and 28 days (Fig. 7.4).

Rapid chloride permeability testing to ASTM C1202 (Fig. 7.5) and hardened air content characteristics (Figs. 7.6 and 7.7) were also measured for all batches.

Advanced testing included an analysis of salt scaling (Fig. 7.8), creep (Fig. 7.9), flexural fatigue (Fig. 7.10) and adiabatic heat development. Diffusivity testing was also proposed (Fig. 7.11) to determine the rate of chloride penetration into hardened concrete.

FIGURE 7.1: Eirich 150 L counter-current mixer

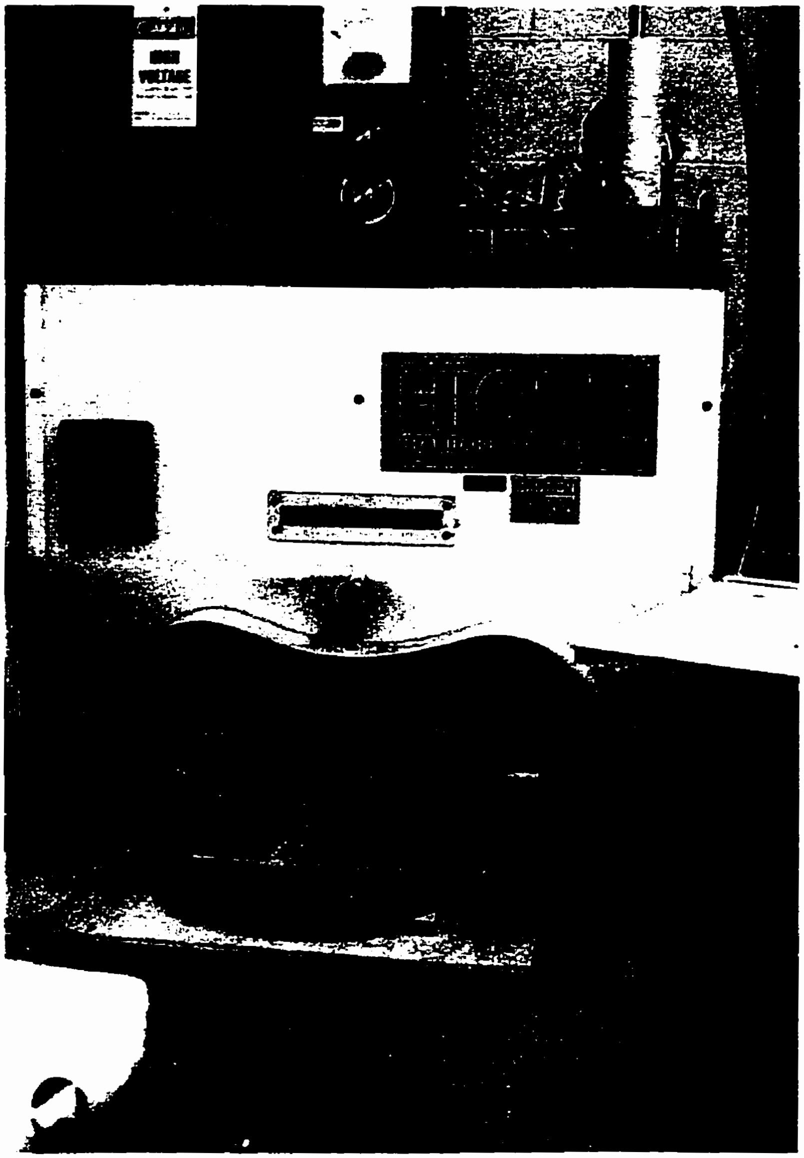


FIGURE 7.2: Hicut cylinder end-grinder

- including cylinder clamp and moving grinder; covers removed

FIGURE 7.3: Technotest compressive stress/strain testing system - 100 mm dia., 7-day cylinder after loading; shield removed

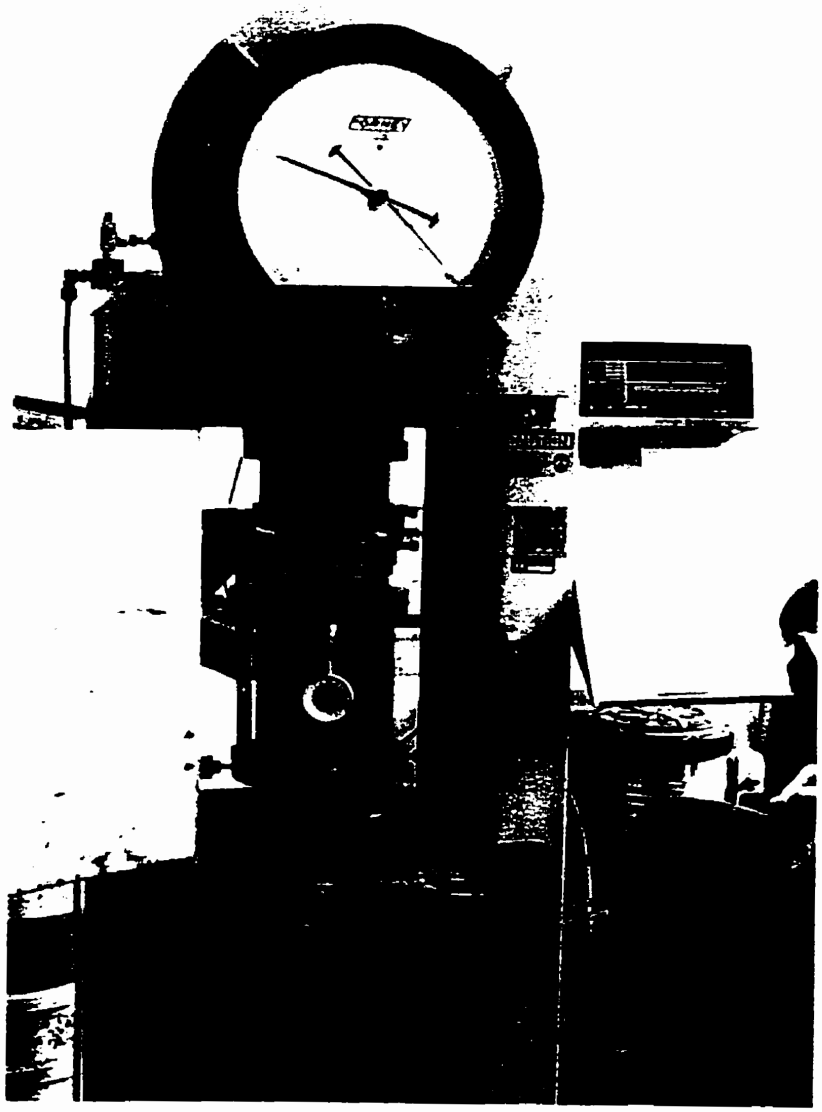


FIGURE 7.4: Forney loading device

- 150 mm dia. cylinder clamped for elastic modulus measurement

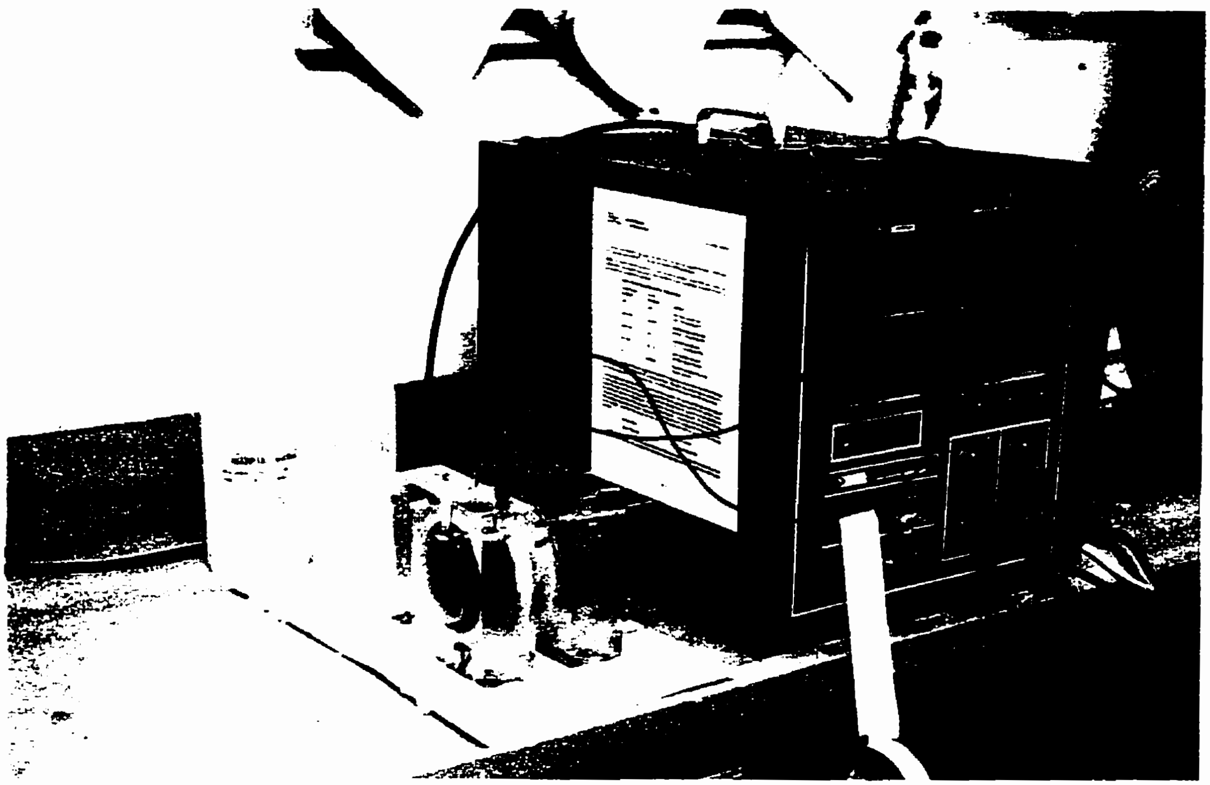


FIGURE 7.5: Rapid chloride test apparatus

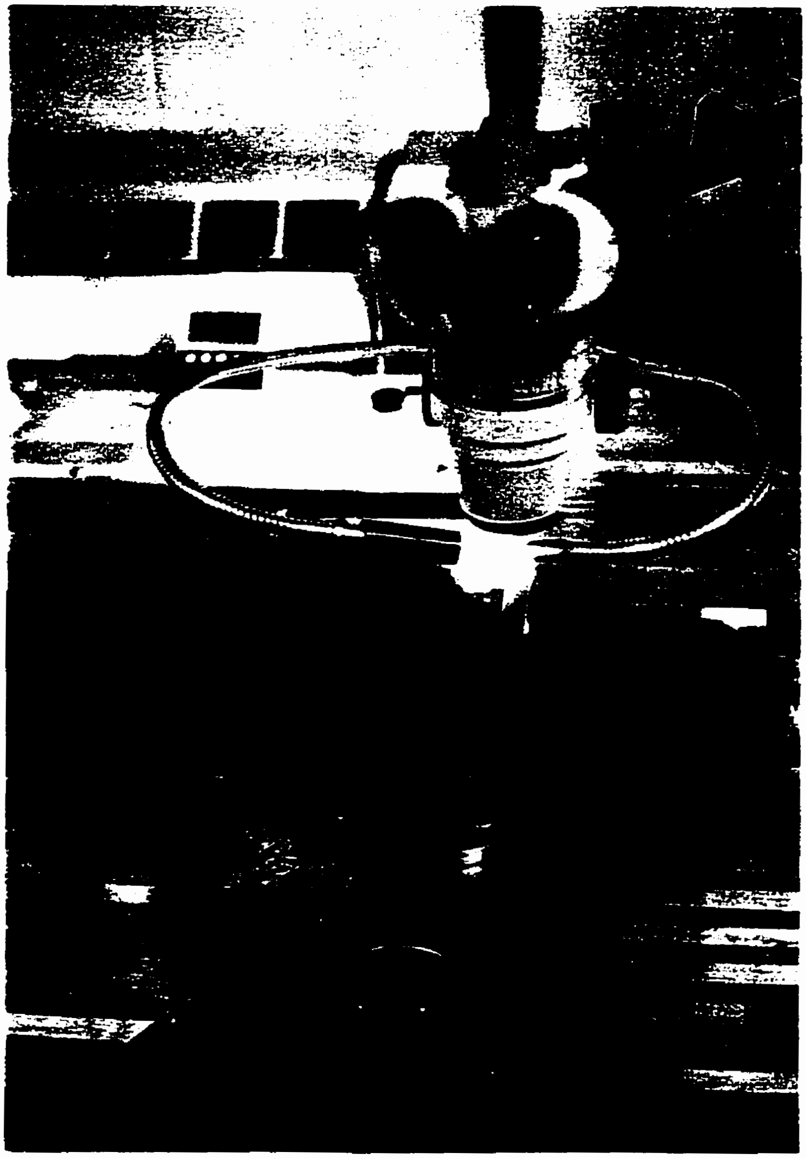


FIGURE 7.6: Hardened concrete air content: microscope and slide

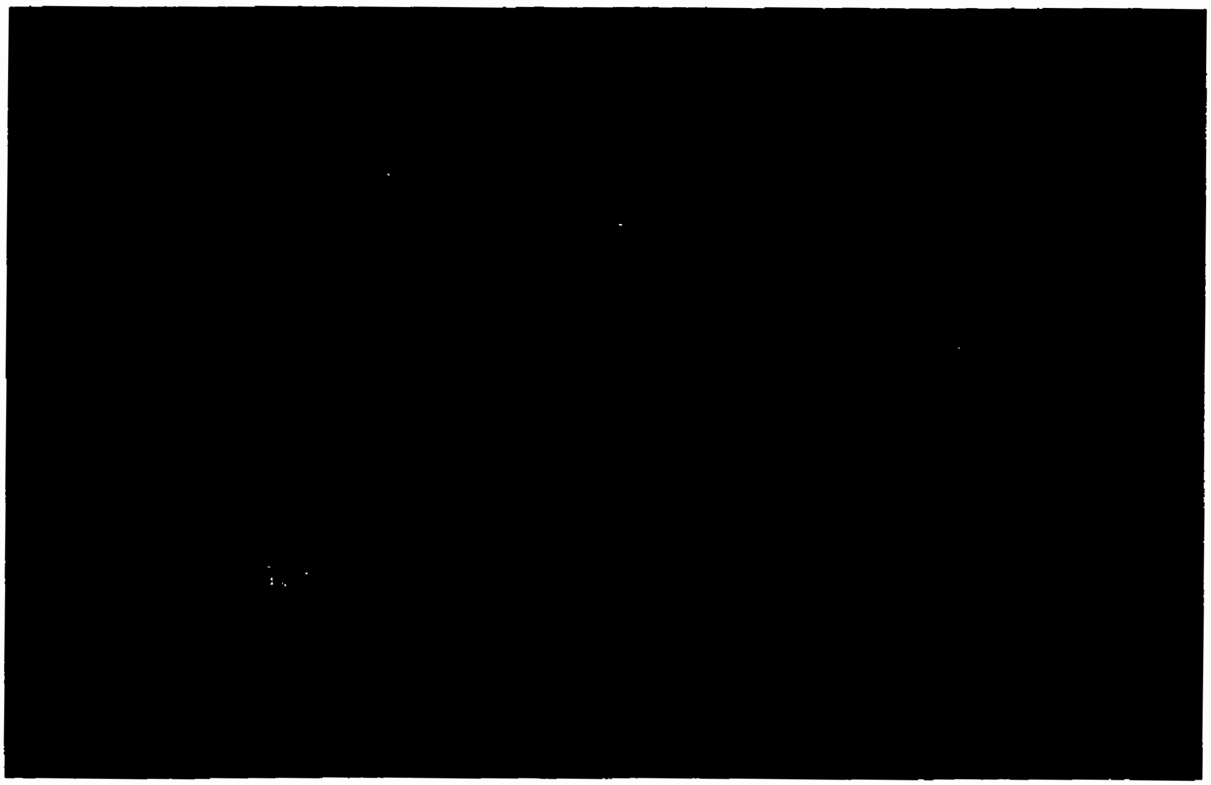


FIGURE 7.7: Hardened concrete air content: view of sample

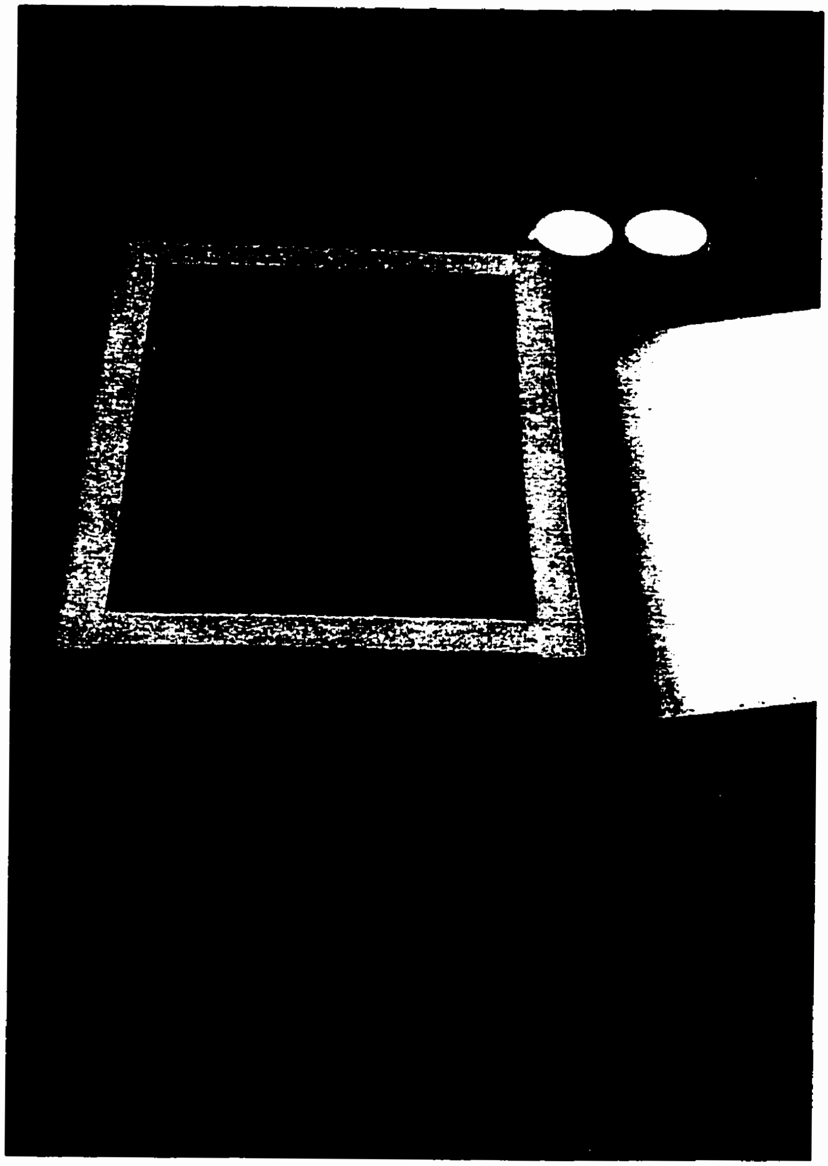


FIGURE 7.8: Salt scaling sample and removed surface fines

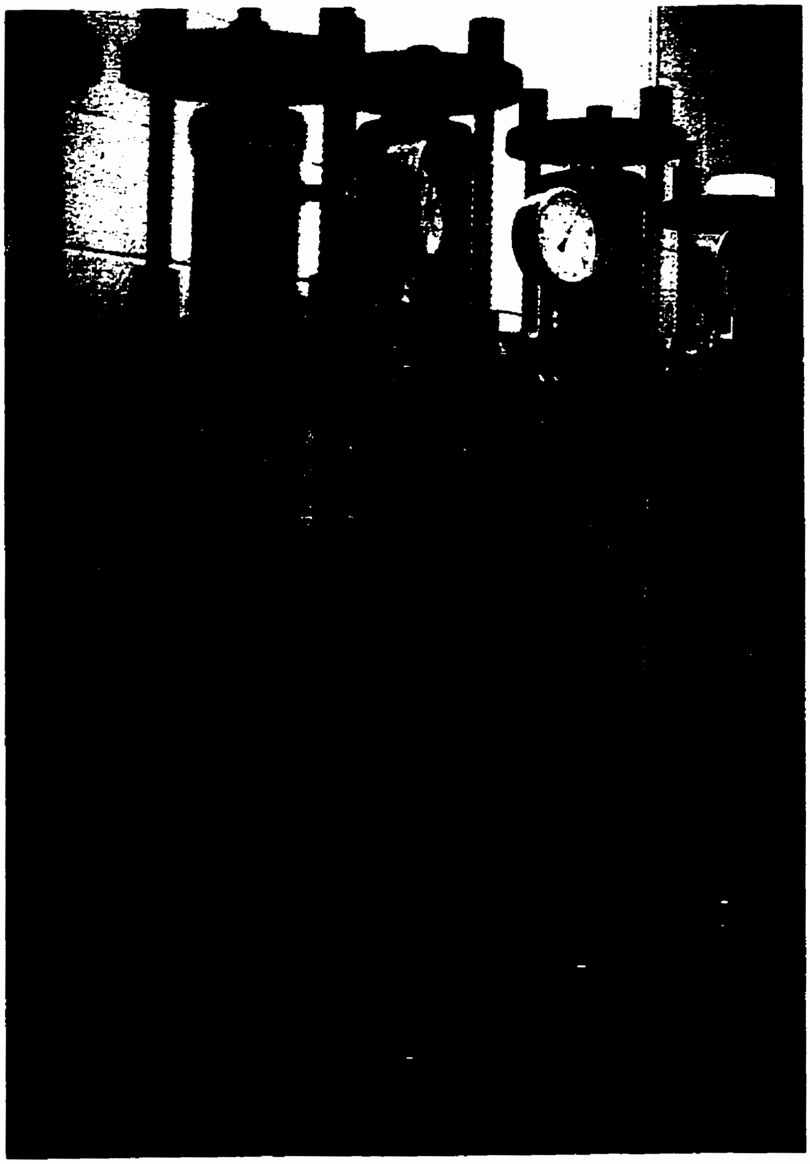


FIGURE 7.9: Creep loading frames and gauges

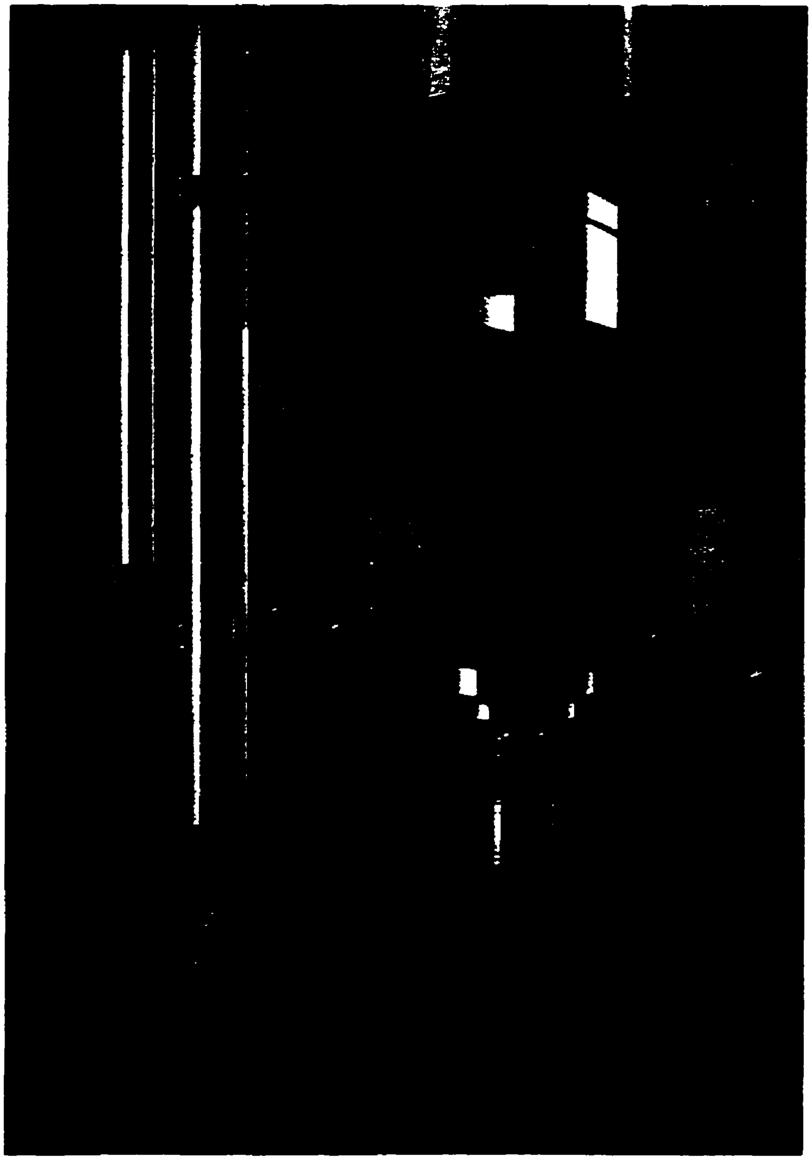


FIGURE 7.10: Flexural fatigue loading frames and gauges

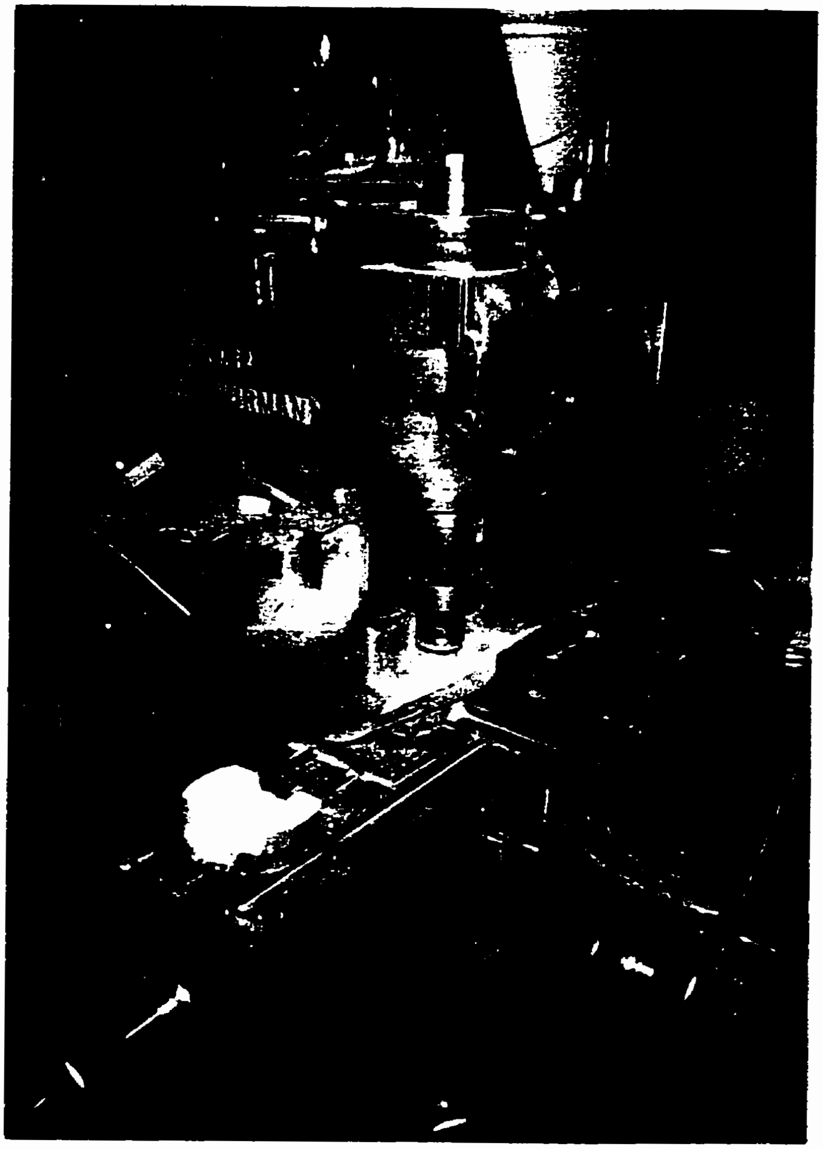


FIGURE 7.11: Diffusivity testing grinder and sample

(sample surfaces are exposed to a salt solution in controlled conditions before grinding)

- sample at one side has been ground to a depth of approx. 15 mm
- University of Toronto, Oct. 1997

7.5 Preliminary concrete mixture design

Given the desired strength and durability of the final concrete mixture design, an estimate was made of an appropriate range of cementitious products and water contents. Reference was made to the literature and to previous experience with high-performance concretes. A total of six concrete mixtures were selected, including three water/cementitious ratios (0.30, 0.33 and 0.36) and two fly ash contents (0 and 20%), to characterize the desired concrete mixture.

In all cases, the concrete mixtures were proportioned to enable the materials to be thoroughly mixed with only a limited amount of water-reducing agent or retarder before the addition of superplasticizer. It was expected that the materials would be batched at a ready-mix plant and mixed in transit, and that only superplasticizer would be added at the site. Accordingly, an initial slump of 20-25 mm was proposed, resulting in a minimum water quantity of 140 L/m^3 and water-reducing or retarding agent applied at a rate of about 2.5 mL per kilogram of cementitious material. Cementitious material quantities were rounded off at 475, 450 and 430 kg/m^3 for the specified water/cementitious contents, and the water contents were calculated to be 142.5, 148.5 and 154.8 L/m^3 respectively. Superplasticizer was added as required to obtain a final slump with a value of approximately 200 mm

Coarse and fine aggregates were selected to provide even grading with coarse aggregates increased where possible to maximize strength.

The preliminary design mixtures are presented in Table 7.3. Test results are described in Table 7.4 and Figures 1 and 2 below. "Equivalent strength results for 150 mm diameter cylinders" are tabled using 95% of the tested 100 mm cylinder strength: the factor reflects calibration tests in the literature but may not be relevant where testing equipment is capable of testing 150 mm cylinders to failure without significant deformation of the loading platens. The sets of data (100 and 150 mm diameter cylinders) could be considered as a probable range of result if 150 mm cylinders were to be tested.

TABLE 7.3: Preliminary concrete mixture designs

mixture no.		1	2	3	4	5	6
LASF	kg/m³	475.0	450.0	430.0	403.8	382.5	365.5
FA	kg/m³	0.0	0.0	0.0	71.3	67.5	64.5
total	kg/m³	475.0	450.0	430.0	475.0	450.0	430.0
water	L/m3	142.5	148.5	154.8	142.5	148.5	154.8
w/c		0.30	0.33	0.36	0.30	0.33	0.36
sand	kg/m³	713	716	716	710	712	712
stone	kg/m³	1,027	1,030	1,030	1,021	1,026	1,026
Darex II as required	L/m3	0.70	0.57	0.61	0.86	0.90	0.80

TABLE 7.4: Preliminary concrete mixture test results

mixture no.		1	2	3	4	5	6
slump:							
- initial	mm	25	25	50	25	30	80
- final	mm	200	170	205	185	185	170
density	kg/m^3	2,369	2,355	2,334	2,348	2,348	2,313
air:							
- plastic	%	6.90	7.00	7.00	6.80	6.90	7.20
- hardened	%	4.56	4.75	4.33	4.60	4.74	5.36
- spacing	mm	0.228	0.203	0.313	0.276	0.227	0.210
- SS	mm^{-1}	22.63	23.11	16.65	18.07	22.73	23.46
conductivity at 28 days	Coul.	754	971	1712	1059	1125	2074
cyl. strength 100mm dia.							
- 1 day	MPa	10.6	17.2	11.1	7.6	7.1	8.0
- 3 day	MPa	42.7	35.7	29.6	30.8	27.8	23.0
- 7 day	MPa	57.1	50.9	40.7	40.2	36.9	30.6
- 28 day	MPa	78.5	68.9	56.8	60.5	55.9	47.1
- 91 day	MPa	91.6	78.2	62.5	70.9	65.6	55.4
equiv. fc' of 150mm dia.							
- 1 day	MPa	10.1	16.3	10.5	7.2	6.7	7.6
- 3 day	MPa	40.6	33.9	28.1	29.3	26.4	21.9
- 7 day	MPa	54.2	48.4	38.7	38.2	35.1	29.1
- 28 day	MPa	74.6	65.5	54.0	57.5	53.1	44.7
- 91 day	MPa	87.0	74.3	59.4	67.4	62.3	52.6
modulus							
- 7 day	GPa	29.4	27.9	25.5	26.1	26.7	23.7
- 28 day	GPa	33.2	31.1	29.8	31.1	29.9	28.0

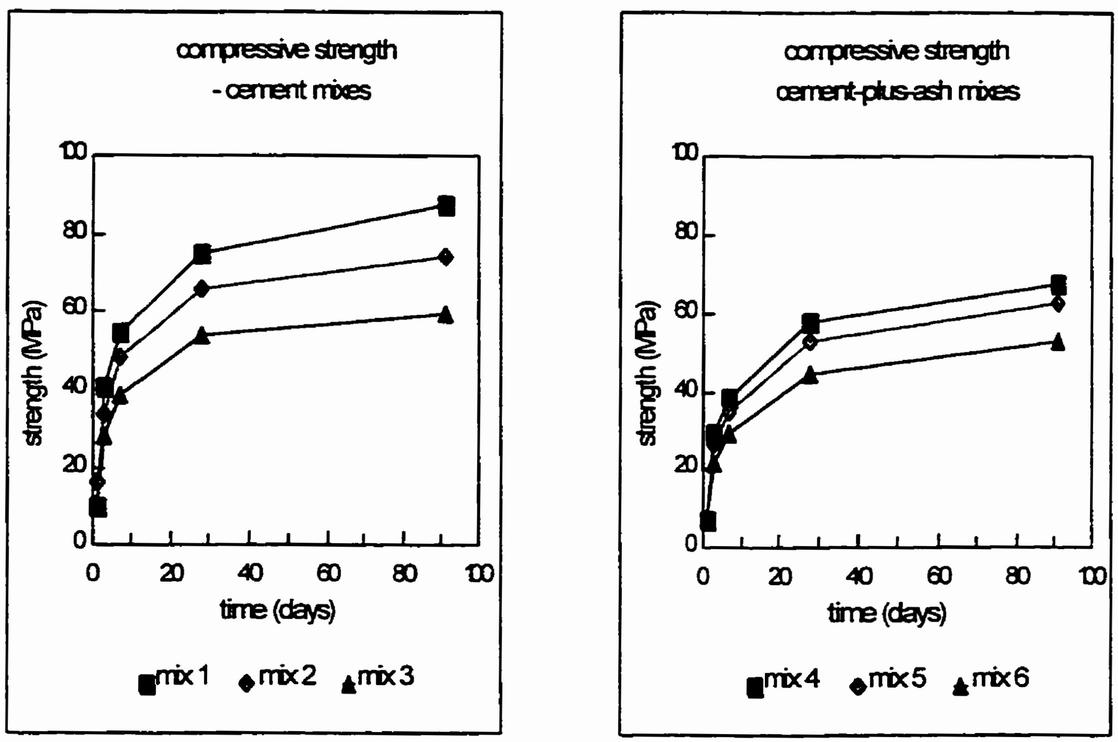
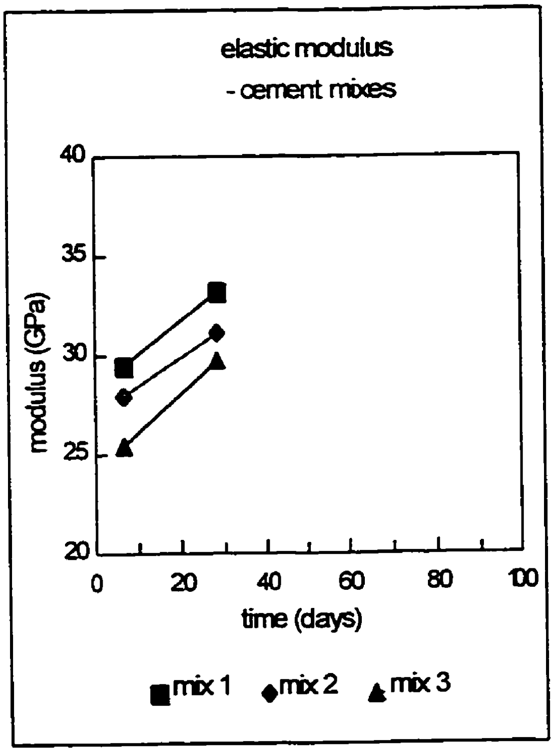


FIGURE 7.12: Preliminary concrete mixtures - compressive strength



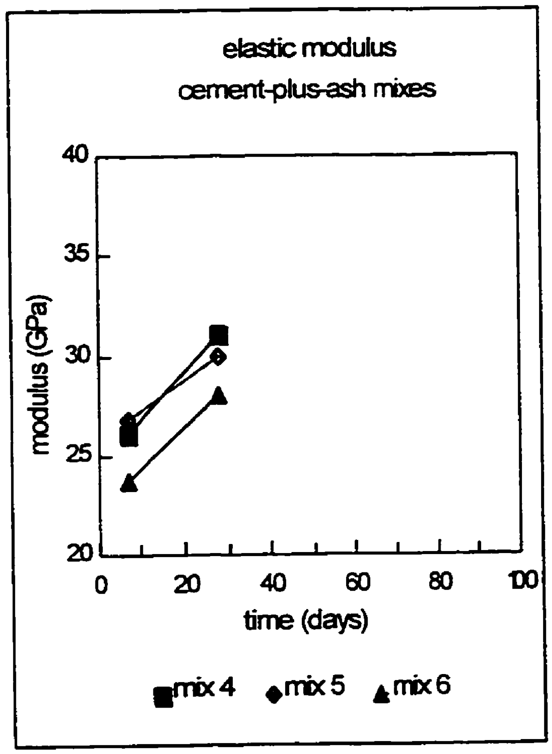


FIGURE 7.13: Preliminary concrete mixtures - elastic modulus

The preliminary tests indicated that a workable mixture with the required strength and durability characteristics could be developed using the described procedure for mixture design. The results indicated that a water/cementitious ratio of close about 0.3 and the use of some fly ash would provide a workable, economic mixture.

The design mixture selected for the advanced testing program and subsequent specification is outlined in Table 7.5.

TABLE 7.5: Design concrete mixture.

cement, silica fume	450 kg/m3	Type 10SF (low alkali)
fly ash	30 kg/m3	Class F
sand	690 kg/m3	fineness modulus 2.60-2.90
stone	1045 kg/m3	max. size 20 mm
water	144 kg/m3	water/cementitious 0.3
superplasticizer	3.5 L/m3	(as required)
air-entraining agent		(as required)
water-reducing agent		(as required)

7.6 Advanced testing

The concrete mixture selected for advanced testing, identified as Mixture #7, was batched on April, 1997. During batching, it was found to be difficult to obtain the required air content; this resulted in what was considered to be an excessive use of air-entraining agent, and was reflected in poor results for hardened air-void characteristics and salt scaling. In subsequent tests (Mixtures #8 and #9) the amount of air-entraining agent added at the start of batching was increased in an attempt to reduce the total amount required. Finally, an alternative air-entraining agent was used to provide the required air-void characteristics (Mixture #10).

However the original air-entrainment product was suitable for plant batching during construction. The difference between laboratory results and those in the field may be due to differences in the gradation of aggregates and/or mixing rates.

Mixture designs are included in Table 7.6. The tests carried out for the preliminary mixtures were repeated in the advanced testing program: results are summarised in Table 7.7. The results of the advanced tests are summarised in Table 7.8.

TABLE 7.6: Advanced concrete testing mixture designs

mixture no.		**7**	**8**	**9**	**10**
date batched		14-Apr-97	18-Jun-97	24-Jun-97	24-Jun-97
LASF	kg/m^3	450.0	450.0	450.0	450.0
FA	kg/m^3	30.0	30.0	30.0	30.0
total	kg/m^3	480.0	480.0	480.0	480.0
water	L/m^3	144.0	144.0	144.0	144.0
w/c		0.30	0.30	0.30	0.30
sand	kg/m^3	690	690	690	690
stone	kg/m^3	1,045	1,045	1,045	1,045
Darex EH	L/m^3	0.82	0.84	0.70	-
MicroAir	L/m^3	-	-	-	0.35
Daratard 17	mL/kg	2.50	2.50	2.50	2.50
WRDA 19	L/m^3	5.00	5.00	5.00	5.00

TABLE 7.7: Advanced concrete testing - test results (a)

mixture no.		7	8	9	10
initial slump	mm	25	50	25	50
final slump	mm	220	220	-	225
density	kg/m^3	2,390	2,362	-	2,313
air plastic	%	6.00	6.10	6.00	7.00
air hardened:	%	3.61		(discarded)	6.10
- spacing factor	mm	0.260			0.182
- SS	mm^{-1}	23.38			25.67
conductivity	28 day	1154			
(Coul.)	91 day	381			
cyl. strength					
- 100mm dia.					
- 1 day	MPa	0.8	-		-
- 3 day	MPa	42.6	-		-
- 7 day	MPa	56.7	54.3		-
- 28 day	MPa	77.0	73.5		-
- 150mm dia.					(tested:)
- 1 day	MPa	0.8	-		8.5
- 3 day	MPa	40.5	-		-
- 7 day	MPa	53.9	51.6		42.0
- 28 day	MPa	73.2	69.8		59.0
modulus of elasticity					
- 28 day	GPa	33.8	-		-

TABLE 7.8: Advanced concrete testing - test results (b)

mixture no.		**7**	**8**	**9**	**10**
salt scaling	kg/m^3	0.7	0.4		0.15
no. of cycles		50	20 (1)		50
creep at 170 d	10^{-6}/MPa	52.6	at 170 days		
flex. fatigue cap.	% of fb	50%			
no. of cycles		2,000,000			
description		normal			
difussivity		(2)			
adiabatic heat development:					
initial set maturity:	h	24.1			
heat dev.	kJ/kg	25.9			
final set maturity:	h	30.8			
	kJ/kg	82.3			
max. heat dev.	kJ/kg.h	12.5			
heat dev. at:	28 days (kJ/kg)	316.2			
	91 days (kJ/kg)	317.1			

notes: (1) test not completed; failure of ponding edge dam

(2) not completed; see discussion

7.7 Test results

7.7.1 Strength

Tests were carried out in accordance with CSA A23.2-6C. The advanced testing results indicate that the mixture is capable of providing compressive strengths in the order of 65-70 MPa in the laboratory and 60 MPa at 28 days in the field.

The preliminary tests indicate that a further strength gain of approximately 10 MPa may be expected between 28 and 91 days for concrete with this combination of cementitious and pozzolanic material.

The advanced testing also demonstrates that the amount of air entrained in the concrete has a major influence on the strength of the concrete. A single percentage increase in air content is reported to decrease the strength of high-performance concrete by an amount in the order of five to ten percent. Table 7.9 indicates an increase in hardened concrete air content of 2.5% between mixtures #7 and #10 and a decrease in compressive strength of 21%.

The strength of the concrete is highly dependent on its final air content. Even though the concrete may be specified to have a narrow range of air content when plastic, its strength is also a function of air-void stability during placement, setting and hardening.

TABLE 7.9: Compressive strength and air content

batch no.	strength at 28 days (MPa)	hardened air content (%)	air content when plastic (%)
7	73-77	3.61	6.0
10	59	6.10	7.0
variance:	-21.3%	+2.49%	

7.7.2 Elasticity

Tests were carried out in accordance with CSA A23.2. The results of the preliminary and advanced tests indicate a strong correlation between strength and elasticity. Measured values for elasticity are compared with the design value for high strength concrete proposed by Collins (1993):

$$Ec = 3320\ (fc^{0.5}) + 6900,$$ with MPa units throughout.

Measured values for elasticity were consistently within 10% of the design value.

TABLE 7.10: Elastic modulus and compressive strength

batch no.		1	2	3	4	5	6	7
measured f'c	(MPa)	74.6	65.5	54.0	57.5	53.1	44.7	73.2
measured Ec	(GPa)	33.2	31.1	29.8	31.1	29.9	28.0	33.8
design Ec	(GPa)	35.6	33.8	31.3	32.1	31.1	29.1	35.3
variance	(%)	7.2	8.8	5.0	3.2	4.0	3.9	4.4

7.7.3 Creep

Tests were carried out in accordance with ASTM C512. A plot of results is included in Appendix C: a value of 52.6 10^{-6}/MPa was recorded after 170 days, which is consistent with data from other high-performance concrete testing programs.

Creep results reported in the literature are consistently less than values for normal-strength concrete, resulting in less long-term deflection of structures under load.

7.7.4 Flexural fatigue endurance

Flexural endurance tests were carried out to determine the concrete's ability to absorb energy under fatigue loadings. Tests were carried out on 100x100x350 mm samples: specimens were loaded to failure in bending to determine the concrete's flexural strength, then similar specimens were loaded two million times at a various predetermined percentages of the concrete's flexural strength.

Samples from Mixture #7 were found to have a similar flexural fatigue endurance limit to that for normal-strength concrete (50% of flexural strength), indicating that the concrete mixture is no more susceptible to brittle failure than normal-strength concrete [Balaguru & Shah].

7.7.5 Air Content

Tests were carried out in accordance with CSA A23.2-4C for plastic concrete and ASTM C457 for hardened concrete. Mixtures #1 to #7 used the same air-entraining agent. All experienced a loss in entrained air of 2-2.5% from the specified target while plastic. Resulting values for hardened air content ranged from 3.6 to 5.4% and spacing factors ranged from 0.20 to 0.31. A target average value for spacing of 0.23 mm is recommended for exposed concrete surfaces.

An alternate air entraining admixture was used for Mixture #10: the impact on results is notable - less air loss during placement, setting and curing; smaller spacing and greater specific surface; acceptable results using an order-of-magnitude less of addative. It is believed that field results may vary as widely as those in the laboratory with changes in equipment, placing methods, temperatures, etc. The difficulties in stabilizing air content with the first agent contradicts experience elsewhere in the construction of high-performance concrete products, and may be due to the very high mixing rate of the laboratory mixer (155 L Eirich).

Acceptable field results were considered to be achievable with the design mixture. Also, an early review of hardened air characteristics is essential for acceptance of construction methods using HPC.

7.7.6 Salt scaling

Tests were carried out in accordance with ASTM C672-91a.

Mixtures #7 and #10 demonstrate the strong correlation between air-void characteristics and resistance to freezing and thawing. In both cases, sample surfaces were submerged in 4% salt solution and exposed to freezing and thawing conditions daily for over fifty cycles. The quantity of concrete released from the surface was measured every five cycles (Appendix C), and totalled 0.7 kg/m3 in the case of Mixture #7 and 0.15 kg/m3 for Mixture #10. (0.8 kg/m3 is considered an upper limit for contract compliance in Ontario.)

Again, the results indicate that a acceptably low level of scaling may be achieved with the concrete mixture in the prototype bridge provided that the entrained air characteristics are controlled.

7.7.7 Rapid chloride permeability and diffusivity

The permeability and diffusivity tests are carried out as indicators of the rate of penetration of chloride ions through the concrete matrix. Corrosion of reinforcement is considered to occur when chloride ions concentrate in solution at the surface of the reinforcement.

Rapid chloride permeability tests were carried out in accordance with ASTM C1202. With rapid chloride permeability testing an electrolytic cell is established with a concrete sample forming a barrier between sides of the cell (Fig. 7.5). Conductivity is a function of chloride permeability through the sample.

The diffusion of ions through the matrix to the reinforcement can be predicted using Fick's Second Law of Diffusion, using a measured rate of diffusion [eg. Dunaszegi, 1996]. The diffusion is measured by soaking the sample in a salt solution for a known period, then shaving 1 mm layers off the surface (Fig. 7.11) and measuring the chloride content by titration with silver nitrate. The profile of salt concentration with depth is then used to determine the constant in Fick's Law. Results from this procedure are complicated by an initial surface absorbtion process which, in turn, is highly dependent on initial moisture content. However, these absorbtion characteristics have also been used as a means of determining permeability, from which diffusion rates may be estimated [Hooton, 1997].

Both the Rapid Chloride and the Diffusivity methods have limitations, both in the application of theory and in practice. A new technique is currently under development to replace the rapid chloride method in the United States, but the rapid chloride method is most commonly used at present as a simple, practical test with reasonable correlation to chloride ion diffusion. Rapid chloride permeability values have been specified for high-performance concrete bridges in the United States, with different values specified for the deck, the remaining superstructure and the foundations.

An objective of less than 600 Coulomb conductivity after 91 days was established for concrete testing, based on experience with conductivity and diffusivity tests by Jacques, Whitford; this objective was met in laboraty testing (Tables 7.4 and 7.7).

7.7.8 Slump

Tests were carried out in accordance with CSA A23.2-5C. The laboratory testing demonstrated that a well-blended, cohesive mixture could be produced having a slump as low as 20 mm. Also, the cohesive quality of the concrete was maintained for slumps as high as 225 mm after the addition of superplasticizer. The final slump could be regulated by controlling the amount of superplasticizer.

7.7.9 Adiabatic heat development

Tests were carried out in accordance with ASTM standards. Results for Mixture #7 are plotted in Appendix C.

The results are consistent with previous data for high-performance concretes:

- there is no acceleration in initial setting time,
- setting time is readily controlled by retarding admixtures, and
- the maximum heat development is consistent with the quantity of cement and silica fume used.

8. CONSTRUCTION CONTRACT SPECIFICATION

The specifications for concrete materials and placement were developed to match the structural design and construction priorities for the bridge. The tested concrete mixture described above was offered for use by contractors, and the following performance specification was also offered to contractors. The performance specification was based on criteria that had been demonstrated to be practical by the testing program. The full specification of HPC materials, construction, quality control and crack repair is included as Appendix D.

Cementing materials:	minimum	450 kg/m^3
Water/cementitious ratio:	maximum	0.34
Coarse aggregate size:	nominal	20 mm
Slump before superplasticizer:	maximum	60 mm
Slump after superplasticizer:	+/- 30 mm	190 mm
Air content at truck discharge:	+/- 1%	7%
Compressive strength at 28 days:	minimum	60 MPa
Hardened air void spacing factor:	maximum	260 mm
	max. average	230 mm
Rapid chloride permeability (91d):	maximum	600 Coulombs
Delivered concrete temperature:	maximum	25 deg. C
Concrete temperature in situ:	maximum	70 deg. C
Temperature gradient:	maximum	20 deg. C/m

The construction tender included the following features that were specific to this prototype bridge project:

- a half-day workshop for bidders and others,
- a specified concrete mixture design with an alternative performance specification,
- concrete delivery at 3/4 of the ready-mix trucks' capacity to improve mixing,
- a trial slab to prove placement, finishing and curing methods and air voids,
- use of an 'evaporation reducer' between screeding and floating,
- special curing requirements,
- a crack survey and repair after curing to mitigate the reported probability of transverse shrinkage cracking, and
- extra testing technicians on site to facilitate adjustments to the superplasticizing of the deliviered concrete and to take additional tests of slump and air content after pumping.

9. CONSTRUCTION QUALITY

The high-performance concrete mixture was placed in all concrete work for the bridge. Both skips and pumping techniques were used in the footings, abutments and columns. Results from the routine testing of pumped concrete for the foundations was used to verify the mixture design before the critical deck placement without a separate pump test.

Site visits were made by DalTech representatives during initial on-site placement of concrete, before backfilling around the initial concrete work, on completion of foundations and columns, after placement of precast girders and diaphragms, during placement of the trial slab and during placement of the concrete deck. During these visits it was possible to compare the rheology of the commercially batched concrete with that produced in the laboratory and to compare the finished product with predicted results (finish, segregation, cracking etc.). Quality control for concrete materials was carried out by Jacques Whitford and Associates.

9.1 Footings, piers and abutments

The first batch of concrete as delivered to the site on August 28th, 1997. Slump values were 20 mm before the addition of superplasticizer and 220 mm after addition. Two testing technicians who were kept busy with air and slump measurements as the superplasticizer was added. Concrete for the footings was placed by skip. The high slump was appreciated by the contractor because of tight reinforcement details at the column splice (Fig. 9.1).

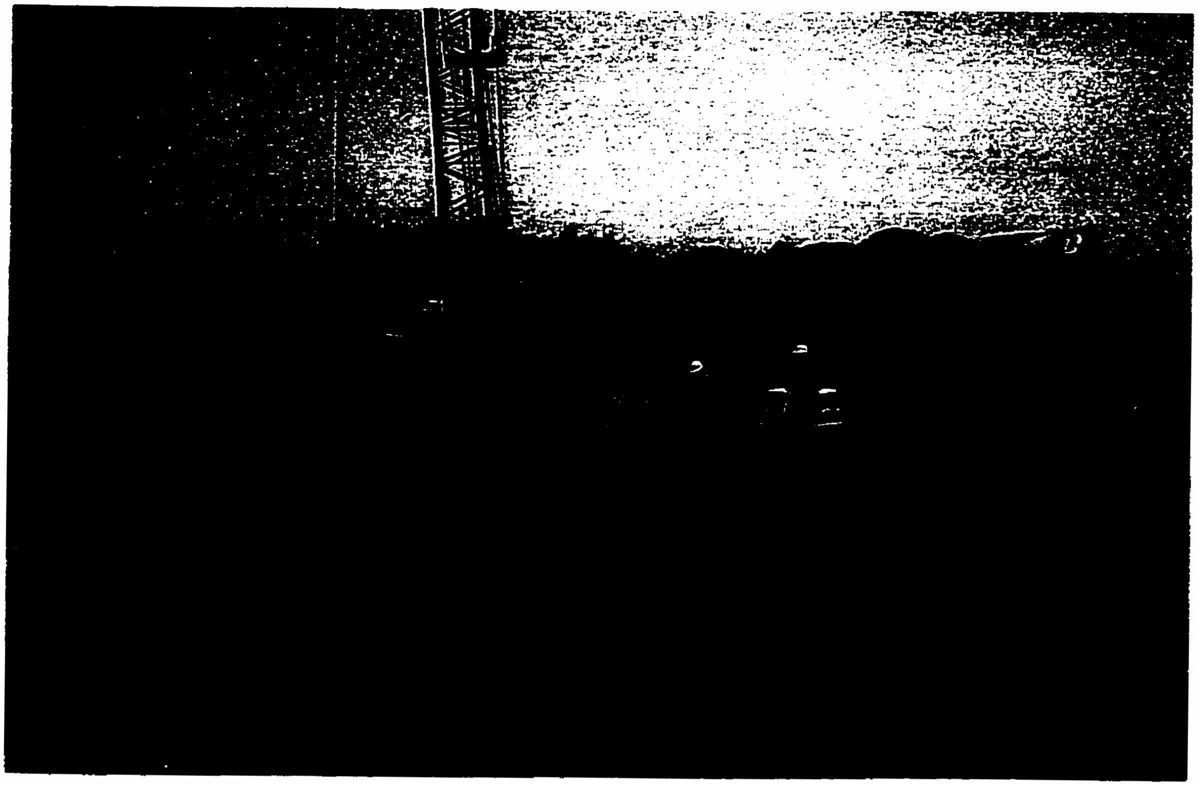


FIGURE 9.1: Concrete placement for footings, August 28th, 1997

Severe plastic shrinkage cracking occured in the footings: procedures for normal-strength concrete had been used by the contractor which rely on free water from the concrete mixture to prevent surface drying; this water is not available in the HPC mixture and external methods are required to limit cracking (wetting, fogging, evaporation inhibitor, sealants and/or cover). No further plastic shrinkage cracking occured in the piers and abutments, and the only imperfection in the finished concrete occured where bleeding of the superplasticized mixture left a surface that adhered to the forms when stripping, or could be removed by light chipping.

Concrete strength and plastic air content were found to vary significantly during the initial phase of the project. Test data are included in Appendix E. It was found that there was a loss of entrained air of 1-2% of total concrete volume due to pumping, and that the air content measured by examining the hardened concrete was in the order of 1% less than that of the plastic concrete as placed.

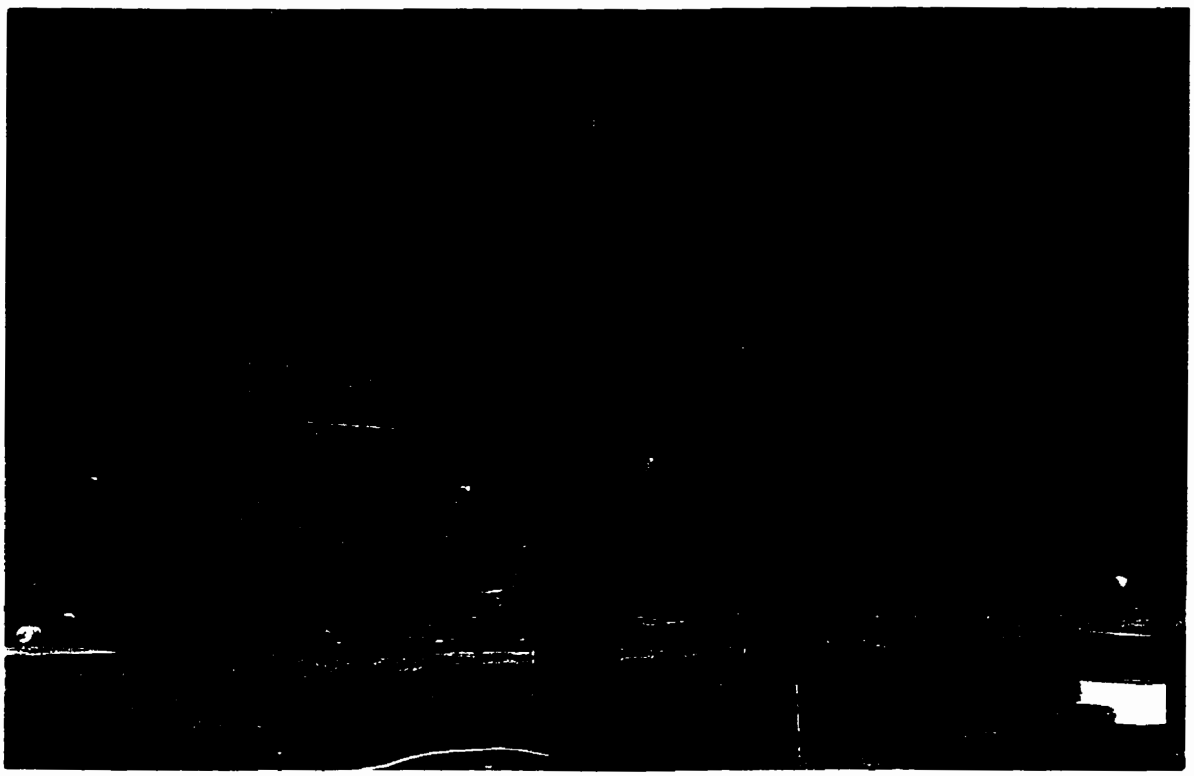


FIGURE 9.2: Centre bridge piers and beam. Sept. 19, 1997

9.2 Precast girders

Girders were cast in an indoor casting bed in two series of three girders using a single mold. Time to release varied between three and five days. The girders were delivered in 36 m lengths to span between supports during construction; abutting girders were connected by grouting reinforcement dowels and sealing with a concrete diagphram (Fig. 9.3).

FIGURE 9.3: Precast girder connection at centre pier, Oct. 30, 1997

9.3 Trial deck slab

A trial slab was placed to demonstrate the materials, equipment and workmanship to be used for the exposed concrete deck. The slab was approximately 9 m by 10 m in size and was placed using a Bidwell mobile screed (Fig. 9.4) with a trailing platform to facilitate tyning, sealing and covering the finished surface (Fig. 9.5).

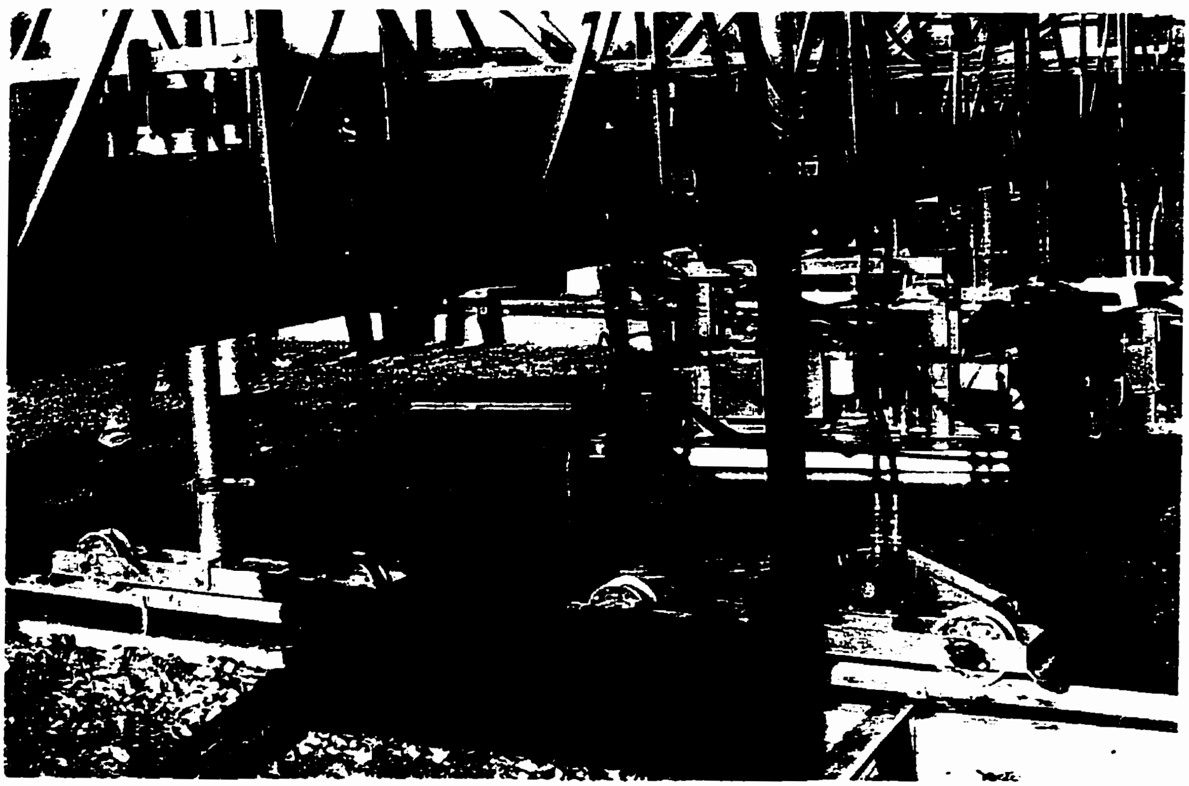


FIGURE 9.4: Trial slab - Bidwell mobile screed, Oct. 30, 1997

FIGURE 9.5: Trial slab - trailing platform; applying sealant; Oct. 30, 1997

Pumping methods used for the trial slab were similar to these used for the deck (Fig. 9.6). Air content, slump and strength testing was carried out on the concrete. It was found that the surface texture applied by the tyning rake was successful when the slump was in the range of 150-220 mm only (Fig. 9.7). Also, there was a loss of entrained air of 0.5-1.5% due to pumping.

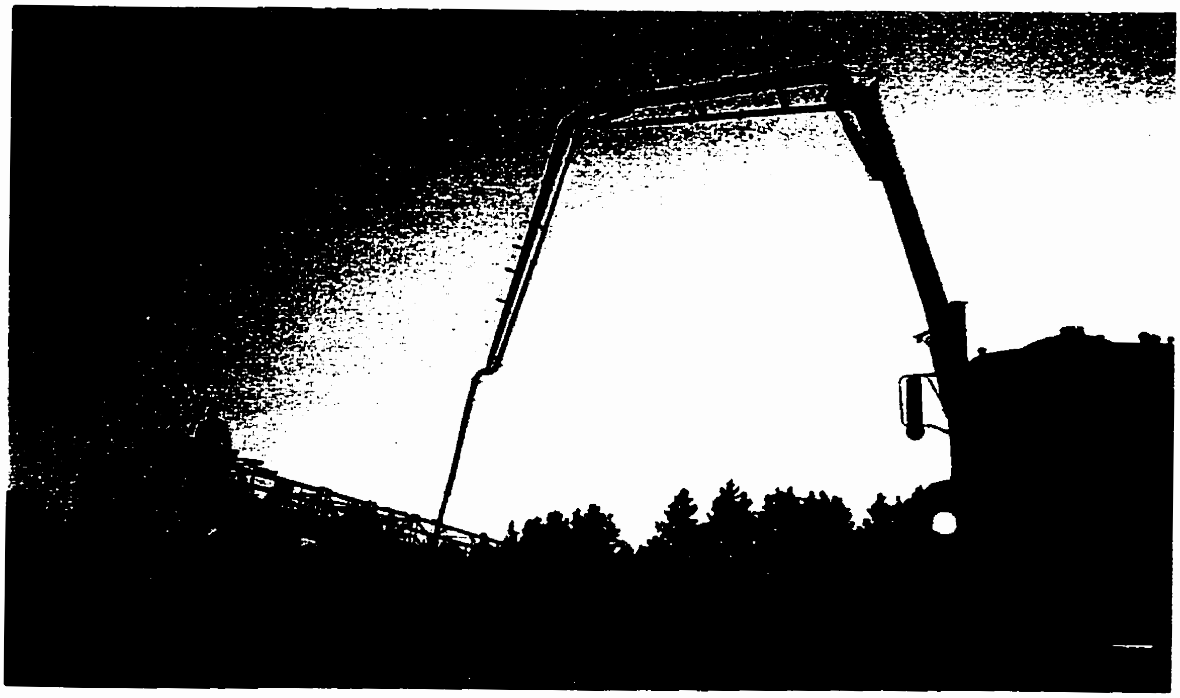


FIGURE 9.6: Trial slab - pumping arrangement; Oct. 30, 1997

FIGURE 9.7: Trial slab - hardened concrete surface, Nov. 24, 1997

9.4 Bridge deck slab

The bridge deck was placed in a single pour on Nov. 24th, 1997, using the ready-mix batching, pumping, screeding, finishing, curing and testing procedures described previously (Fig. 9.8 - 9.11). The deck was also fitted with conductivity meters for corrosion measurement (Fig. 9.12). Cold, damp weather reduced the risk of plastic shrinkage cracking and transverse shrinkage cracking: the bridge was found to be generally free of cracks when inspected in mid-December.

During placement some concrete was accepted with too high a slump, resulting in some segregation in a heavily reinforced area and poor finishing. A recommendation for future projects would be further reduction in the slump as delivered, either through reduction in water content or reduction in water-reducing agent, and stricter control of the slump after applying the superplasticizer to ensure a slump of 150-200 mm at the pump nozzle.

FIGURE 9.8: Bridge deck slab - testing and superplasticizer addition

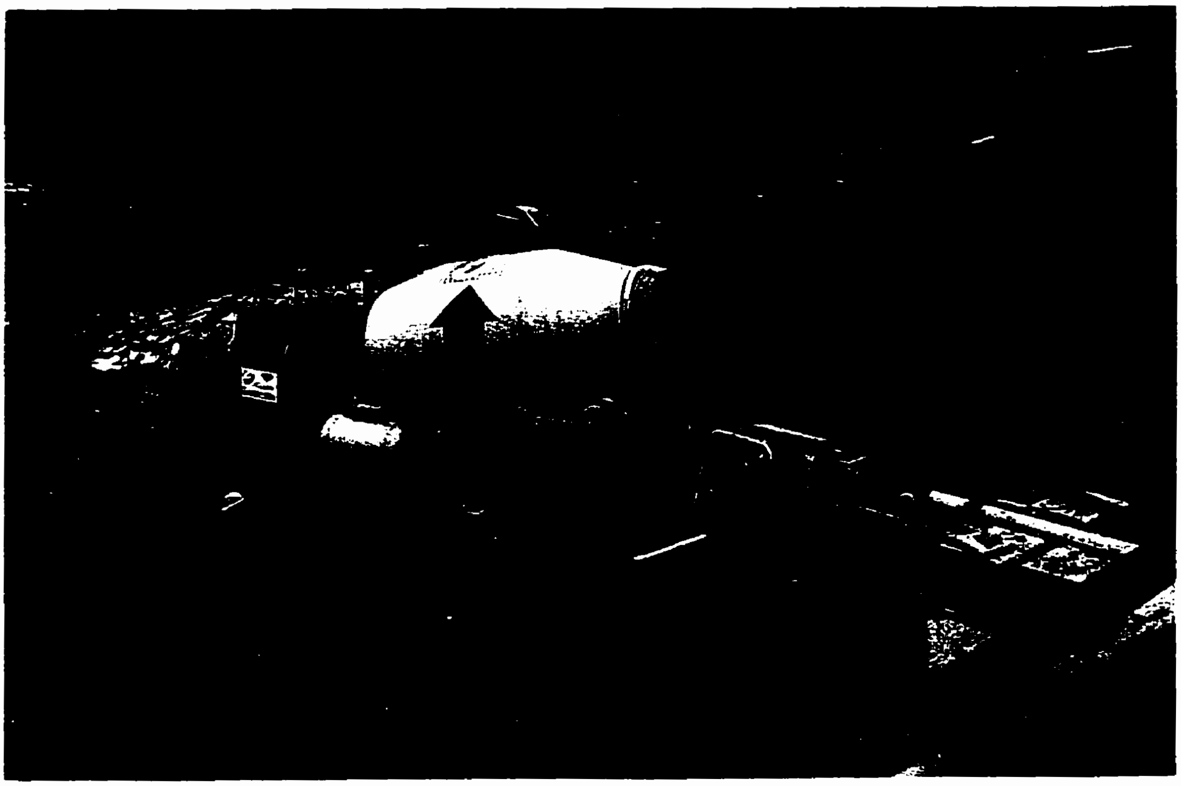


FIGURE 9.9: Bridge deck slab - concrete discharge into pump

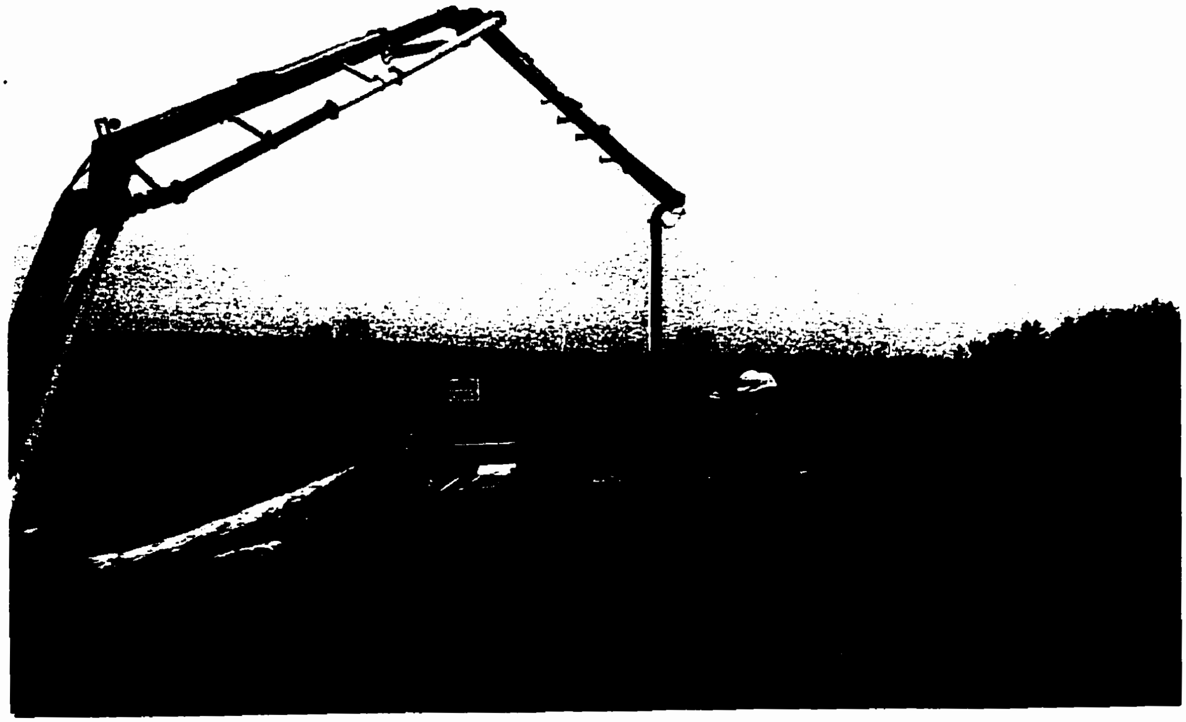


FIGURE 9.10: Bridge deck slab - pump discharge and deck screeding arrangement

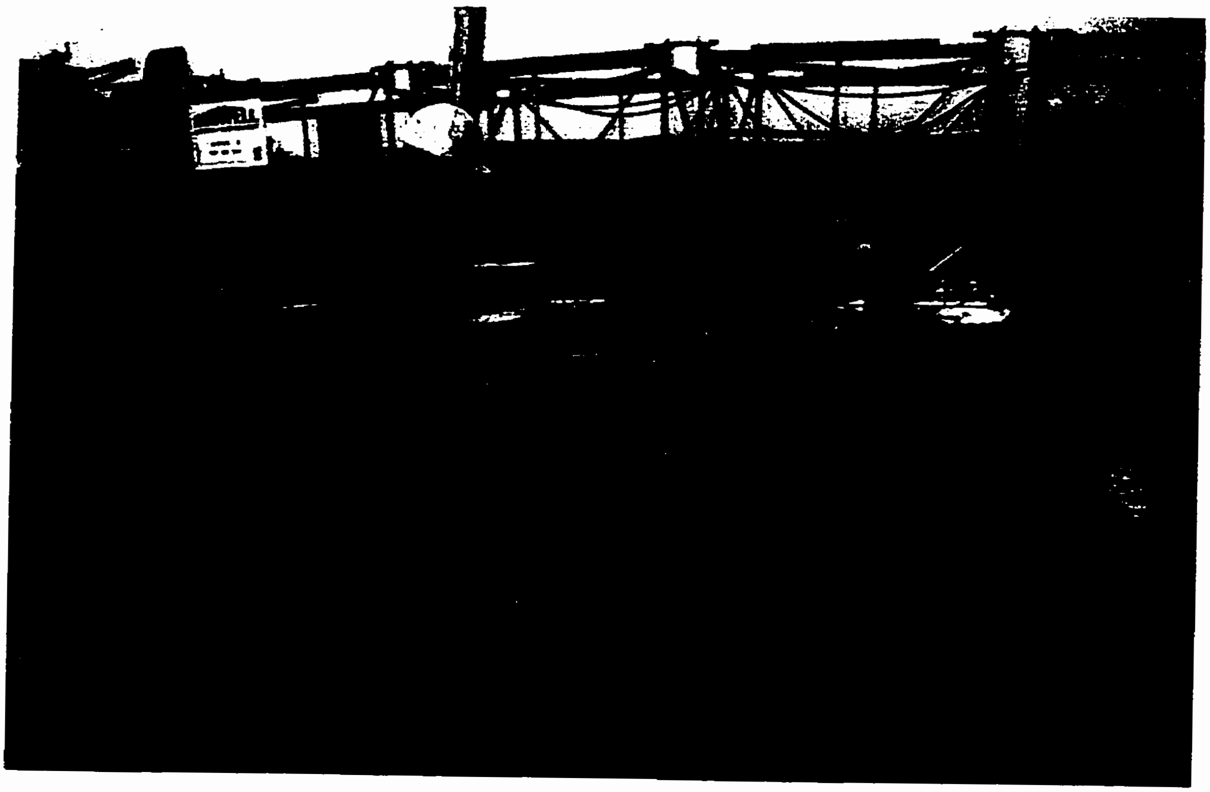


FIGURE 9.11: Bridge deck slab - deck screeding (excessive slump)

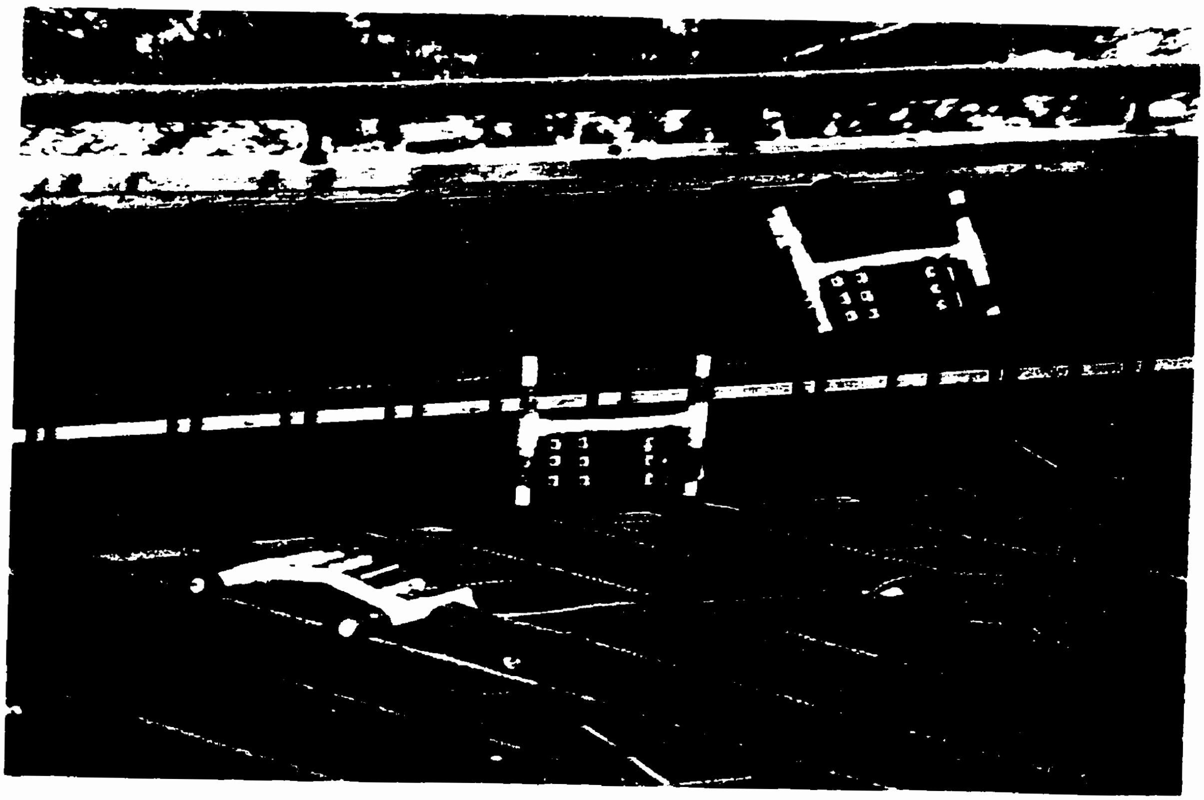


FIGURE 9.12: Bridge deck slab - conductivity meters

10. ECONOMIC EVALUATION

10.1 Preliminary assessment

As described previously, two HPC options were compared with a reference "normal-strength concrete" design concept:

TABLE 10.1: Summary of normal-strength and HPC design concepts

	NSC	HPC-A	HPC-B
GIRDER DESIGN:			
no. of girders	4	3	3
prestressed tendons	48 - 13 mm	-	42 - 16 mm
release stress	30 MPa	-	54 MPa
28-day strength	35 MPa	-	65 MPa
DECK DESIGN:			
asphalt cover?	yes	yes	no
main reinforcement	#15 @ 300	#20 @ 250	#15 @ 300
	#20 @ 300	#20 @ 250	#20 @ 300
epoxy coated?	yes	no	no
concrete cover	50 mm	50 mm	65 mm
conc. thickness	200 mm	200 mm	215 mm
28-day strength	35 MPa	60 MPa	60 MPa

Construction cost estimates were developed for the three options using actual construction prices from previous NSDoT projects. Only the items that vary from option to option were included in the analysis; excavations, embankment work etc. were not included.

TABLE 10.2: Estimated capital costs of design options

item	NSC	HPC-A	HPC-B
footings	$31,500	$37,800	$37,800
abutments, piers, beams, abut.	27,217	31,300	31,300
girders	298,200	264,759	264,759
girder diaphrams	7,200	8,280	8,280
deck concrete, haunches	52,102	60,438	60,438
deck reinforcing	24,330	20,198	16,220
parapet walls	22,624	26,018	26,018
waterproofing	12,514	12,514	0
80 mm asphalt topping	9,010	9,010	0
TOTAL:	**$484,697**	**$470,317**	**$444,815**

Note that the girders are a large part of the variable cost. Also, the initial cost of reinforcing, waterproofing and asphalt are not highly significant. Epoxy-coating is not a major cost item either, as can demonstrated by comparing the NSC and HPC-A designs.

Maintenance requirements were estimated for all the options (appendix F). The present-values of all the estimated construction and maintenance work was then estimated for each concept (Table 10.3).

The equation for net present value for a government agency does not include taxes, and is simply:

$$PV = PC \cdot (1+e)^{n} \cdot (1+d)^{-n}$$

(- the present cost PC is escalated by rate 'e' for 'n' years to an estimated cash amount at the time that the work is to be done; this future cash amount is brought back to the present value PV using the discount rate 'd' over 'n' years)

The escallation rate is a reflection of the rate of price increase with time while the discount rate is determined by the rate at which money is borrowed to carry out the work. For example, in the case of the exposed concrete deck, Option B it is hoped that the only repair will be the replacement of expansion joints after 35 years and only $10,000 would have to be invested today to pay for the work in 35 years, even though it would cost about $50,000 to carry out the work if it were done today.

TABLE 10.3: Estimated life-cycle costs of design options

item	year	NSC $ (PV)	HPC-A $ (PV)	HPC-B $ (PV)
construction:	0	$484,697	$470,317	$444,815
replace asphalt:	12 24	$20,426 $11,502	$20,426 $11,502	$0 $0
rehab. / repairs:	35	$55,007	$15,630	$9,772
replace asphalt:	47 59 71	$3,826 $2,155 $1,213	$3,826 $2,155 $1,213	$0 $0 $0
total, net present value:	0	**$578,827**	**$525,070**	**$454,587**

Various conclusions can be drawn including that, if the construction estimates are reasonably accurate and the scope of maintenance is correct then life-cycle costs for the high-performance options are consistently lower than for normal-strength concrete.

A sensitivity analysis was also carried out (Appendix F) to determine which variables may be critical to the precision of the "bottom line" present values. The analysis was carried out for a range of discount rates, escalation rates and maintenance schedules as these were believed to be less predictable than actual present costs. Each variable was examined independently with the base case value used for the other variables. Variation ranges included 5-9% for discount rate, 1-3% for escalation and acceleration or delay of maintenance by 50%. From these data the largest departures from the tabulated savings of $124,000 for HPC Option [b], relative to the normal strength concrete design, were an increase in savings of 54% using a discount rate of 5%, and a decrease in savings of 32% using a delay of 50% for all maintenance work.

10.2 Contract prices

Actual prices were received as part of a multi-million dollar contract for highway development. Prices included unit rates for supply and installation of precast concrete, cast-in-place concrete, reinforcement and various items related to earthwork and highway construction.

It is usual for tendered prices for individual unit rate items to fluctuate significantly from tender to tender and, where the total amount tendered is much greater than the items of interest, it may be very misleading to attempt to use the tender data to infer any degree of precision in the preliminary estimates. Also, the confidentiality of bids between bidder and NSDoT is essential to contractors.

However, it may be concluded that:

- the degree of variation among bidders in the price of HPC items was consistent with other tenders for normal-strength concrete products,
- the variation in total price for the bridge structure was consistent with other tenders,
- the variation in the price of HPC items between the bids and the initial estimate (Appendix F) was consistent with normal-strength concrete tenders, and
- the total price for the bridge structure was marginally less than the estimate included in Appendix F.

11. SUMMARY AND CONCLUSIONS

Various points may be made in summary:

A prototype bridge was constructed in 1997 using high-performance concrete. The bridge is in two spans with a total length of 70.7 m and width of 8.85 m.

The design objective was that HPC elements are not to require repair during the structure's design life of 80 years.

The multi-disciplined approach used in the project's development was essential to the success of the project.

Two approaches were considered in the design of the deck for the HPC bridge:
a] use traditional components and compare the durability of the HPC deck to a 'normal strength concrete' (NSC) deck, and
b] maximize the exposure of the HPC deck through elimination of the waterproofing membrane and asphalt wearing surface.

The exposed concrete deck option was built because of its economic potential and because the exposed deck can be more easily assessed for deterioration.

A single mixture design was developed for the precast bridge girders and the cast-in-place concrete elements. Two phases of testing were carried out before specifying the mixture:

1) preliminary tests with six mixture designs in which the water/cementitious ratio and fly ash content were varied, and
2) advanced testing on a selected mixture, including salt scaling, flexural fatigue endurance and creep tests.

The testing program succeeded in identifying a concrete mixture that was suitable for use in high-performance concrete construction, and confirmed appropriate performance criteria for acceptance of other concrete mixture designs.

It was demonstrated that, with increased control, an order-of-magnitude increase in design concrete strength is practical and, in some cases, economical for both cast-in-place and precast concrete.

It was also confirmed that the permeability of concrete can be reduced by several orders of magnitude without changing the entrained air quality of the concrete. The resulting improvement in concrete durability may enable concrete work to be exposed to harsher environments, including exposed concrete highway bridge decks.

Further use of exposed concrete decks is recommended on the basis of:

1] the initial economy and life-cycle cost reduction of the prototype structure, and
2] the ability to repair, if necessary, by microsurfacing or milling.

The scaling of exposed decks may be an issue and can be reviewed visually at the West River East Side Bridge.

For production specifications, the concrete mixture selected for this bridge structure is reproducible and will be economic for similar prestress levels and severity of exposure. Higher strengths may be appropriate for more slender structures, but attempts to further reduce the permeability of the mixture may increase the variability of the mixture unless stringent quality control is provided at the site during placement.

The use of epoxy-coated reinforcement is recommended for the deck structure because of the additional protection provided at a relatively nominal cost.

The West River East Side Road Bridge may serve as a demonstration project for technological advance and reduction in the price of bridge structures in Nova Scotia. All the development objectives were met and the structure is now exposed to the test of time.

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APPENDICES

APPENDIX A

PROTOTYPE BRIDGE PROJECT SCHEDULES

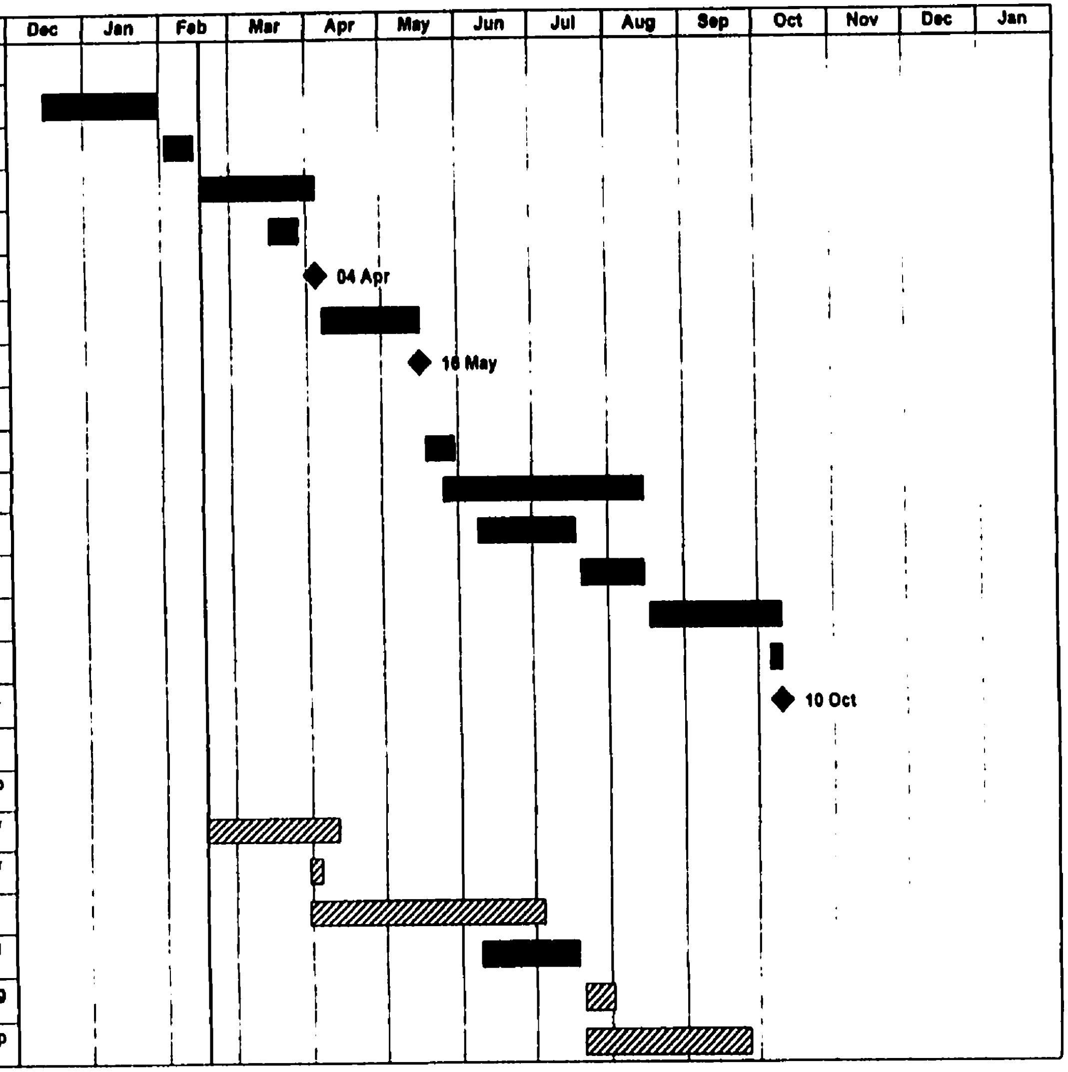
CONTENTS:

a] proposed schedule, Feb. 17, 1997

b] actual schedule

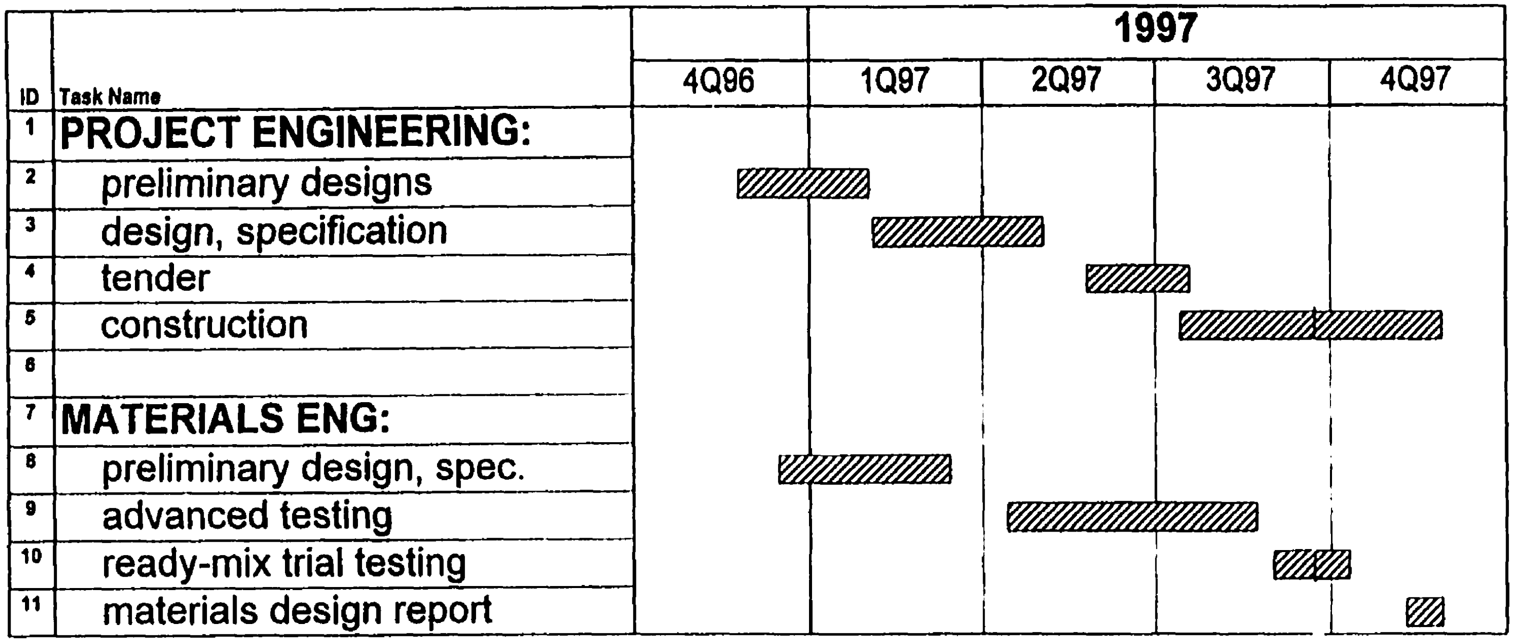
NSDoT&PW: HIGH PERFORMANCE BRIDGE PROJECT - SCHEDULE (1997)

ID	Task Name	Dur.	Start	Finish
1	PROJECT ENGINEERING:	43w	16 Dec	10 Oct
2	preliminary designs	7w	16 Dec	31 Jan
3	prelim. design approval	2w	03 Feb	14 Feb
4	design, specification	7w	17 Feb	04 Apr
5	design check	2w	17 Mar	28 Mar
6	DESIGN COMPLETE	0w	04 Apr	04 Apr
7	tender	6w	07 Apr	16 May
8	TENDER AWARD	0w	16 May	16 May
9	construction:	21w	19 May	10 Oct
10	mobilization	2w	19 May	30 May
11	abutments, piers	12w	26 May	15 Aug
12	ready-mix trial testing	6w	09 Jun	18 Jul
13	precasting	4w	21 Jul	15 Aug
14	deck construction	8w	18 Aug	10 Oct
15	touch-ups	1w	06 Oct	10 Oct
16	BRIDGE COMPLETED	0w	10 Oct	10 Oct
17				
18	MATERIALS ENG:	32w	17 Feb	26 Sep
19	preliminary design	8w	17 Feb	11 Apr
20	specification	1w	31 Mar	04 Apr
21	advanced testing	14w	31 Mar	04 Jul
22	ready-mix trial testing	6w	09 Jun	18 Jul
23	materials design report	2w	21 Jul	01 Aug
24	construction tests, inspect.	10w	21 Jul	26 Sep



J Fletcher, 17 Feb., 1997

NSDoT&PW: HIGH PERFORMANCE BRIDGE PROJECT - SCHEDULE



APPENDIX B

A REVIEW OF TECHNOLOGY RELATED TO THE USE OF FLY ASH IN CONCRETE

- adapted from a report forming part of
studies in cement and concrete chemistry,
directed by J F Trottier, TUNS. April 1995.

James Fletcher, Dec. 1997

ABSTRACT

The manufacture and chemistry of Portland cement, pozzolans and concrete are described as the basis for understanding the design of high-performance concretes.

CONTENTS:

TABLES:

FIGURES:

INTRODUCTION

Various texts on cement chemistry and technology (1,2,3) introduce the subject with a description of the architectural and engineering achievements of the Egyptian, Greek and Roman eras, where lime, then a mixture of lime and, from the Bay of Naples, crushed volcanic tuff from Pozzoli, were used to create cementitious materials. A striking example of the technology of the times is the Roman Pantheon which includes a domed roof made of lightweight concrete from pumice, Pozzolanic fines and lime. The structure spans 43 m and has survived almost two millennia.

The technology of pozzolanic action disappeared with the Roman Empire and the cement technology that followed was limited to a craft of burning lime (dehydrating and calcifying chalk and other materials rich in calcium carbonate) in kilns for use as an active ingredient in mortar, and the use of partially hydrated calcium sulphate (gypsum) as plasters. These materials are, however, soluble in water.

In the nineteenth century, hotter kilns and specific combinations of chalks and clays were found to create products that were stronger and more durable in the presence of water. In 1824 Joseph Aspdin patented the process of burning lime and clay to produce a clinker which could be ground and hydrated to form a hard mortar looking much like rock formations found on the Portland Isles in the English Channel: Portland Stone.

The technology of Portland cement concrete has developed largely through empirical investigations, assisted by a gradual increase in understanding of some of the chemistry in action. Major advances have included the development of continuous processes to manufacture cement more economically and the use of embedded steel as reinforcement, then by the use of admixtures to entrain air for improved durability and de-flocculant admixtures to improve placement "workability". Most recently, significant improvements in the performance of concrete have been realized through research that has focussed on applications and through advances in related sciences and technology.

While most of the research and development of materials in the last century has related to Portland cement, the pozzolanic reaction reported by Vitruvius (4) has again become relevant through the use of various industrial by-products in Portland cement concrete. Fly ash, ground or granulated blast-furnace slag and non-ferrous slags, rice-husk ash and, more recently, silica fume are considered to have cementing qualities when used in conjunction with Portland cement (5). The potential for use of these by-products is driven by the economics of:

a] the quality and price of the concrete product and

b] the avoided costs of disposing of the by-product.

Where locally available, fly ash has proved to be economic as a replacement for a major portion of the Portland cement in concrete, as an additive to achieve certain performance criteria, and also as "flowable fill" (6, 7). Other industrial by-products which are used as aggregates or fillers include pelletized fly ash (8), ash cinders, slags and crushed building debris.

This review summarizes reference material on the chemistry of, and applications for, cementitious materials (ordinary Portland cement, fly ash and other pozzolans).

PORTLAND CEMENT BASICS

The essential ingredients of Portland cement concrete include cementitious powder, water and aggregates. Air may become entrapped during mixing and, by using admixtures, air may be entrained as bubbles in the order of 0.05-1.25 mm in size. The mixture sets by creating a solid gel and interlocking crystal formations out of the cementitious material and water; a process of hydrolysis and hydration.

As the concrete sets, its volume remains approximately constant, about 3% greater than the original volume of the aggregates. However, the hydration process reduces the volume of the cement and water that hydrates by 18.5% of the cement volume (2).

Water in the mix becomes:

a] combined, through reaction with cementitious particles. A mass of water equal to about 23% of the cement that hydrates is used up in this way;
b] gel water, adsorbed or physically held in pores in the hydrating cement gel. These pores are about 0.002 um in diameter. The water used this way amounts to about 19% of the mass of cement that hydrates; and
c] capillary water (whatever is not fixed in the gel).

Capillary pores form in the gel. It is inferred from discussions in the referenced texts that these capillaries enable the migration of mix water, curing water and/or air as the hydration of cement particles continues after the initial "set" of the gel. The water in the capillary pores may react with the cement or be adsorbed into the cement gel and, as the gel volume increases, the size of the capillary pores is reduced. Alternatively, the capillary water may eventually be removed by drying of the concrete mass, or it may be recharged or contaminated by an external source, eg. sea-water immersion. The extent to which the capillary pores are interconnected is the main determinant of the concrete's permeability.

Neville (2) describes unreinforced, normal-strength concrete as a two-phase medium of hydrated paste and aggregate, with an interface between the phases that has a major affect on compressive strength. A more complete model would include capillary pores as probably the most critical component to the mechanical behaviour of traditional concrete mixtures: strength and durability are largely functions of the concrete's density and impermeability (2).

A relatively low-strength concrete is used for an example (2): Fig. 1 describes the volumetric proportions during hydration of the concrete. The proportions are described at 0% hydration (at time of mixing), 70% hydration (set) and 100% hydration (a theoretical limit). The mix proportion is 1:2:4 (cement: fine aggregate: coarse aggregate), w/c ratio is 0.55, 2.3% entrapped air is present and the concrete is cured in moist conditions. Points to note include:

a] Porosity is the total volume of water and air in the gel, the capillaries and voids (in this case 15.8% at 70% hydration); permeability is a measure of hydraulic conductivity, primarily through the capillaries.

b] Capillary voids are reduced considerably during the final stages of hydration,

c] for this mixture, capillaries make up 6.0% of the total volume at 100% theoretical hydration.

d] As hydration takes place, the reduction in volume of cement and mix water causes the sample to imbibe 1.9% of its volume in curing water. The absence of curing water would have resulted in voids being filled with air.

e] About a quarter of the original mix water is surplus to the hydration process in this example: it is shown as capillary water. Reducing the water/cement ratio of this mixture to 42% (if practical) would result in all the mix water being used to fully hydrate the cement, and would result in a capillary volume of about 2%.

f] Theoretically, the capillary volume would be eliminated by using a water/cement ratio of 0.36 and by providing water during curing to complete the hydration of the cement particles. The use of any lower water/cement ratio would result in incomplete hydration because of lack of space to complete the hydration process.

g] Water must be provided during curing for the capillaries to be totally filled. However, the ability of water to migrate into the concrete becomes limited as the capillaries fill and permeability is reduced. Some residual capillary pore volume is inevitable.

ENGINEERING PROPERTIES OF PORTLAND CEMENT CONCRETE

The success factors for concrete in the construction industry include (3) convenience, economy, adaptability, strength, and durability.

The raw materials have to be commonly available and economically manufactured; practical, reproducible methods are required for mixing, placing and curing; dependable levels of strength and durability are needed. These factors are interrelated and frequently an optimum result is a balance between conflicting objectives.

The most important qualities of cast-in-place structural concrete products are dimensional stability (related to shrinkage, elastic stiffness and creep), compressive strength and durability. The reliability of all these qualities is also of major importance.

a] Shrinkage creates deformations and stresses in finished concrete that are significant from a design and performance viewpoint. Shrinkage includes:

- plastic shrinkage during setting of the plastic mix (1000×10^{-6} to 2500×10^{-6}, increasing with increased cement content),
- autogenous shrinkage due to hydration(50×10^{-6} to 100×10^{-6}),
- drying shrinkage: $0\text{-}800 \times 10^{-6}$, due to drying out of capillary pores. Drying shrinkage includes a reversible portion in the order of 40-70% of the total drying shrinkage,
- carbonation shrinkage: $0\text{-}800 \times 10^{-6}$, due to reaction between carbon dioxide gas and the hydrated cement (2), and
- wetting expansion: the reversible portion of drying shrinkage.

b] Initial deflection under load is approximately linear at low levels of loading but increases non-linearly with the size and duration of loading. A secant modulus is used to represent initial deflection under sustained design stresses (typically 20-30 GPa for traditional concretes), resulting in strains in the order of 500×10^{-6} at 50% of f_c'.

The secant modulus of concrete is greater than that for cement paste and less than that for aggregates. Both the deformation under load and the Poison Ratio of concrete increase dramatically with loading to greater than $0.3f_c'$ because of the growth of bond cracks at the cement/aggregate interface.

c] Creep, the increase in strain under sustained constant stress, is typically 1-3 times greater than the initial deformation under load. Creep is a long-term process: characteristically, it takes 14 days for the first 25%, 3 months for 50% and a year for 75% of the full long-term creep to develop (2).

The extent of creep deformation depends on the degree of hydration before and after application of the load: the long-term creep of concrete loaded at 28 days after mixing may be in the order of double that for concrete loaded one year after mixing (2). The ambient humidity and shape of specimen (shape influencing capillary pore vapour pressure) are also significant, with long-term creep in submerged conditions (100% relative humidity) characteristically one-third that for 50% humidity (2). Creep is also dependent on the magnitude of load, with larger creep values experienced when the stress exceeds 50% of f_c'. Removal of loading results in some instantaneous recovery, some creep recovery and some residual deformation.

Long-term deflection is frequently a limiting criteria in the design of beams and slabs. It also influences the buckling behaviour of eccentrically loaded columns and causes loss of prestress in prestressed reinforcement.

d] The compressive strength of concrete is usually less than that of the paste or the aggregate. Where laterally unrestrained, strains and fractures along discontinuities in the matrix cause lateral tensile stresses which ultimately cause bursting of the concrete section. Lateral restraint of the concrete results in much higher compressive load resistance, with failure due to crushing.

The compressive strength of concrete is the major determinant of the strength of columns, arches and other elements subjected to compression, and has a lesser influence on the strength of members subjected to bending.

e] Durability may be the most significant quality of all concrete components. It is the resistance of concrete components to environmental influences that, over time, damage the concrete.

Physical influences on durability include frost action, mechanical wear, temperature changes, changes in volume of the concrete constituents and variations in loading. Chemical influences include attack by de-icing and/or sea-water salts, attack by sulphates, chlorides, organic acids and carbon dioxide, leaching, long-term changes in composition of the cement gel and alkali-aggregate reaction. Some physical influences may be addressed by air entrainment, finishes, design detailing, etc.. However, in all cases, the concrete's permeability and its chemical composition are critical to the rate of deterioration.

CEMENT COMPOSITION

Cement is manufactured by grinding and burning certain types of limestone, chalk, alumina and silica (found in shales and clays) with coal to about 1400°C in a rotary kiln. As the material heats up it dries; adsorbed water is driven out, carbon dioxide is driven out of the calcium carbonate and other carbonate minerals, and various chemical transformations take place as about 20-30% of the material liquifies. The material is then removed and cooled rapidly.

The resulting clinker has particles ranging in size from about 3 to 25 mm. It is ground to a powder with a particle size averaging about 15-20 um. Some gypsum is mixed in to improve resistance to vapour attack prior to use and to prevent a "flash set" on mixing.

The most typical constituents of the Portland cement powder are (1):

calcium oxide,	CaO,	abbreviated to:	C	60-65% by weight
silicon dioxide,	SiO_2		S	20-25%
aluminum oxide,	A_2O_3		A	5-10%
iron oxide,	Fe_2O_3		F	0.5-10%
magnesium oxide,	MgO		M	0.1-5.5%
sodium oxide, Na_2O			N	0.5-1.3%
potassium oxide,	K_2O		K	0.5-1.3%
sulphate,	SO_3		$\underline{S}$	1- 3%

These constituents are combined by the liquefaction and clinkering process into a collection of alkali salts, defined by equations which approximate the percentage mass of the oxide, after Bogue (3):

a] calcium aluminoferrite, C_4AF is itself an approximation which represents a series of solid solutions ranging from C_2F to C_6A_2F that crystallize between 1389°C and 1335°C. Bogue Number: $C_4AF = 3.04 \times F$

b] tricalcium aluminate, C_3A is formed by solid phase reaction at high temperature in mixtures where the ratio $A/(C+A) < 0.49$. The alumina that is not used as C_4AF is used up as C_3A. Bogue Number: $C_3A = 2.65A - 1.69F$

c] dicalcium silicate, C_2S exists in three distinct crystalline forms at temperatures below 1446°C, and is capable of dissolving small percentages of metal oxides at high temperature. Rapid cooling of the cement clinker causes the higher-temperature monoclinic C_2S structure to persist after cooling: this eventually causes "dusting" as the conversion to the low-temperature orthorhombic structure takes place spontaneously, shattering the structure. The orthorhombic structure has a specific gravity of 10% less than that of the monoclinic structure. Bogue Number:

$$C_2S = 2.87S - 0.754C_3S$$

d] tricalcium silicate, C_3S is created by a solid-state reaction of calcium oxide and dicalcium silicate at temperatures above 1250°C, provided the ratio $C/(C+S) > 65\%$. C_3S is unstable below 1250°C and, if annealed, will revert to $C + C_2S$; however, it can exist indefinitely at temperatures below 700°C. In cement manufacture, the tricalcium silicate crystals contain various oxides in solution, including A, F and M: these oxides stabilize the crystalline structure to room temperature and they also influence the cement properties. Bogue Number:

$$C_3S = 4.07C - 7.60S - 6.72A - 1.43F - 2.85SO_3.$$

Lea (1) notes that the actual C_3S content in clinker is hard to measure, but is about 10% less than the Bogue value because of incomplete sintering and the rate of cooling.

Fig. 2 shows the relationship between calcium, silicon and aluminum oxide products after sintering, and the area of interest for Portland cement. Characteristic proportions (1) include 45% C_3S, 25% C_2S and less than 30% ($C_3A + C_4AF$).

HYDRATION PROCESSES (1)

a] calcium silicates C_3S and C_2S:

These compounds represent in the order of 70% of the cement composition and are the primary source of long-term strength of ordinary Portland cement concretes. Reactions may be approximated as:

$$2C_3S + 6H \Rightarrow C_3S_2H_3 + 3Ca(OH)_2,$$
$$2C_2S + 4H \Rightarrow C_3S_2H_3 + Ca(OH)_2.$$

Finely ground tricalcium silicate starts to hydrate quickly, with both lime and silica passing into solution. The concentration of lime in solution increases steadily while that of silica decreases, and the hydration results in the formation of calcium hydroxide crystals and a gelatinous or amorphous mass of hydrated calcium silicate. The gel forms around the cement particles and is relatively impervious to water, making complete hydration extremely slow.

Dicalcium silicate reactions cannot be detected microscopically until after many weeks. A hardened paste forms with varying C:S ratio and as much as 15% unhydrated C_2S has been found in the paste after four years.

In C_2S compounds in which some of the lime has been replaced, eg.$KC_{23}S_{12}$, the hydration process is observed within a day or so.

The composition of the calcium-silicate-hydrate (C-S-H) gel changes during the reaction process, stabilizing as combination of:
a] poorly crystallized foils or platelets with a C:S molar ratio of 0.8-1.5, which forms at relatively low concentrations of lime in solution, referred to as CSH (I), and
b] a fibrous structure with a C:S ratio of 1.5-2.0, CSH (II), which forms in high concentrations of dissolved lime.

The C-S-H gel is stable while in contact with the saturated lime solution, but if it is placed in water some lime in the gel hydrolyses until an equilibrium concentration is restored or continued extraction removes all lime from the gel. This system is reversible: the C:S ratio may be increased by adding lime. The C-S-H gel, the crystallized calcium hydroxide and the solution must therefore be considered as interdependent, although changes in the solid formations are slow because of low solubility.

The presence of alkalis Na_2O and K_2O may also affect composition of the gel by drawing calcium silicates into solution as NC_2S, NCS, KC_2S and KCS. Also, Al^{3+}, Fe^{3+}, Mg^{2+} and SO_4^{2-} and other ions may enter and react with the C-S-H gel.

b] tricalcium aluminate C_3A:

Finely ground C_3A reacts very rapidly with water, initially forming relatively soluble C_4AH_{19} and/or C_2AH_8 crystals and, later, less soluble C_3AH_6.

Hydrated calcium aluminates are susceptible to attack by carbon dioxide, forming $C_3A.CaCO_3.11H_2O$.

c] tetracalcium aluminoferrite "C_4AF" (C_6AF_2 to C_6A_2F):

Though less reactive than C_3A, hexagonal plate crystals of C_4AH_{19} and/or C_2AH_8 form, together with hydrated iron oxide. The C-A-H formation converts to the cubic C_3AH_6 form at temperatures above 15°C. In the presence of excess lime, the reaction is less rapid and produces soluble compounds $C_4A.aq$ and $C_4F.aq$ which convert in temperatures above 15°C to crystalline C_3AH_6 and C_3FH_6.

d] gypsum $CaO.SO_3.2H_2O = C\underline{S}H_2$:

Calcium sulphate dissolves readily in the alkaline cement mixture. The sulphate ion can be combined by the C-S-H gel at early hydration ages, and released at later ages (9). It also can alter the C_3A and, to a lesser extent, the C_4AF reaction processes.

e] calcium hydroxide CaO or $Ca(OH)_2$:

A saturated solution of CaO has a pH of 12.45 and contains 1.14 g/L of water at 25°C; there is some variation with temperature. Solubility is greatly reduced by the presence of alkali hydroxides, and crystallization occurs slowly in supersaturated solutions, with relatively large crystals formed.

f] other hydrates that may form in concrete include:

- magnesium silicate, eg. M_4SH_{11}, which may be formed in set cement under attack by magnesium sulphate.
- magnesium aluminate, eg. $M_2AH_{9.9}$.
- calcium ferrites, eg. C_2FH_5.
- hydrogarnet solid solutions of C_3AH_6, C_3FH_6, C_3AS_3 and C_3FS_3.
- calcium sulphoaluminates, eg $C_3A.3CaSO_4.31H_2O$ (high sulphate form, found in ettringite)
- calcium sulphoferrites.
- calcium carboaluminates, eg. $C_3.A_{12}O_3.3CaCO_3.30H_2O$, and carboferrites.
- calcium chloroaluminates and chloroferrites
- magnesium hydroxide $Mg(OH)_2$
- silica $Si(O_3.OH)$

HYDRATION OF PORTLAND CEMENT

When water is mixed with ordinary Portland cement the liquid phase becomes a solution of the hydroxides and sulphates of calcium, sodium and potassium:

$$CaSO_4 + 2\,(Na,K)OH \Leftrightarrow (Na,K)2SO_4 + Ca(OH)_2$$

Solid calcium hydroxide persists at all ages, while the gypsum is used up as increasing amounts of calcium sulphoaluminate and C-S-H are formed. C-A-H, calcium sulphoaluminate and C-S-H precipitate, and some of the A, F and $\underline{S}$ are taken up in the C-S-H. The hydration process is summarized in Fig. 3.

The rate of hydration depends on the cement composition, water content, cement fineness and temperature. Hydration is generally in the sequence C_3A, C_4AF, C_3S, then C_2S, and this is also the sequence for contribution, on an individual unit mass basis, to heat of hydration over the first 28 days. C-S-H gel formed from C_3S is responsible for the early strength of concrete; C_2S starts to contribute more than C_3S, on a unit mass basis, after about 3 months (3).

Characteristic values for the average depth of hydration of cement particles over time are (1):

1 day:	0.43 um
7 days:	2.60 um
28 days:	5.37 um
5 months:	8.9 um

The hydrating mass sets on partial hydration of the products of C_3A, C_4AF and C_3S. The set occurs after 1/2 h to 2 h as the colloidal dispersion of hydrating cement particles starts to break down into disordered couplings and forms a condensed crystallization network. The reaction is accelerated by the breakup and removal of hydrated coatings on the cement particles, possibly due to osmotic pressure. The unstable gel then fills with hydration compounds.

The C-S-H gel includes sheet-like layers 3-4 molecules thick (30-40 A), with pores in the form of thin slits 15-30 A wide. The gel also contains calcium hydroxide crystals, residues of unhydrated cements, capillaries and entrapped/entrained air voids. (Fig. 4)

Physical effects of hydration on the concrete mass include:

a] surface tension due to thermal expansion stresses (caused by heat of hydration),

b] shrinkage cracking of cooling concrete surfaces,

c] surface compression due to cooling of the core of mass concrete, and

d] tension and creep due to shrinkage during hydration.

REACTIONS AFFECTING DURABILITY

Leaching: lime and some alumina are dissolved by water passing through concrete, resulting in gradual decomposition. Calcium carbonate and carboaluminate deposits may form where the leachate is exposed to the atmosphere. The pH of capillary water is replenished by solution of hydroxide salt crystals until they are substantially used up; the pH then decreases with an accelerating effect on the breakdown of the C-S-H gel.

Carbonation: carbon dioxide dissolves to form carbonic acid which reacts with calcium hydroxide in the capillary water. The reactants form calcium bicarbonate in a neutral pH range, which is relatively soluble, or carbonate deposits at high pH. Carbonation proceeds very slowly into the exposed concrete surface: the rate and extent of carbonation depend on the permeability of the concrete, capillary moisture content and ambient carbon dioxide levels. If the carbonation penetrates significantly, it neutralizes the alkaline environment which protects reinforcement. However the reaction reduces the surface permeability, possibly due to the release of water in the carbonation reaction (1), and there is considered to be a limiting depth of carbonation. Carbonation is also considered to contribute to overall volume shrinkage (2). Similar effects occur in the presence of other acid solutions.

Sulphate attack: sulphates in acids or sodium and/or potassium solutions react with free calcium hydroxide and C-A-H, forming calcium sulphate and sulphoaluminate deposits which are characterized by cracking (due to an increase in volume) and white formations. Magnesium sulphate also reacts with C-S-H, forming calcium sulphate, magnesium hydroxide and aqueous silica: this reaction proceeds very slowly because of the low solubility of magnesium hydroxide, provided the pH remains above 10.5; however, if leaching reduces the pH then a non-binding deposit of magnesium silicate is formed which further reduces the strength of concrete.

Chlorides: gypsum and ettringite (calcium sulphate and sulphoaluminate) are more soluble in the presence of chloride ions than sulphate ions. The calcium formations tend to leach out of concrete in the presence of chloride solutions, eg. road salt, resulting in an increase in porosity and a potential for galvanic pitting of reinforcement.

Seawater: both sulphates and chlorides are present. Concrete tends to expand slowly in this environment due to both the formation and the leaching of gypsum and ettringite and due to the formation of magnesium sulphate. Salts tend to crystallize above high water at a point of evaporation of pore water. The rate of deterioration is highly dependent on permeability.

Alkali-aggregate reactions: reactions may occur at the aggregate/cement interface. Reactive silica aggregates, including opaline silica and flints, react with sodium or potassium hydroxide from the cement and other sources, after the initial set and hydration. The reactants form an alkali-silicate gel, causing swelling at the aggregate/gel interface and eventual bursting of the concrete mass. In some cases, a soft gel is observed at the cracked surface, possibly a combination of alkali-silicates, gypsum and/or ettringite.

The rate and extent of the reaction is influenced by:

a] quality of silica and fineness of aggregate (size of reactive surface),
b] availability of water for reaction, which depends on saturation and permeability, and
c] availability of free alkali in the cement.

Alkali-carbonate reactions are rare and do not generate a gel: the reaction requires the presence of dolomitic limestone and acid-insoluble clay, degraded quartz or mica, and water.

Delayed ettringite formation: considerable research is currently taking place into the influence ettringite formation in concretes after they have set. While the extent to which delayed ettringite formation affects durability is unclear at present, it is known that there is a steady-state condition of ettringite presence to which concrete pastes will tend and that, the development of ettringite crystals being expansive, timing of the development of ettringite is critical. The formation of ettringite has been correlated to the presence of sulphate ions (>1%) and magnesium oxide (>0.5%), and to alkalinity and cement fineness. Ettringite develops during the initial setting of cement paste: C_3A particles are typically surrounded by a layer of hydrated aluminum monosulphate which in turn is surrounded by ettringite crystals. However, above 80^0C the formation of ettringite is not as extensive as at lower temperatures; subsequent reduction in temperature results in the formation of ettringite in the hardened paste over time. The literature includes reference to volumetric expansion of as much as 0.5% over 12 years, which may be observed as separation between aggregate and paste and cracks in the paste only between coarse aggregate particles. The voids left between the paste and aggregate particles may become filled with ettringite or products of alkalis-aggregate reaction.

TYPES OF CEMENT

ASTM standards for Portland cement include five basic chemistries to suit different objectives (10). Characteristic compositions are given in Table 1 (2). The cements include:

Type 1	ordinary	- general, most widely used
Type 2	modified	- similar strength gain to Type 1; moderate heat

		gain and sulphate resistance
Type 3	rapid hardening	- higher C_3S, greater heat than Type 1; similar setting time
Type 4	low-heat	- reduced C_3S and C_3A
Type 5	sulphate resistant	- low C_3A and C_4AF

Portland-pozzolan cements are grouped by ASTM (11):

Type IP Portland-pozzolan cement for general construction: 15-40% pulverized fly ash (PFA)

Type P Portland-pozzolan cement for use where high strengths at early ages are not required (15-40% PFA)

Type I(PM) Pozzolan-modified Portland cement for general construction (>15%PFA)

The properties and testing of pozzolans are also specified by ASTM (12, 13).

Other types of cementitious material include:

- high-alumina cement
- cements with granulated blastfurnace slag (GBS):
    - slag cement (hydrated lime and GBS)
    - Portland blastfurnace cement (interground Portland cement and GBS)
    - supersulphated cement (GBS, Portland cement or lime and $CaSO_4$)
- oil-well cements: coarse-ground, with retarders

- masonry cements: eg. Portland cement and lime; fine ground, with air entraining
- gypsum plasters:
    - partially hydrated $CaSO_4$
    - Plaster of Paris: $CaSO_4.0.5H_2O + CaSO_4$ + keratin
- expansive cements, which make use of the expansive reaction to calcium sulphoaluminate
- white Portland cement (low in F)
- coloured Portland cement (including pigments)

POZZOLANS

Pozzolans are not cementitious in themselves, but combine with lime and water to form stable, insoluble cement products.

Natural pozzolans are generally volcanic, including tuff from volcanic ash and dust. The term is also applied to certain clays, shales and silicious rock, including opal, which have lost much of their basic oxides: generally, these have limited pozzolanic properties but may be improved by burning to remove adsorbed water and/or carbonates (1). Natural pozzolans comprise a mixture of glass and crystalline silica, eg. leucite, KAS_4 ($K_2O.Al_2O_3.4SiO_2$).

Artificial pozzolans include burnt clays, shales and other sedimentary rocks, coal ash and burnt bauxite. The clays and shales are typically hydrated aluminum silicates, eg. kaolinite (AS_2H), though iron, magnesium, sodium and calcium silicates are also present.

Examples of the use of burnt sedimentary rock include gaize and moler, which have been used with Portland cement for several decades in France and Denmark, respectively, for marine concrete structures.

The use of fly ash in concrete has been reported since 1914 (14), but the first comprehensive study of its pozzolanic properties was not reported until 1937 (15). Large amounts of fly ash were used in the Hungry Horse Dam in the United States (the fourth largest dam in the world when built in the late 1940's); the research before and during construction and reports on the performance of the dam led to more extensive use of fly ash worldwide. The use of fly ash in concrete is considered to be beneficial to the environment as well as economic: in 1983, the US Environmental Protection Agency required the procurement of "the highest percentage of recovered material practicable, given that reasonable levels of competition, cost, availability and technical performance are maintained" for federally funded works, as part of its Resource Conservation and Recovery Act, in line with its objectives of "protection of human health and the environment and conservation of valuable material and energy resources" (16).

The characteristic composition of some artificial pozzolans is described in Table 2 and Fig. 5. The composition of fly ash varies considerably with source of coal and burner operation, but is approximately 40-50% silica, 20% alumina and 10% ferrous oxide. Lime is also significant in some types of ash.

FLY ASH PRODUCTION AND COMPOSITION

The coal that fly ash comes from is classified by:

a] type: bright, semi-splint and splint (dull); banded, non-banded

b] rank: degree of coalifaction (conversion from having a high oxygen and hydrogen content to a high carbon content). From lowest to highest, the ranks are lignite, brown coal, sub-bituminous, bituminous and anthracite, and

c] grade: amount of organic material, which reduces with run-of-mine coal mined away from the seam.

In conventional boilers, the coal is pulverized into a powder and blown into a furnace where it burns at a high temperature in an upward flowing gas stream. For the bituminous coals burned in Nova Scotia, the ash produced is typically between 8 and 15% of the original coal mass. About 85% of this ash leaves the boiler as fly ash and the remainder falls and is discharged as bottom ash. The process of combustion and ash formation is described in Fig. 6.

During the process of burn-out, the charred particles fracture, typically to between three and five times the number of coal particles: these pieces include hollow spherical particles (cenospheres) which are believed to be formed from small inclusions in the coal or from the fusion of a few nearby inclusions. Also,

a] heavy metals in the coal boil or sublime to vapour,

b] the alkali-metal salts in the char vaporize at about 1080°C and

c] above 1630°C, silica volatilizes in the presence of carbon:

$SiO_2 + C$ (carbon) => $SiO + CO$

As the furnace gases cool, the inorganic vapours coalesce to particles less than 0.1 um in size, or condense as a coating on the fly ash particles, which are 0.1-50 um in size. At lower temperatures, some unburned organic compounds are also absorbed onto the surfaces of the fly ash particles.

Fly ash particles are collected from the flue gas stream using mechanical collectors, fabric filters, wet scrubbers and/or electrostatic precipitators. The collection efficiency of these systems for various particle sizes in given in Table 3. Electrostatic precipitators, which are most common in Nova Scotia, force the flue gas between charged plates to which the particles are attracted and then fall from the gas stream into hoppers. Conditioning agents may be added to the flue gas to improve collection efficiency, resulting in a change in fly ash composition.

Table 4 describes the mineral composition and particle size distribution of fly ash collected by two methods (mechanically and by precipitator). Particle size distributions of three fly ashes are compared with Portland Cement in Fig. 7, and the shape of the individual particles is shown in Fig. 8. Spherical glass particles are by far the most abundant component in the ash: they are quartz and mullite with potassium, sodium, iron and sulphur coating the surface, and have a size distribution similar to cement. The chemical composition of ash (Table 5) is dominated by silica and alumina and, in the case of lower-ranking coals, lime.

As shown in tables 3 to 5, there is a wide range in the physical and chemical properties of fly ash; this range is related to the type of coal and the processes used to burn the coal and collect the ash. Another major influence is the way in which the boiler is operated: the carbon content in the ash increases considerably when the rate of burning is adjusted to meet changing electrical loads. "Base loaded" plants operating under stable load conditions are able to optimize their heat rate through more complete combustion: they produce ash with a lower carbon content.

POZZOLANIC REACTION CHEMISTRY

Little or no research into the performance of fly ash and lime is reported, possibly because of the construction industry's familiarity with the properties of Portland cement or because Portland cement is in the same price-range as lime in most regions of North America. Also, there may be shortcomings in attempting to have a lime/fly ash mixture replicate the process of construction using Portland cement.

However, there has been significant research in the past decade into the chemistry and performance of concrete containing a mixture of fly ash and Portland cement. Scanning electron microscopy has confirmed that glass and metals on the fly ash particle surface hydrolyse and react with the pore fluid, forming deposits on the glass surface or in the C-S-H gel (6). Fig. 9 shows particles in a "High Volume Fly Ash" (HVFA) paste that are etched by the pore solution. The micrograph images also show deposition on the inner and outer surfaces of the cenospheres and the leaching away of the original particle.

The aluminosilicate glass is a disordered three-dimensional -Si-O-Si- network (polymer) with aluminum and some iron substituting for approximately one-third of the silicon compounds in the network. In the presence of strong alkalis or acids, depolymerization occurs, forming siloxane groups (O_3-Si-O-Si-O_3) and silaloxane groups (O_3-Si-O-Al-O_3) which hydrolyse to form silanols (O_3-Si-OH) and alanols (O_3-Al-OH):

$$>Si\text{-}O\text{-}Si< \; + \; M^+ OH^- \quad \Rightarrow \quad >Si\text{-}OH \; + \; >Si\text{-}O^- \; + \; M^+$$

$$>Si\text{-}O\text{-}Al< \; + \; M^+ OH^- \quad \Rightarrow \quad >Si\text{-}OH \; + \; >Al\text{-}O^- \; + \; M^+$$

$$\text{or} \quad >Al\text{-}OH + >Si\text{-}O^- \; + \; M^+$$

(M represents Na or K)

Most of the hydrolysed groups remain attached to the matrix, while the aluminate, silicate and aluminosilicate particles enter solution and eventually precipitate by combination with calcium hydroxide.

The reaction rate is slow and is thought to be determined by the rate of diffusion of ions through the hydrolysing glass surface (17). Reaction products include the C-S-H, C-A-H and C-A-S-H formations as for Portland cement, and hydrolysed fly ash particle "pseudomorphs" (6). Calcium hydroxide is consumed in the reaction.

It is inferred from the references that the pozzolanic reaction described for fly ash takes place when other glasses and silicates, eg. blast-furnace slag, silica fume and alkali-silica reactive aggregates, are combined with water and either lime or Portland cement.

HIGH VOLUME FLY ASH CONCRETE

Fly ash is commonly used in conjunction with ordinary Portland cement. Commercially, the fly ash is usually considered as a direct substitute for, or an additive to, Portland cement. Direct "substitutions" in the range of 10-25% are typical throughout the USA and in Canada where market conditions permit.

As the proportion of fly ash in concrete is increased, the properties of the cementitious material and the concrete matrix start to differ significantly from ordinary Portland cement and concrete, and the concepts of substitution and addition become less appropriate. High volume fly ash (HVFA) concrete is discussed here because of its growing significance in the concrete industry and also because it demonstrates the properties that are altered to a lesser extent by nominal substitutions of fly ash for Portland cement.

The first uses of HVFA concrete reported in Nova Scotia (18) were in the Park Lane shopping/office complex and foundation caissons for Purdy's Wharf Phase 2 office tower. The shopping/office complex design required relatively high concrete strength in the lower columns on completion of the project: this enabled the cross-sectional shape of the columns to be maintained over the height of the building while minimizing the loss of occupancy space. A 120-day strength of 50 MPa was specified for the lower columns, beams and slabs. In the case of the pile foundation, a cohesive, workable mixture was required for placement. The design required a strength of 45 MPa at 28 days. Cost of concrete was a major consideration in both cases.

The same concrete mixture was used for both projects, except that the maximum size of coarse aggregate was limited to 14 mm for the foundation design:

Portland cement:	180 kg/m^3
Fly ash (ASTM Class F):	220 kg/m^3
Coarse Aggregate:	1100 kg/m^3
Fine Aggregate:	800 kg/m^3
Water:	110 L/m^3
Superplasticizer:	6 L/m^3

This mixture was developed through a testing program which examined a number of mixtures using the 28-day strength as a benchmark for comparison.

As with ordinary Portland cement concretes, reducing the water content increases the strength at 28 days, but this comes at the cost of workability during placement. However, the water/Portland cement ratio of HVFA is high, about 61% in the above mixture, and the reaction of C_3A, C_3S and C_4AF in the cement particles is not restrained by lack of water. Initially, the HVFA concrete may be considered to be a low-cementitious, well-hydrated Portland cement concrete with very low porosity because of the presence of an ultra-fine aggregate (fly ash). It is inferred that the ash particles may release alkali ions into solution initially, but the aluminosilicate glass in fly ash does not react significantly until after the concrete has set.

The long-term strength gain of concretes with fly ash is, however, much greater than for ordinary Portland cement concrete. The slow reaction of the aluminosilicate glass in fly ash adds to the strength gains from reaction of the C_2S in Portland cement. Fig.10 indicates strength gains of about 40-70% for various mixtures of HVFA between 28 days and one year from placement. The gains and variability in strength in the Park Lane project are described in Fig. 11.

Superplasticizers are typically used to achieve the required workability. Plasticizers and superplasticizers are water-reducing admixtures which retard the flocculation of particles during hydration: cement particles become negatively charged and coated with an oriented layer of water molecules, causing dispersion. The superplasticizers are used to reduce the water content while maintaining workability.

At low ratios of fly ash replacement, however, the fly ash is considered to improve the workability and cohesiveness of the mixture. Initially, this improvement was considered to be due to the smooth spherical shape of the fly ash particles, but more recent studies describe flowing cement as a motion of flocs containing numerous fly ash particles, in which the particle shape is not particularly significant. It is suggested (6) that the improved workability is the result of an adsorption and dispersion process similar to water-reducing admixtures: the surface of fly ash particles are negatively charged, and it is suggested that very fine fly ash particles adhere to the positively charged calcium aluminate cement particles, reducing or reversing their attraction to the negatively charged surfaces in the fluid.

In developing suitable mixtures for Park Lane and Purdy's Wharf 2, it was concluded that, from a commercial and/or performance viewpoint, an optimum fly ash content may be in the order of 55-60%. Table 7 provides approximate values for the current cost of materials to ready-mix concrete suppliers in the Halifax area, together with representative mixture designs and composite costs. HVFA concrete is more economical to produce, but:

a] use of fly ash typically requires investment in a fly ash silo at the concrete manufacturer's site, and
b] trained supervision and inspection are required in the use of superplasticizers.

The performance of HVFA concrete is discussed below ("Durability"). The material is considered to perform at least as well as ordinary Portland cement except where high early strength is required or where there is unusual surface abrasion and a concern with dust or scaling. HVFA concrete has been used successfully in:

a] road construction (20), which may be roller-compacted, eliminating the expense of plasticizers,
b] mass concrete structures, including bridge piers, dams and mat foundations, in which the lower heat of hydration and drying shrinkage are important; again, roller-compaction eliminates the need for plasticizers, and
c] general structural applications (foundations, buildings, containers etc.).

HIGH PERFORMANCE CONCRETE (HPC)

These concretes are generally specified for their high compressive strength, although their low water content, use of superplasticizers and low porosity leads to their having improved placement and durability characteristics.

Concrete strengths in the range of 70-100 MPa have been developed and used in:

a] large offshore structures and bridges, particularly in Europe,

b] high-rise buildings in North America,

c] general prefabrication (high early strength), and

d] prefabricated struts, eg. in reticulated 3-D structures.

Fly ash and blast-furnace slag may be considered as supplementary ingredients in HPC and are used primarily to reduce the superplasticizer dosage and possibly eliminate the need for retarders (21).

"High strength concrete" is a type of HPC. The Portland Cement Association describe high strength concrete as having a strength in excess of 50 MPa and considers the ultimate limit for 90-day strength to be in the order of 200 MPa (29 Ksi). Fly ash is recommended as a mineral admixture for high strength concrete and silica fume is recommended for strengths above 100 MPa (22).

The mix proportions of some commercially available high strength concretes are given in Table 7. Characteristic properties include:

a] a cementitious content of 360-600 kg/m^3 (600-1000 lb/yd),

b] a water/cementitious ratio of 0.22-0.35, and

c] as much as 10 L/m^3 of superplasticizer.

The cement in HPC's forms an amorphous microstructure with less porosity than traditional concrete, particularly at the aggregate/paste interface. This results in more stress being transferred through the aggregate and greater durability and strength. The reduced porosity of the cement is achieved through controlling the particle size distribution of the cement and admixtures to maximize the composite density, and by limiting the water and sulphate contents. Silica fume, with an average particle size of 0.1 um, reduces the porosity of the mixture and also reacts with lime to contribute to gel strength.

Coarse and fine aggregates may be optimized in relation to the strength of the paste, the aggregate and the cement/aggregate bond. The perfect aggregate would be crushed, clean, cubical in shape and conditioned to a "saturated, surface dry" condition. The bond is more critical for coarse aggregates, while the shape and grading are more significant for fine aggregates. For example, in some Norwegian offshore construction, the sand is classified into eight fractions which are then blended for concrete manufacture (23).

Entrained air reduces the density and strength of the cement paste: a strength reduction of 5% is referenced for every 1% increase in air content (22). However, at strengths above 150 MPa, the need for entrained air is eliminated by the absence of pores capable of retaining freezable water (21).

The mixing and placing of HPC is similar to ordinary-strength concrete, with plasticizer added at the batching plant and/or in increments at the site. As with HVFA concrete, the use of HPC requires close cooperation between the engineer, the concrete producer, the construction contractor and the testing agency (22, 24).

In Canada, HPC has been used in bridges in Quebec (25), in the Hibernia offshore structure (23) and high-rise buildings (24). The advantages found in bridge construction included:

a] durability - resistance to freeze-thaw attack, chloride attack and scaling, because of low porosity and permeability; higher cracking loads,
b] structural - increased flexural and compressive strength; reduced elastic shortening and creep under prestress, and
c] constructibility - reduced time in prestress beds; reduced transfer length and "harping" of prestress tendons.

In offshore construction, the benefits of high strength concrete again include:

a] durability (low permeability),
b] structural - reduced cross-sections for platform cells and arches, which are principally in compression, and
c] constructibility - high slump concrete is required as reinforcement quantities of up to 1000 kg/m3 cause severe congestion (Fig. 12).

In the Toronto area, 70-85 MPa concrete has been used in Scotia Plaza, BCE Place and the Adelaide Centre, using cementitious blast-furnace slag and silica fume in conjunction with Type 10 Portland Cement (24).

Research in Japan (26) indicates that a "classified fly ash" can be used to improve the workability and reduce the cost of 70 MPa concrete: the ash was graded to have a maximum size of 5 um, partly to reduce impurities in the ash. 60 MPa concrete has been developed for building construction without fly ash, silica fume or blast-furnace slag by using a water/cement ratio of 0.285 and high-range water reducing agents (27).

DURABILITY EFFECTS OF FLY ASH USE

The permeability of Portland cement concrete depends on the water/cement ratio, the chemical composition of the reactants, the rate and extent of hydration, curing time and fineness and quantity of cement. When pozzolans are added, all of these factors are changed.

Most importantly, much of the leachable calcium hydroxide produced in ordinary Portland cement hydration is combined with the relatively insoluble C-S-H gel (Fig. 13): this leads to long-term gains in watertightness and resistance to aggressive environments as well as strength. Fig.14 shows the results of 30% replacement of cement by fly ash (14). The reduction in permeability results in increased resistance to leaching, carbonation, chemical attack and chloride penetration. However, the curing of the concrete surfaces is more critical for HVFA concretes than for ordinary Portland cement concretes.

Leaching: reduced with reduced permeability and reduced lime available to enter solution.

Carbonation: reduced, provided the surface is moist-cured for extended periods.

However, air drying after 7 days has been found to cause increased shrinkage cracking and permeability at the surface of fly ash concretes (14).

Sulphate attack: (sulphates react with free calcium hydroxide and C-A-H; magnesium sulphate also reacts with C-S-H): Pflughoeft-Hassett (30) comments on "conflicting and misrepresented information" in current literature on the effects of fly ash on sulphate attack. She concluded that low-calcium (Class F) fly ash improves Types 1 and 2 Portland cement at 25-45% replacement rates, consistent with Tikalski and Carrasquillo (31) and Dunstan (32); also that further research is needed on the subject. It is suggested (31) that the reaction of C-A-H is "diluted" in the presence of fly ash, and that the pozzolanic reaction reduces permeability by removing lime and precipitating C-S-H gel. It is also suggested (33) that fly ashes that are rich in calcium oxide can produce sulphate-resistant concretes at 25-70% cement replacement if additional gypsum is included in the mixture: the additional sulphate ions present on mixing are thought to form expansive ettringite while the concrete is still in its plastic state, thereby removing the risk of expansion on setting.

Shrinkage and shrinkage cracking: reduced by the presence of fly ash (32), but the presence of calcium in the ash reduces this effect.

Chloride attack: reduced with reduced permeability and reduced leachable lime.

Seawater attack: reduced with reduced permeability and reduced leachable lime.

Alkali-aggregate reactions: alkali-silica reactions are reduced significantly by the removal of lime in reaction with fly ash, which reduces the pH of the pore fluid and the capacity for increases in pH caused by the aggregate. Figure 15 describes dramatic reductions in the effects of alkali-silica reactions through the use of 20% fly ash in one case. The beneficial effect of fly ash is, however, highly dependent on the composition and fineness of the aggregates, the cement and the pozzolan, and on moisture conditions and permeability. For fly ashes with more than 1.5% alkali content, it has been found that small replacements of ash for cement in the mixture result in increased reactivity, while larger replacements have a beneficial effect (34).

Aggregates found throughout most of mainland Nova Scotia tend to be very reactive, and their use without fly ash has resulted in major deterioration in structures in as little as fifteen years.

Freeze-thaw attack: the expansion of freezing pore water can severely weaken concrete, and agents are typically used to entrain air which to absorb the expansion. The size of the air bubbles depends on the air-entraining agent, typically
0.05-1.25 mm in diameter, with an average separation of 0.25 mm (2). The demand for air entraining admixture increases with fly ash content and, as shown in Fig. 16, there is no appreciable increase in air content until a threshold admixture content is reached. The threshold is a function of carbon content ("Loss On Ignition") and possibly fineness and pH, and may vary considerably.
However, air entrainers are not significant to the cost of concrete (Table 6). A simple "foam index" procedure is used to predict air entrainment requirements at the site. Fly ash concretes are also more prone to air loss with prolonged mixing.

Scaling: the resistance of HVFA concrete surfaces to scaling from de-icer salts is less than that for ordinary Portland cement concrete (17): it is inferred that the preparation and curing of the surface are critical to the penetration depth of de-icers, as for carbonation and surface shrinkage. However, the resistance to de-icer scaling is better with lower rates of fly ash than with no replacement (20).

RELIABILITY AND QUALITY CONTROL

The components in fly ash which cause the greatest variability in the performance are calcium oxide (lime) and carbon. ASTM standards limit the carbon content at 6%, with an exception to allow up to 12% carbon if approved by the user. Limits are placed on the lime content by requiring minimum contents of the combination (S+A+F); revised criteria which would classify fly ash by lime content have been proposed by several sources (14).

Two classes of fly ash are defined by ASTM Standard C-618 (Table 8):

- Class F fly ash is normally produced from burning anthracite or bituminous coal. It has a relatively low proportion of lime; (S+A+F) > 70%; pozzolanic only; and

- Class C fly ash is normally produced from burning lignite or sub-bituminous coal. It has a relatively high proportion of lime; (S+A+F) > 50%; pozzolanic and cementitious.

Methods for sampling and testing fly ash are contained in ASTM C311-85 (13) and blended hydraulic cements are specified in ASTM C595 (11).

Areas that are seen to require research include the development of rapid test methods that can be used to predict long-term performance, and mathematical modelling to specify materials appropriate for individual projects (14).

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NOTATION (17)

Cement chemist's shorthand notation will be used throughout this report to describe the principle cement oxides (SiO_2 = S; CaO = C; Al_2O_3 = A; Fe_2O_3 = F; SO_3 = $\bar{S}$; H_2O = H) and related phases:

C_3S (alite)	tricalcium silicate ($3CaO{\cdot}SiO_2$)
C_2S (belite)	dicalcium silicate ($2CaO{\cdot}SiO_2$)
C_3A	tricalcium aluminate ($3CaO{\cdot}Al_2O_3$)
C_4AF	calcium aluminoferrite ($4CaO{\cdot}Al_2O_3{\cdot}Fe_2O_3$)
C_2AS	gehlenite ($2CaO{\cdot}Al_2O_3{\cdot}2SiO_2$)
CH	calcium hydroxide (portlandite) ($Ca(OH)_2$)
C–S–H	amorphous or semi-crystalline calcium silicate hydrates
C–A–S–H	amorphous or semi-crystalline calcium aluminosilicate hydrates
$C\bar{S}$	anhydrite ($CaO{\cdot}SO_3$)
$C\bar{S}H_2$	gypsum ($CaO{\cdot}SO_3{\cdot}2H_2O$)
$C\bar{S}H_{0.5}$	calcium sulphate hemihydrate ($CaO{\cdot}SO_3{\cdot}0.5H_2O$)
AFm	monosulphate aluminoferrite, monosulphoaluminate
AFt	trisulphate aluminoferrite (ettringite)
$C_3A{\cdot}3C\bar{S}{\cdot}H_{31}$	ettringite ($6CaO{\cdot}Al_2O_3{\cdot}3CaSO_4{\cdot}31H_2O$) (also $32H_2O$)
$C_3A{\cdot}C\bar{S}{\cdot}H_{13}$	monosulphoaluminate ($3CaO{\cdot}Al_2O_3{\cdot}CaSO_4{\cdot}13H_2O$)
C_2ASH_8	gehlenite hydrate, stratlingite ($2CaO{\cdot}Al_2O_3{\cdot}SiO_2{\cdot}8H_2O$)
$C_{12}A_3FS_4H_{16}$	hydrogarnet ($12CaO{\cdot}3Al_2O_3{\cdot}Fe_2O_3{\cdot}4SiO_2{\cdot}16H_2O$)

Selected mineral names and formulae

ferrite spinel	$(Mg,Fe)(Fe,Al)_2O_4$
hauyne	$Ca_2(NaAlSiO_4)_6(SO_4)_2$
hematite	Fe_2O_3
lime	CaO
magnetite	Fe_3O_4
melilite	$Ca_2(Mg,Al)(Si,Al)_2O_7$
merwinite	$Ca_3Mg(SiO_4)_2$
mullite	$Al_6Si_2O_{13}$
periclase	MgO
portlandite	$Ca(OH)_2$
quartz	SiO_2

Abbreviations

AAS	atomic absorption spectroscopy
ASTM	American Society for Testing and Materials
BEI	backscattered electron image (mode)
DTG	derivative thermogravimetric analysis
EDXA	energy dispersive X-ray analysis
HVFA	high-volume fly ash
MPa	megapascal
psi	pounds per square inch
QXRD	quantitative X-ray diffraction
SEI	secondary electron image (mode)
SEM	scanning electron microscopy
TGA	thermogravimetric analysis
w/c	water to cement ratio
w/s	water to solid ratio
XRD	X-ray diffraction

ASTM type	CaO	MgO	Al_2O_3	Fe_2O_3	SiO_2	TiO_2	Na_2O	K_2O	SO_3	Free CaO	C_4AF	C_3A	C_3S	C_2S
I	63·8	3·7	5·6	2·4	20·7	0·23	0·21	0·51	1·6	0·4	7	11	55	18
	63·1	2·5	4·7	3·0	22·1	0·21	0·06	1·30	1·7	0·2	9	7	47	28
	65·8	1·1	4·7	2·1	22·2	0·30	0·04	0·19	1·6	1·6	6	9	54	23
	62·8	1·7	6·7	2·5	21·1	0·39	0·95	0·51	1·8	2·0	8	14	33	35
II	61·4	3·1	4·8	4·8	20·8	0·21	0·06	1·30	1·8	0·9	15	5	44	26
	64·9	1·9	4·0	2·1	24·0	0·23	0·23	0·55	1·7	1·5	6	7	41	38
III	65·6	1·4	5·2	2·5	20·0	0·27	0·21	0·44	2·3	1·8	8	10	63	10
	63·3	4·3	5·1	2·0	20·3	0·21	0·19	0·28	2·5	1·9	6	10	51	19
IV	59·6	3·0	4·6	5·0	22·9	0·23	0·06	1·19	1·3	0·4	15	4	25	47
	63·6	1·1	3·7	3·1	25·2	0·19	0·33	0·01	1·9	0·4	9	5	31	49
V	64·3	1·7	3·1	3·3	24·4	0·19	0·08	0·22	1·4	0·5	10	3	45	36
	64·2	2·5	1·9	1·3	26·1	0·12	0·10	0·15	2·0	1·8	4	3	35	48
	63·3†	1·2	3·3	4·7	23·1	—	0·08	0·37	1·7	—	14	1	49	30

† Corrected for free CaO.

Table 1 Composition of some Portland Cements (1)

Pozzolana	SiO_2	Al_2O_3	Fe_2O_3	CaO	MgO	Na_2O and K_2O	SO_3	Ignition loss*
Burnt clay	58·2	18·4	9·3	3·3	3·9	3·9	1·1	1·6
Burnt clay	60·2	17·7	7·6	2·7	2·5	4·2	2·5	1·3
Spent oil shale	51·7	22·4	11·2	4·3	1·1	3·6	2·1	3·2
Raw gaize	79·6	7·1	3·2	2·4	1·0	—	0·9	5·9
Burnt gaize	88·0	6·4	3·3	1·2	0·8	—	Trace	—
Raw moler	66·7	11·4	7·8	2·2	2·1	—	1·4	5·6
Burnt moler	70·7	12·1	8·2	2·3	2·2	—	1·5	—
Raw diatomite (U.S.A.)	86·0	2·3	1·8	Trace	0·6	0·4	—	8·3
Burnt diatomite (U.S.A.)	69·7	14·7	8·1	1·5	2·2	3·2	—	0·4
Fly-ash (U.S.A.)	47·1	18·2	19·2	7·0	1·1	3·95	2·8	1·2
Fly-ash (U.S.A.)	44·8	18·4	11·2	11·6	1·1	3·14	2·0	7·5
Fly-ash (British)	47·4	27·5	10·3	2·1	2·0	5·7	1·8	0·9
Fly-ash (British)	45·9	24·4	12·3	3·6	2·5	4·2	0·9	4·1

* Includes carbon in case of spent oil shale and fly-ash.

Table 2 Percentage Composition of some Artificial Pozzolans (1)

General class	Specific type	Typical capacity	Overall efficiency (%)	Fractional efficiency in percent for various size ranges in μm 0-5	5-10	10-20	20-44	>44
Mechanical collectors	Baffle	1000-3500 ft^3/min per ft^2 of inlet area	60	7.5	22	43	80	90
	Conventional cyclone		65	12	35	57	82	91
	High-efficiency cyclones	2500-3500 ft^3/min per ft^2 of inlet area	85	40	79	92	95	97
Fabric filters	Automatic	1-6 ft^3/min per ft^2 of fabric area	99+	99.5	100	100	100	100
Wet scrubbers	Impingement baffle	400-600 ft^3/min per ft^2 of baffle area	—	—	—	—	—	—
	Packed tower	500-700 ft^3/min per ft^2 of bed cross-sectional area	94	90	96	98	100	100
	Venturi	6000-30,000 ft^3/min per ft^2 of throat area	99+	99	99.5	100	100	100
Electrostatic precipitators	Dry, single-field	2-8 ft^3/min per ft^2 of electrode collection area	97	72	95	97	99+	100
	Wet (charged-drop scrubber)	5-15 ft^3/min per ft^2 of electrode collection area	—	—	—	—	—	—

Source: Adapted from K. Wark and C. F. Warner. *Air Pollution, Its Origin and Control,* New York: IEP/A Dun-Donnelley Publisher, 1976.

1 ft^3/min/ft^2 = 0.00508 m/s

Table 3 Operating Characteristics of Particle Collectors (14)

	Electrostatically precipitated					
	% retained on sieves*					
Constituent	74 µm No. 200	44 µm No. 325	37 µm No. 400	25 µm No. 500	% passing No. 500	Whole sample, %**
Glass	32	49	52	56	87	79
Magnetite-hematite	2	14	13	14	5	6
Carbon	33	8	9	5	1	4
Anisotropic material	27	22	18	15	3	6
Aggregates	6	7	8	10	4	5
Totals	100	100	100	100	100	100

	Mechanically collected					
	% retained on sieves*					
Constituent	74 µm No. 200	44 µm No. 325	37 µm No. 400	25 µm No. 500	% passing No. 500	Whole sample, %**
Glass	35	55	61	58	63	58
Magnetite-hematite	8	5	20	26	16	16
Carbon	47	29	7	8	5	13
Anisotropic material	3	3	3	3	5	4
Aggregates	7	8	9	5	11	9
Totals	100	100	100	100	100	100

*Percentage is based on count of more than 300 particles in each sieve fraction.
**Percentage is based on gradation of as-received sample and on distribution of constituents of wet sieved fractions.

Table 4 Mineralogical Compositions of Particle Size Fractions of Two Fly Ashes (14)

	Abdun-Nur 1961	Jarrige 1971		Sersale 1980	
Component	Coal	Coal	Lignite	Bituminous coal	Lignite
SiO_2	28-51	47-54	18-25	50	23-50
Al_2O_3	15-34	28-35	12-15	30	8-14
Fe_2O_3	4-26	4-12	6-8	7	8-20
CaO	1-11	1-4	43-49	2	18-50
Free CaO	—	0.1	18-25	—	—
MgO	0-3*	1-2.5	2-3	5	—
Na_2O	—	0.2-2	5	5	—
K_2O	—	1-6			—
SO_3	0.2-4	0-1	5-9	—	—
C	—	0.5-12	1-3	1-6	—
LOI	0.56-32	2	1	5-12	—

Table 5 Chemical Composition of Various Fly Ashes (14)

CONCRETE MIX COST COMPARISON

NOTE: This comparison is based only on the material costs to the ready mix producer for the material required to produce the various concrete mixtures

UNIT PRICES

cement	$120.00	per tonne
flyash	$60.00	per tonne
sand	$9.00	per tonne
stone	$10.00	per tonne
air entrainment	$1.05	per litre
wrda82	$1.40	per litre
wrda19	$3.00	per litre

PRICE SUMMARY		SAVINGS (per m^3)
NO FA	$63.32	
20% FA	$60.58	$2.74
55% FA	$58.15	$5.17

MIX A
30 MPa non flyash

cement	355	kg.	$42.60
flyash	0	kg.	$0.00
sand	778	kg.	$7.00
stone	1040	kg.	$10.40
air	230	ml	$0.24
wrda82	2200	ml	$3.08
		TOTAL:	$63.32 per cubic metre

MIX B
30 MPa 20 % flyash

cement	296	kg.	$35.52
flyash	74	kg.	$4.44
sand	750	kg.	$6.75
stone	1040	kg.	$10.40
air	240	ml	$0.25
wrda82	2300	ml	$3.22
		Total	$60.58 per cubic metre

MIX C
37.5 MPa High volume flyash syster

cement	150	kg.	$18.00
flyash	190	kg.	$11.40
sand	750	kg.	$6.75
stone	1100	kg.	$11.00
air	190	ml	$0.20
wrda19	3600	ml	$10.80
		Total	$58.15 per cubic metre

Table 6: Concrete Mix Cost Comparison (19)

Ingredient, units per cu yd (m^3)	Mixture number					
	1	2	3	4	5	6
Cement, Type I, lb (kg)	950 (564)	800 (475)	820 (487)	950 (564)	800 (475)	551 (327)
Silica fume, lb (kg)	—	40 (24)	80 (47)	150 (89)	125 (74)	45 (27)
Fly ash, lb (kg)	—	100 (59)	—	—	175 (104)	147 (87)
Coarse agg., SSD, lb (kg)**	1800 (1068)	1800 (1068)	1800 (1068)	1800 (1068)	1800 (1068)	1890 (1121)
Fine agg., SSD, lb (kg)	1090 (647)	1110 (659)	1140 (676)	1000 (593)	1000 (593)	1251 (742)
HRWR, Type F, fl oz (litre)	300 (11.60)	300 (11.60)	290 (11.22)	520 (20.11)	425 (16.44)	163 (6.30)
HRWR, Type G, fl oz (litre)	—	—	—	—	—	84 (3.24)
Retarder, Type D, fl oz (litre)	29 (1.12)	27 (1.05)	25 (0.97)	38 (1.46)	39 (1.50)	—
Total water, lb (kg) †	267 (158)	270 (160)	262 (155)	242 (144)	254 (151)	238 (141)
Water-cement ratio	0.281	0.338	0.320	0.255	0.318	0.432
Water-cementitious materials ratio	0.281	0.287	0.291	0.220	0.231	0.320

**Maximum nominal aggregate size: Mixtures 1-5, 1/2-in.; Mixture 6, 1-in.
†Weight of total water in mix including water in admixtures.

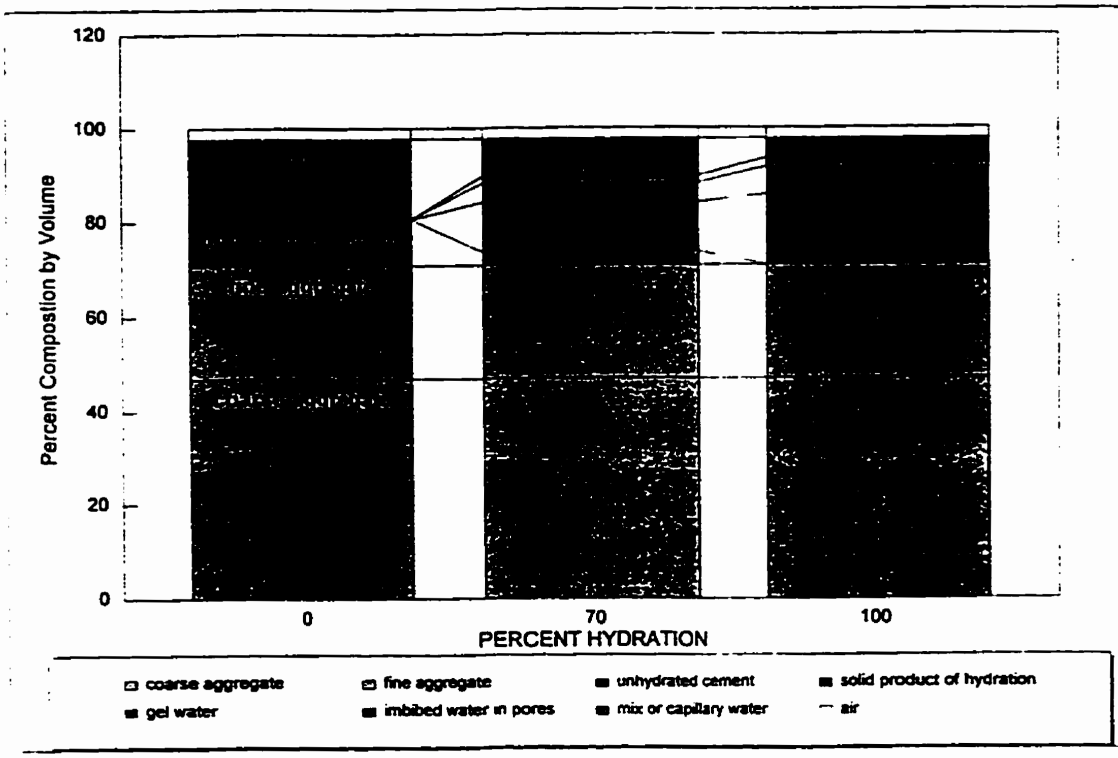
Table 7 Typical Proportions in Commercially Available HSC Mixtures (22)

	Class F	Class C
A. Chemical requirements		
(SiO_3 + Al_2O_3 + Fe_2O_3), min.,%	70.0	50.0
SO_3, max., %	5.0	5.0
Moisture content, max., %	3.0	3.0
Loss on ignition, max., %	6.0*	6.0
Optional chemical requirements		
Available alkalies, as Na_2O, max., %	1.50	1.50
B. Physical requirements		
Fineness:		
Amount retained when wet-sieved on No. 325 (45 μm) sieve, max., %	34	34
Pozzolanic activity index:		
With portland cement, at 28 d, min., % of control	75	75
With lime, at 7 d, min., psi (kPa)	800(5500)	
Water requirement, max., % of control	105	105
Soundness:		
Autoclave expansion or contraction, max., %	0.8	0.8
Uniformity:		
Specific gravity, max. variation from average, %**	5	5
Percent retained on No. 325 (45 μm) sieve, max. variation, percentage points from average**	5	5
Optional physical requirements		
Multiple factor, product of LOI, %, and fineness, %, max.	255	—
Increase of drying shrinkage of mortar bars at 28 d, max., %	0.03	0.03
Uniformity requirements:		
When air-entraining concrete is specified, quantity of AE agent required to produce air content of 18.0 vol % shall not vary from the average of previous 10 by more than, %	20	20
Reactivity with cement alkalies:		
Mortar expansion at 14 d, max., %	0.020	0.020

*Reduced from 12.0% in 1984, with the provision that Class F pozzolan containing up to 12.0% loss on ignition may be approved by the user if either acceptable performance records or laboratory test results are made available.
**Average established by the ten preceding tests, or by all the preceding tests if the number is less than ten.

Table 8 Chemical and Physical Requirements Defined in ASTM C618-85 for Fly Ashes for Use in Portland Cement (14)

HYDRATION (%):	0	70	100
Composition by volume (%):			
air	2.3	2.3	2.3
mix or capillary water	17.1	8.0	4.1
imbibed water in pores	0.0	1.4	1.9
gel water	0.0	4.1	5.9
solid product of hydration	0.0	10.5	15.0
unhydrated cement	9.8	2.9	0.0
fine aggregate	23.9	23.9	23.9
coarse aggregate	46.9	46.9	46.9
TOTAL:	100	100	100
Water content (% of concrete volume):			
combined water	0.0	5.0	7.1
gel water	0.0	4.1	5.9
mix or capillary water	17.1	8.0	4.1
TOTAL:	17.1	17.1	17.1
Porosity (% of concrete volume):	19.4	15.8	14.2



Mix proportions 1:2:4 (cement:fine aggregate:coarse aggregate, by mass)
water/cement ratio 0.55
entrapped air 2.3%
water added during curing
no air entrainment

figure 1: volumetric proportions of concrete during hydration.

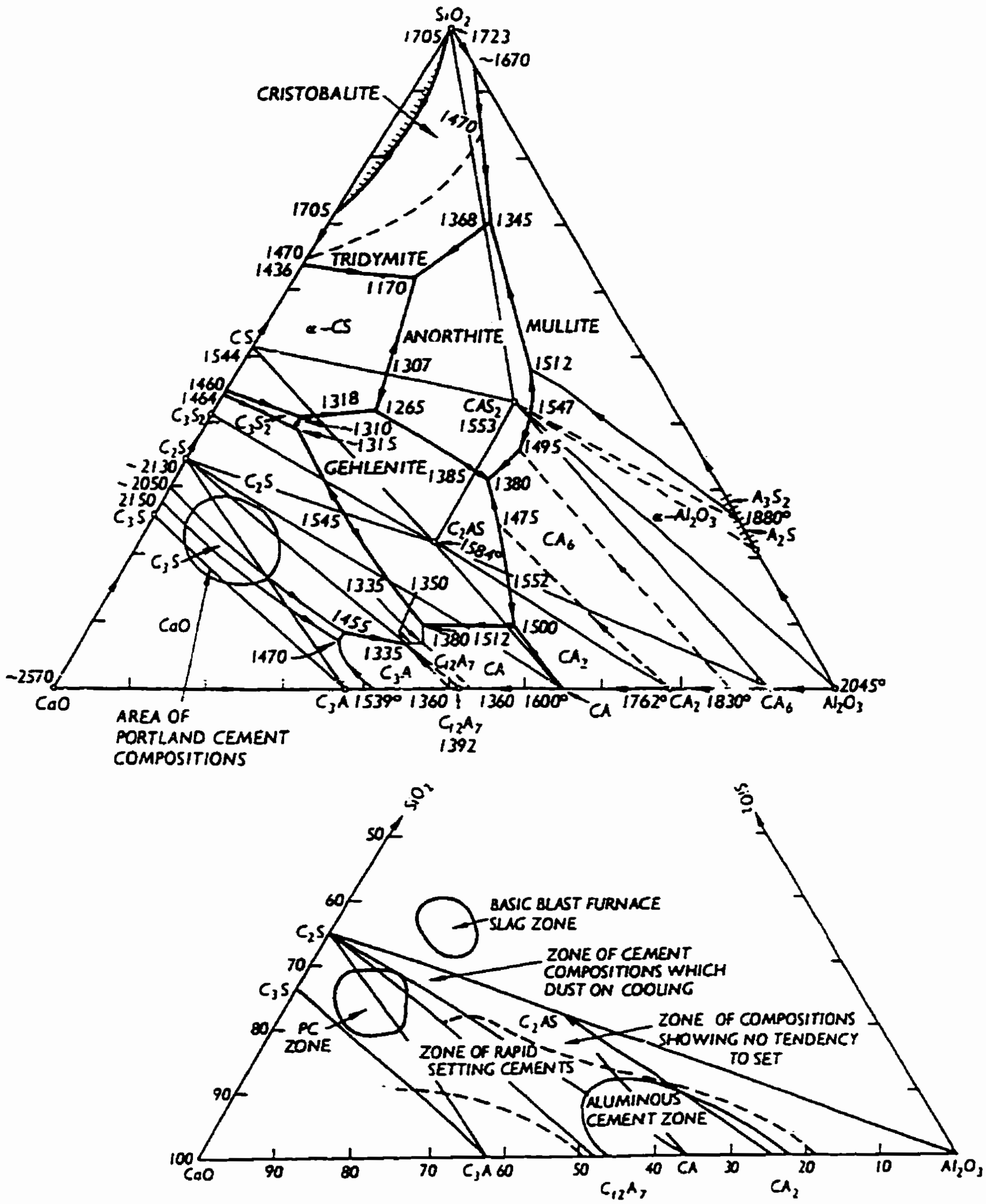


Figure 2 Equilibrium Diagram, System CaO-Al_2O_3-SiO_2 (1)

$2CaO.SiO_2$ (β-form) $\xrightarrow[\text{Slow}]{\text{I}}$ $2CaO.SiO_2.aq$

$3CaO.SiO_2$ $\xrightarrow[\text{Moderate}]{\text{Ia}}$ $3CaO.SiO_2.aq$

$2CaO.SiO_2.aq$ + $3CaO.SiO_2.aq$ $\xrightarrow{\text{II}}$

$Ca(OH)_2$ + calcium silicate hydrate gel (*C/S* about 1·5) incorporating some Al_2O_3, Fe_2O_3 and SO_3

$\downarrow$ IIIa

Calcium silicate hydrate gel (*C/S* 1·5–1·8) containing Al_2O_3

$\downarrow$ IIIb

More crystalline product (?)

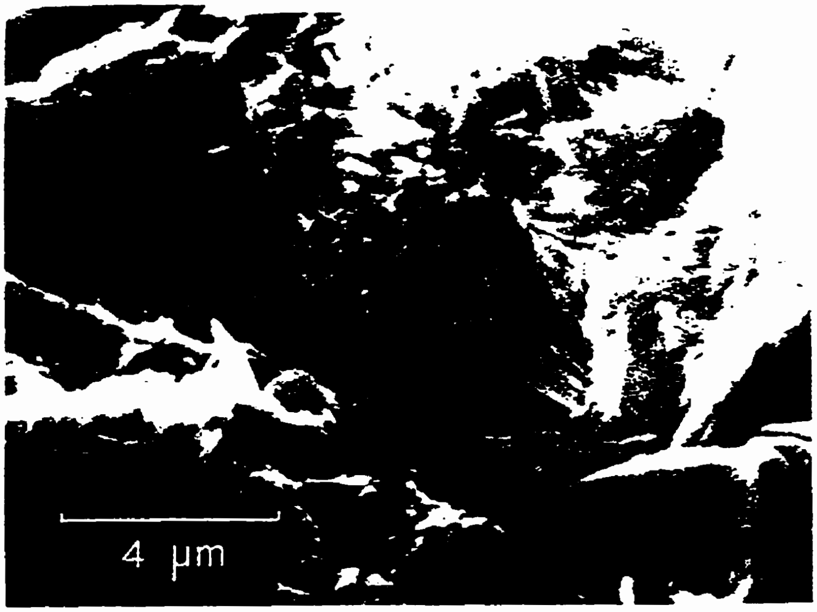
$3CaO.Al_2O_3 + CaSO_4.2H_2O$ $\xrightarrow[\text{Immediate}]{\text{IV}}$ Needles of $3CaO.Al_2O_3.3CaSO_4.aq$ $\xrightarrow{\text{V}}$ Hexagonal-plate solid solution $3CaO.Al_2O_3.Ca(SO_4, (OH)_2).aq$

$4CaO.Al_2O_3.Fe_2O_3 + CaSO_4.2H_2O + Ca(OH)_2$ $\xrightarrow[\text{Quick}]{\text{IVa}}$ Needles of solid solution $3CaO.(Al_2O_3, Fe_2O_3).3CaSO_4.aq$ $\xrightarrow{\text{Va}}$ Hexagonal-plate solid solutions $3CaO.(Al_2O_3, Fe_2O_3).CaSO_4.aq$ and/or $3CaO(Al_2O_3, Fe_2O_3).Ca(SO_4, (OH)_2).aq$

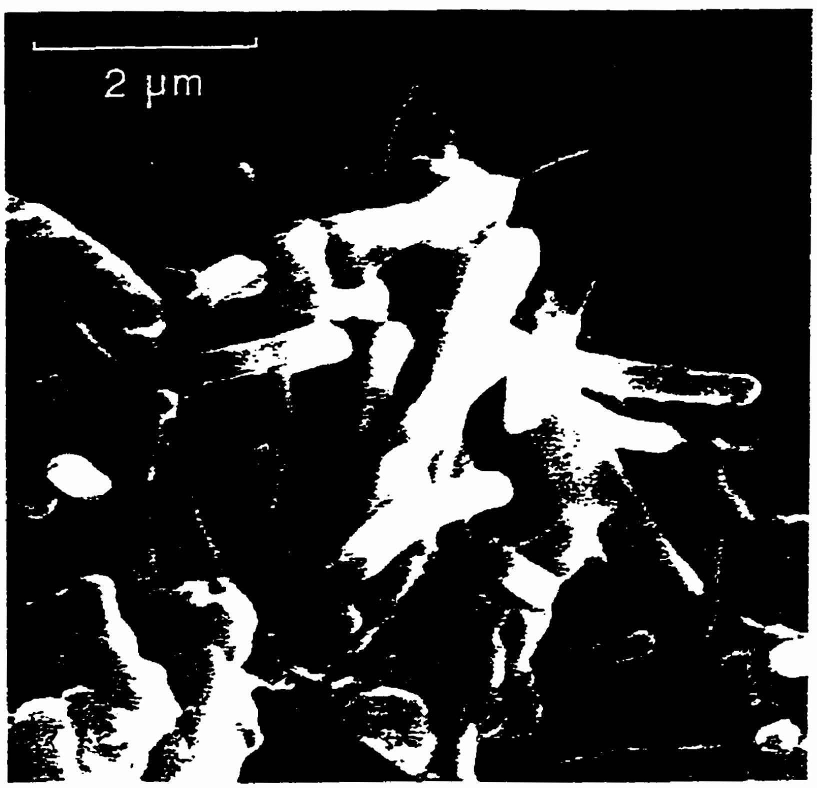
$\xrightarrow{\text{VI}}$

Hexagonal-plate solid solution $3CaO.(Al_2O_3, Fe_2O_3).Ca(SO_4, (OH)_2 SiO_3).aq$ and possible formation of $4CaO.(Al_2O_3, Fe_2O_3).aq$
Hydrogarnet phase containing alumina, ferric oxide and silica

Figure 3 Hydration of Portland Cement (1)



(i) Scanning Electron Micrograph of CSH Crystals in Set Cement



(ii) As XI (i) but at Higher Magnification

Figure 4 Scanning Electron Micrograph of CSH and CaO Crystals in Set Concrete (1)

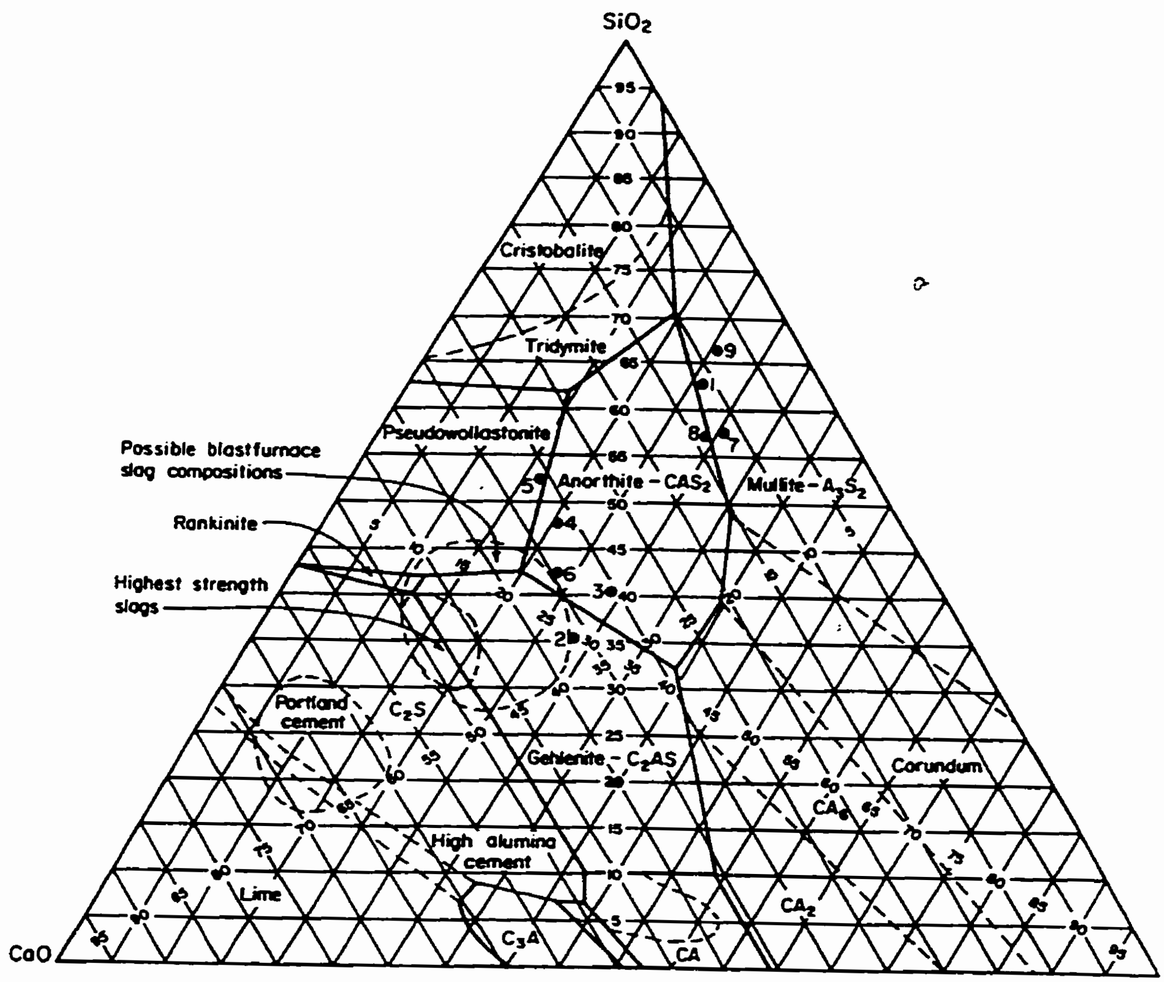


Figure 5 Comparison of Composition of Nine Fly Ashes with Blast Furnace Slags and Portland Cements, System $CaO-Al_2O_3-SiO_2$ (14)

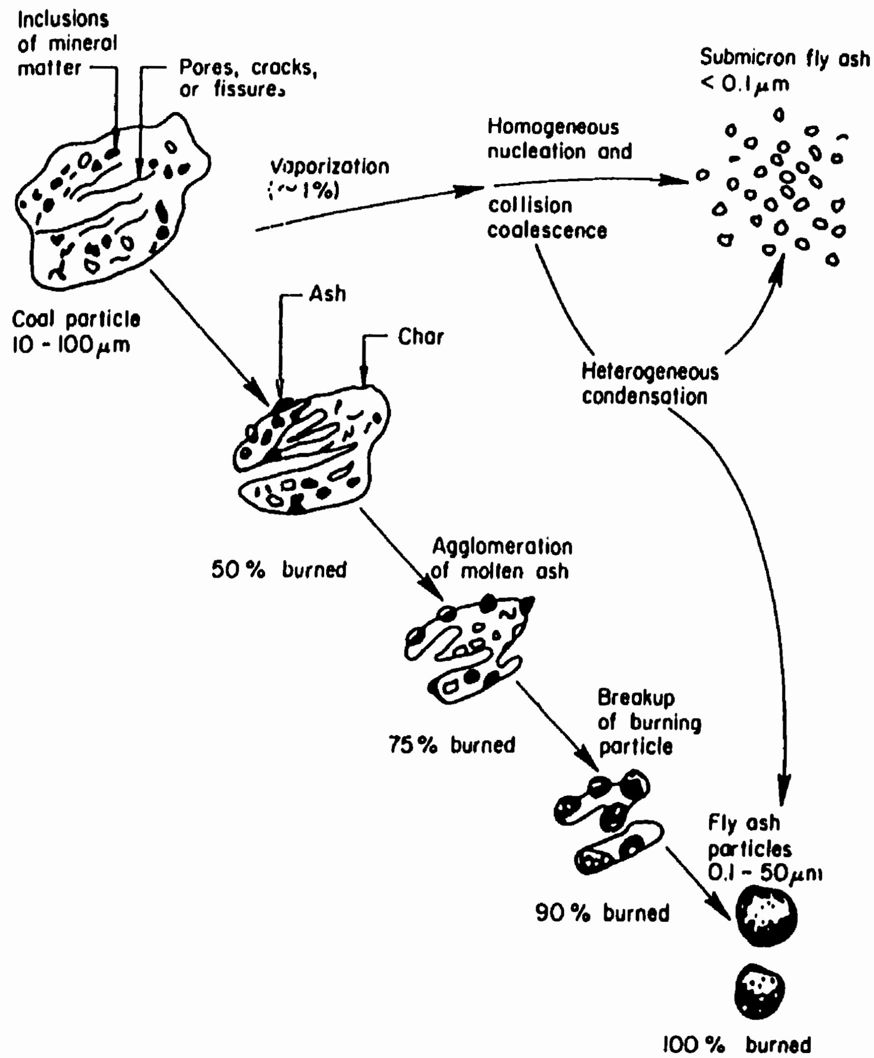
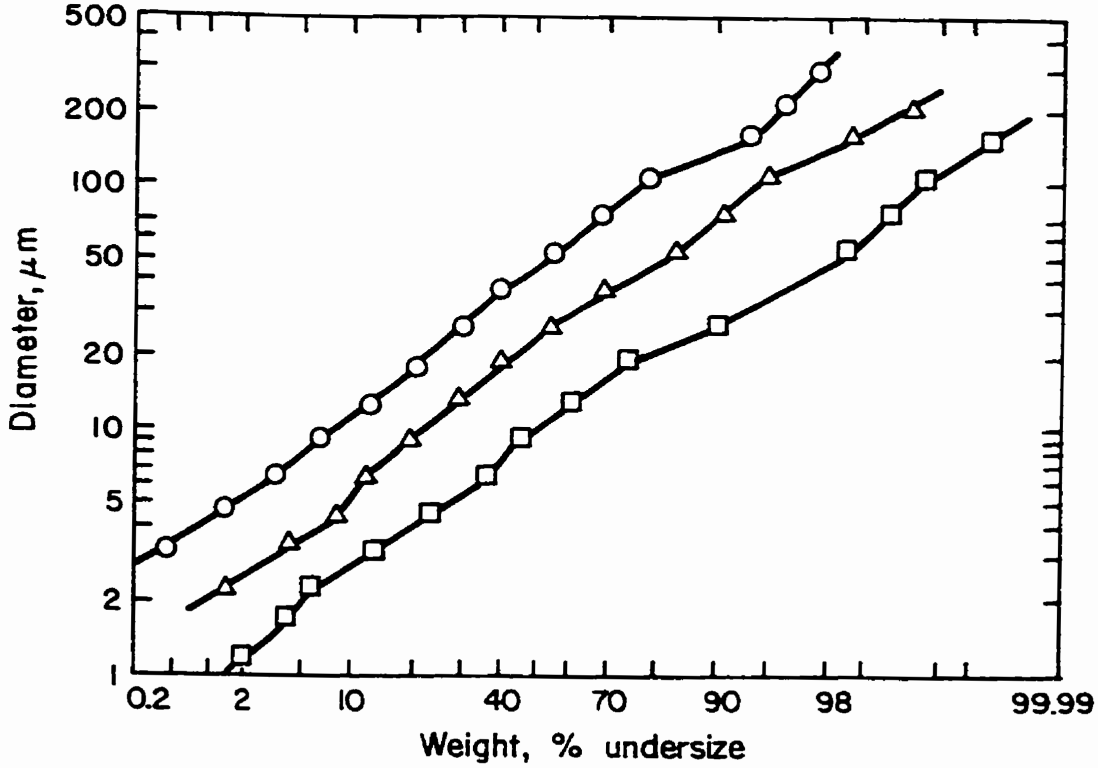


Figure 6 Coal Combustion and Ash Formation Processes (14)



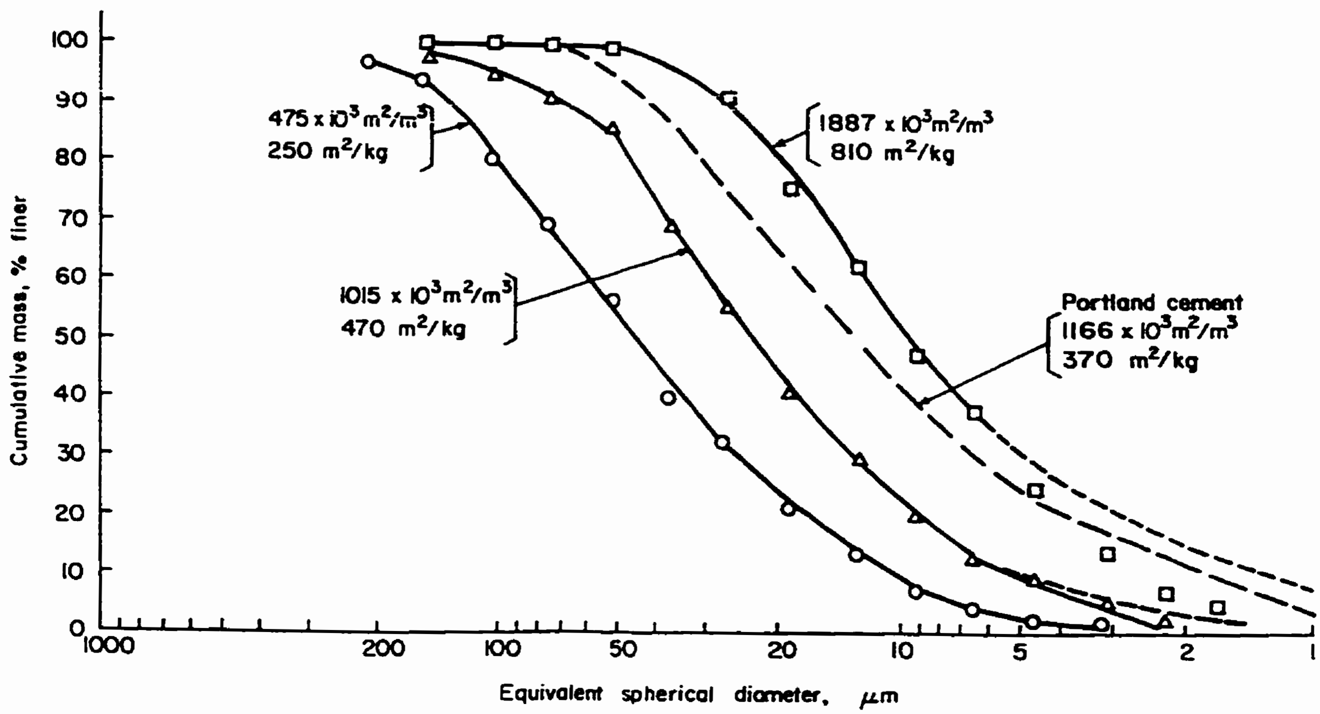


Figure 7 Particle Size Distributions of Three Fly Ashes & Type 1 Portland Cement (14)

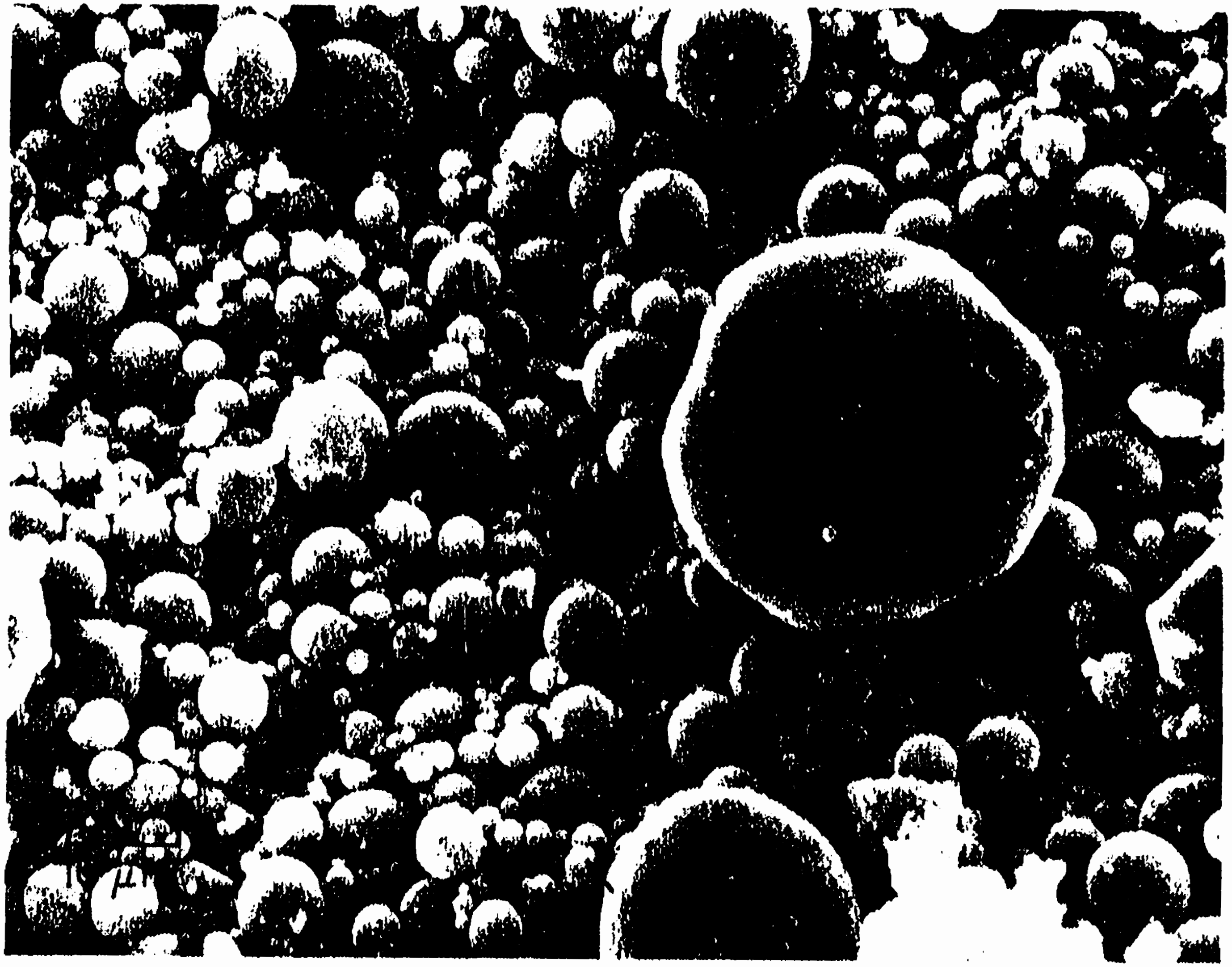


Figure 8 Scanning Electron Micrograph (SEM) of Class F Fly Ash (14)

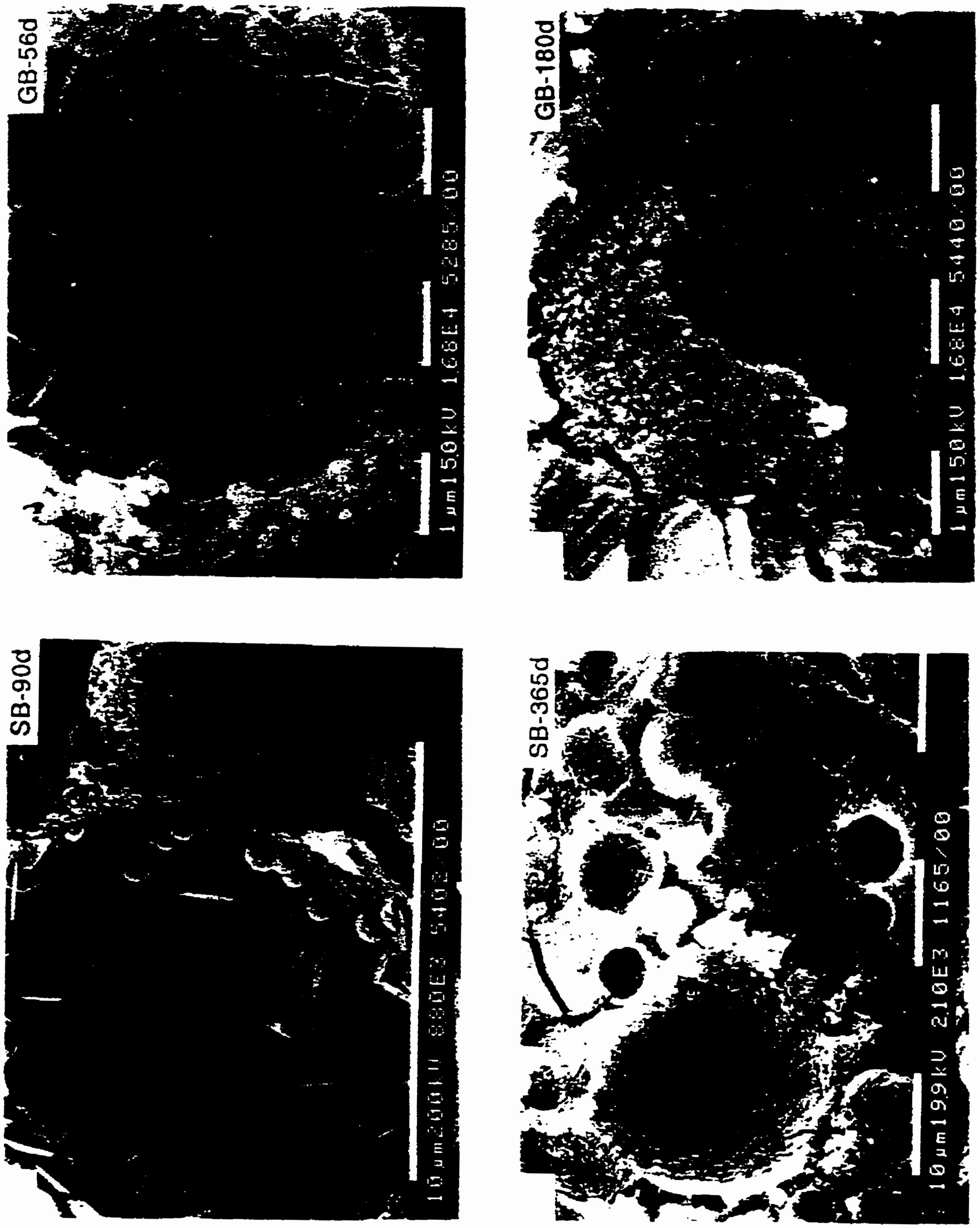


Figure 9(a): SEM of fracture surfaces in HVFA pastes (17)

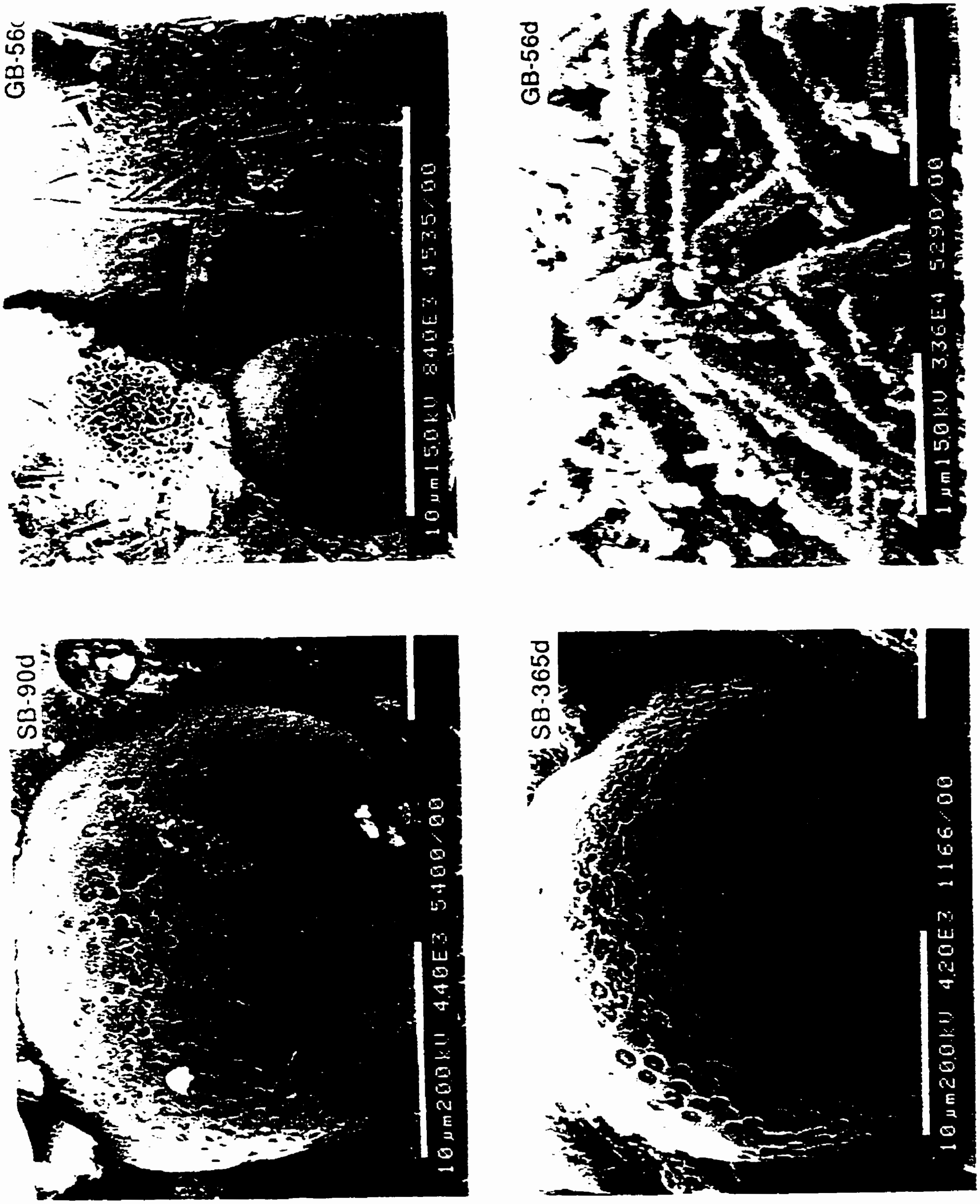


Figure 9(b): SEM of fracture surfaces in HVFA pastes (17)

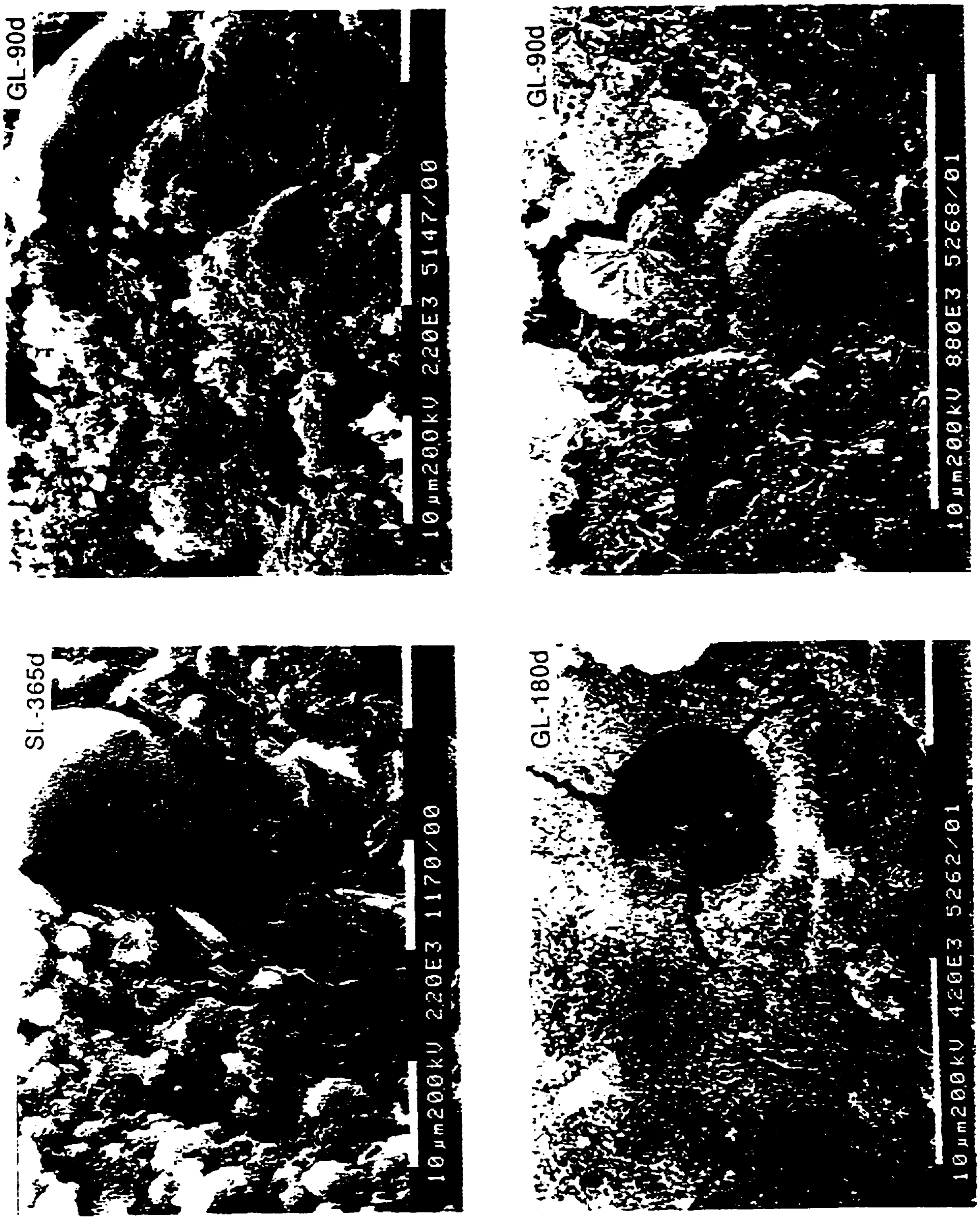
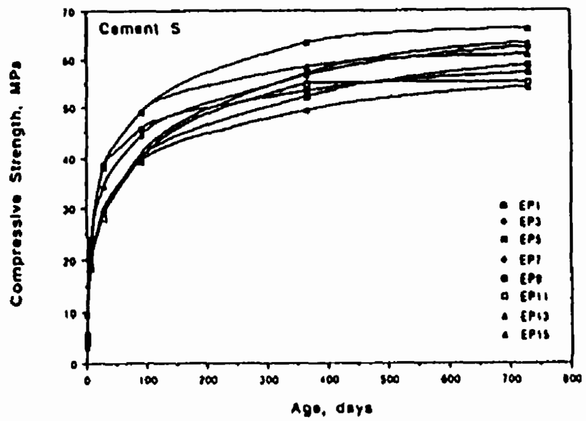
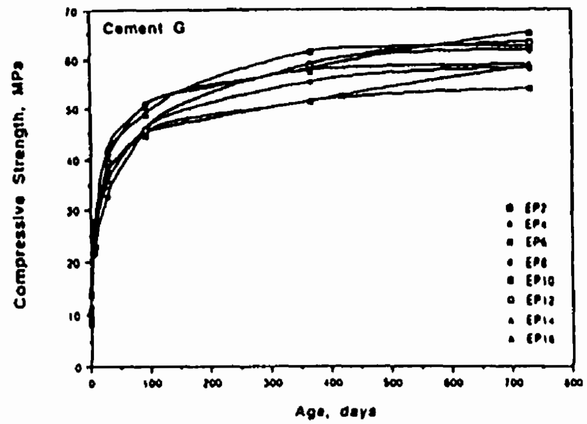


Figure 9(c): SEM of fracture surfaces in HVFA pastes (17)





Mixture No.	Cement	Fly Ash Source	Density kg/m^3	Compressive Strength, MPa 1 d	7 d	28 d	91 d	1 year	2 years
EP1	S	Montour	2380	5.6	18.8	28.8	39.7	52.0	58.4
EP2	G	Montour	2385	10.3	24.9	39.3	50.9	57.9	65.2
EP3	S	Bowen	2400	4.9	18.3	29.5	40.6	56.8	63.0
EP4	G	Bowen	2345	10.5	21.2	32.5	45.0	58.8	61.6
EP5	S	Leland Olds	2365	9.7	23.9	38.1	45.6	53.2	56.8
EP6	G	Leland Olds	2375	13.9	26.3	37.2	45.0	51.3	53.8
EP7	S	Gibson	2395	3.1	22.5	34.3	44.3	56.3	62.2
EP8	G	Gibson	2380	11.5	24.8	37.4	45.6	55.2	58.1
EP9	S	Coal Creek	2385	4.1	23.0	38.5	48.9	63.1	65.9
EP10	G	Coal Creek	2380	9.1	27.6	41.4	48.8	61.2	62.4
EP11	S	Belews Creek	2375	5.8	18.2	27.8	39.5	54.7	55.0
EP12	G	Belews Creek	2365	10.8	22.6	35.0	45.8	58.7	63.2
EP13	S	Gorgas	2390	5.6	19.0	30.1	39.1	49.2	53.9
EP14	G	Gorgas	2360	10.3	21.9	36.9	44.7	51.4	58.2
EP15	S	Navajo	2360	5.1	21.6	38.1	48.9	58.2	60.6
EP16	G	Navajo	2365	8.4	26.5	42.2	50.4	57.5	58.7

Figure 10 Compressive Strength Development of Concretes made with various HVFA Mixtures (17)

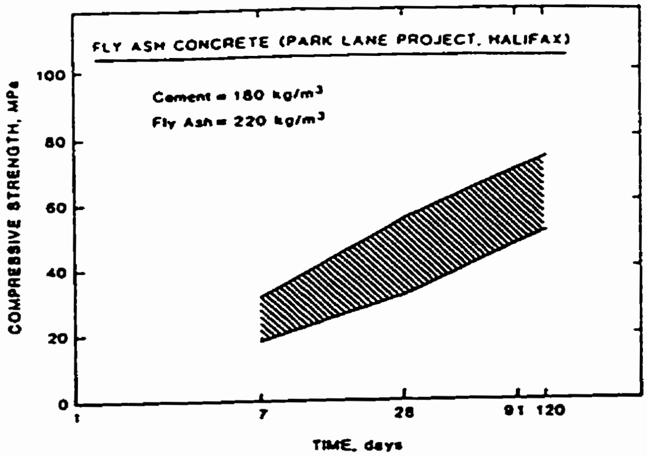


Figure 11 Envelope of Compressive Strength in a Structural Field Application (18)

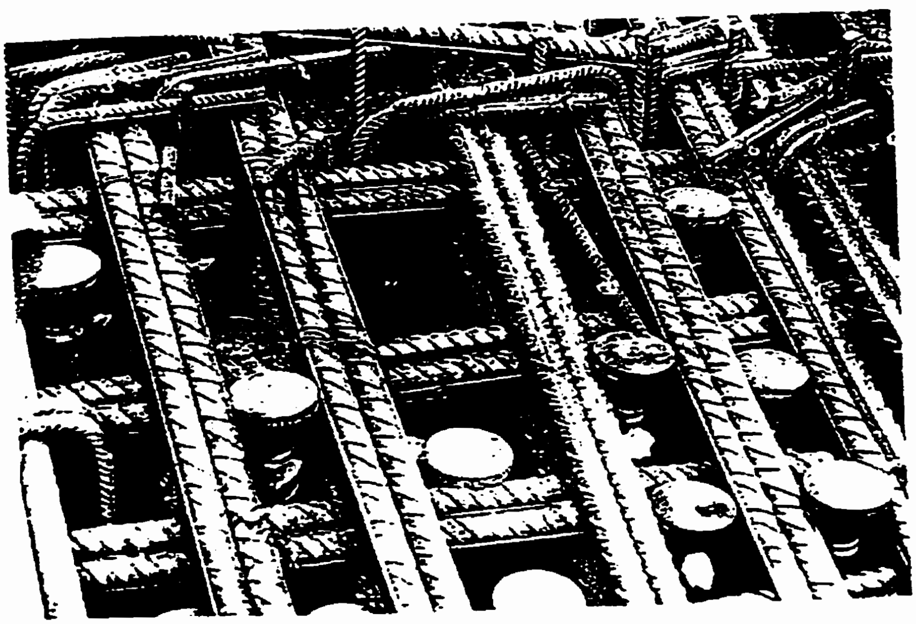


Figure 12 Reinforcement of Shell Element for an Offshore Platform (23)

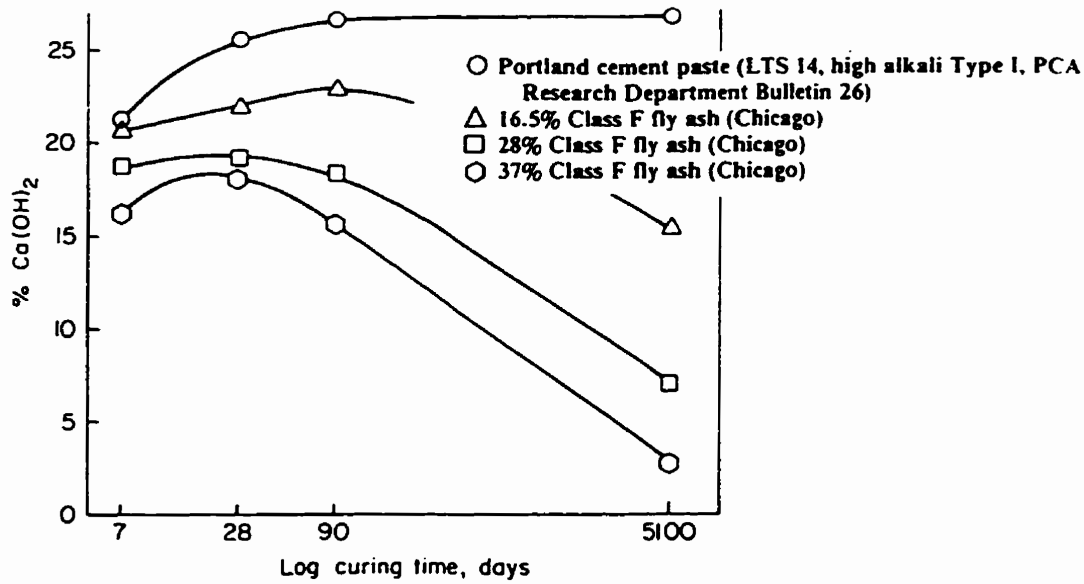


Figure 13 Changes in Calcium Hydroxide Contents of portland and Pozzolanic Cement Pastes During 14 Years Moist-Curing (14)

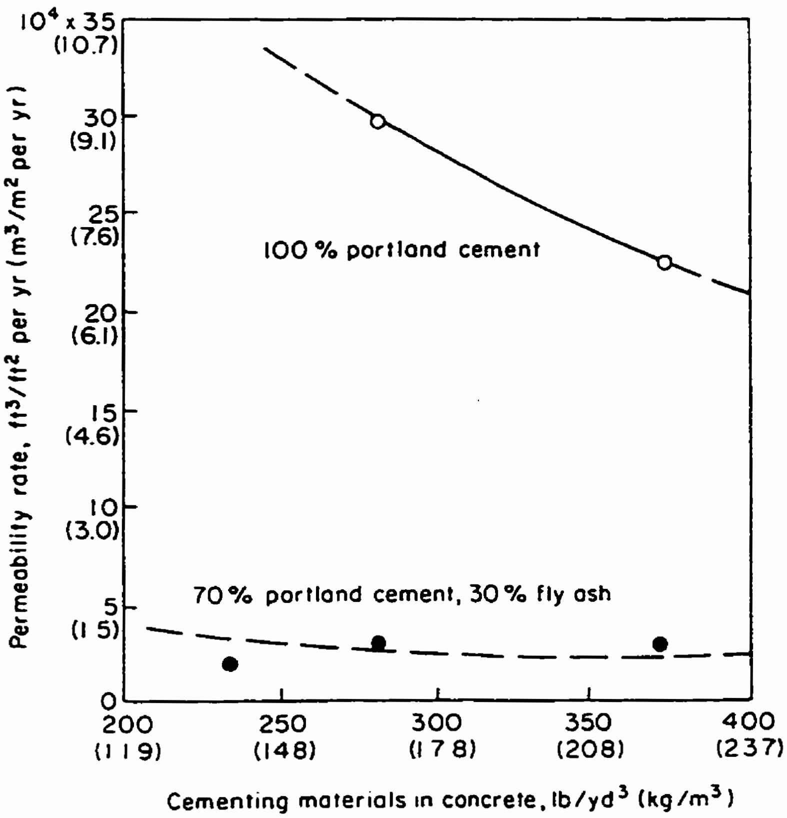


Figure 14 Permeability of Concretes Made With and Without Fly Ash (14)

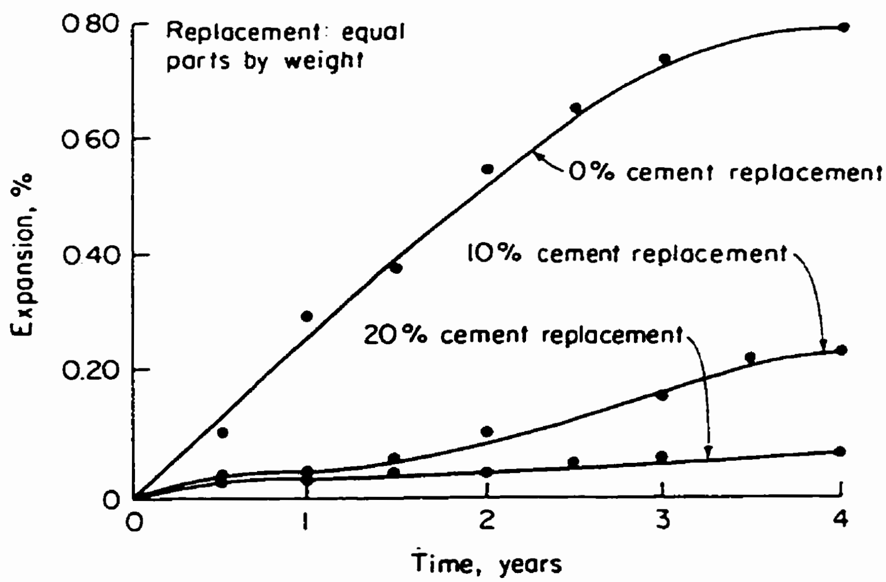


Figure 15 Use of Fly Ash to Control Alkali-Aggregate Reactions (14)

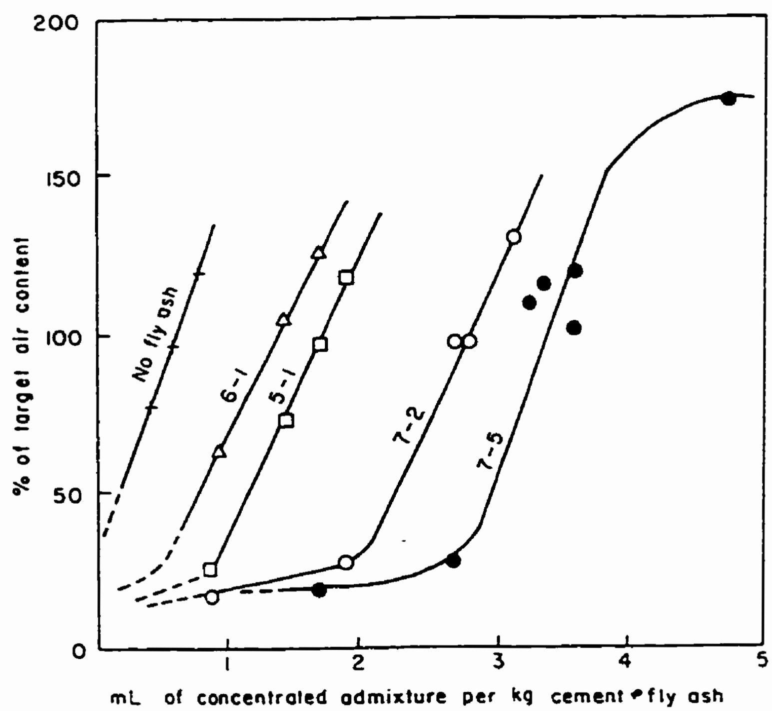


Figure 16 Typical Air Content Versus Air-Entraining Agent Dose Rate: Mortars With Four Different Fly Ashes (14)

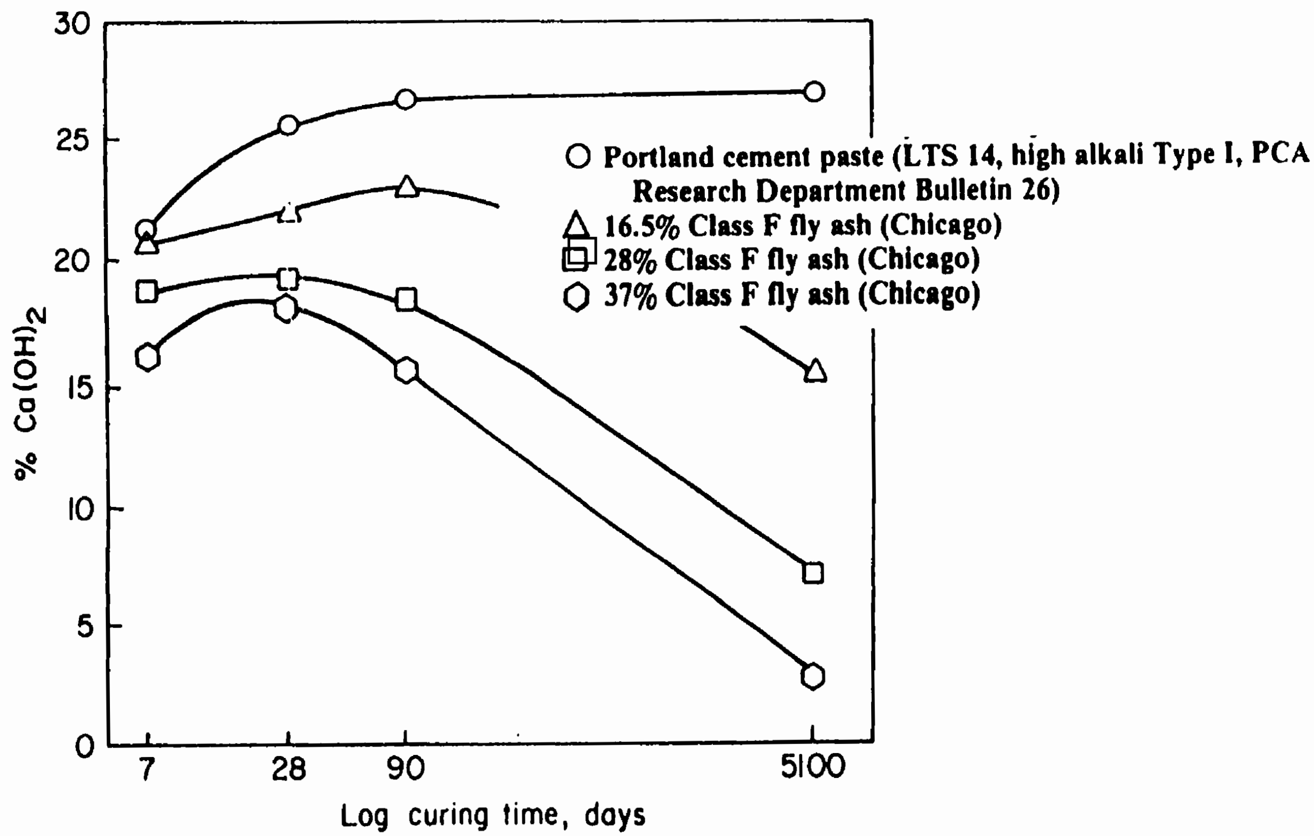


Figure 13 Changes in Calcium Hydroxide Contents of portland and Pozzolanic Cement Pastes During 14 Years Moist-Curing (14)

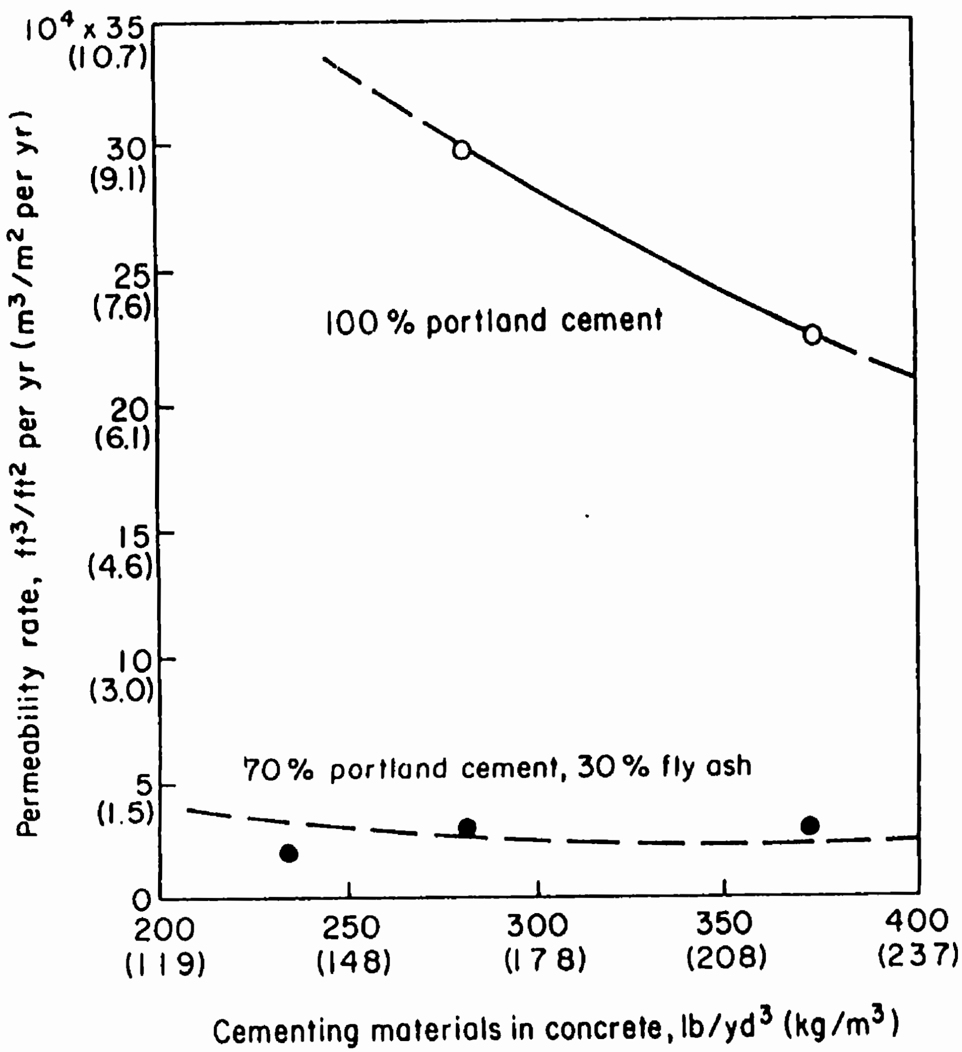


Figure 14 Permeability of Concretes Made With and Without Fly Ash (14)

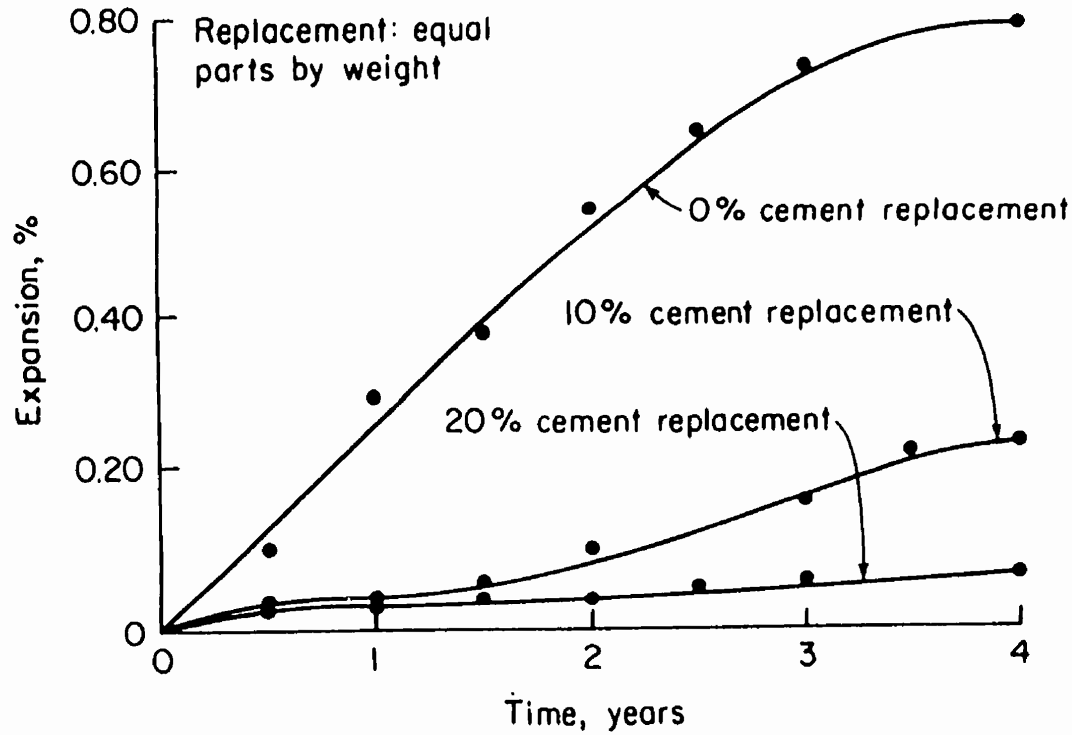


Figure 15 Use of Fly Ash to Control Alkali-Aggregate Reactions (14)

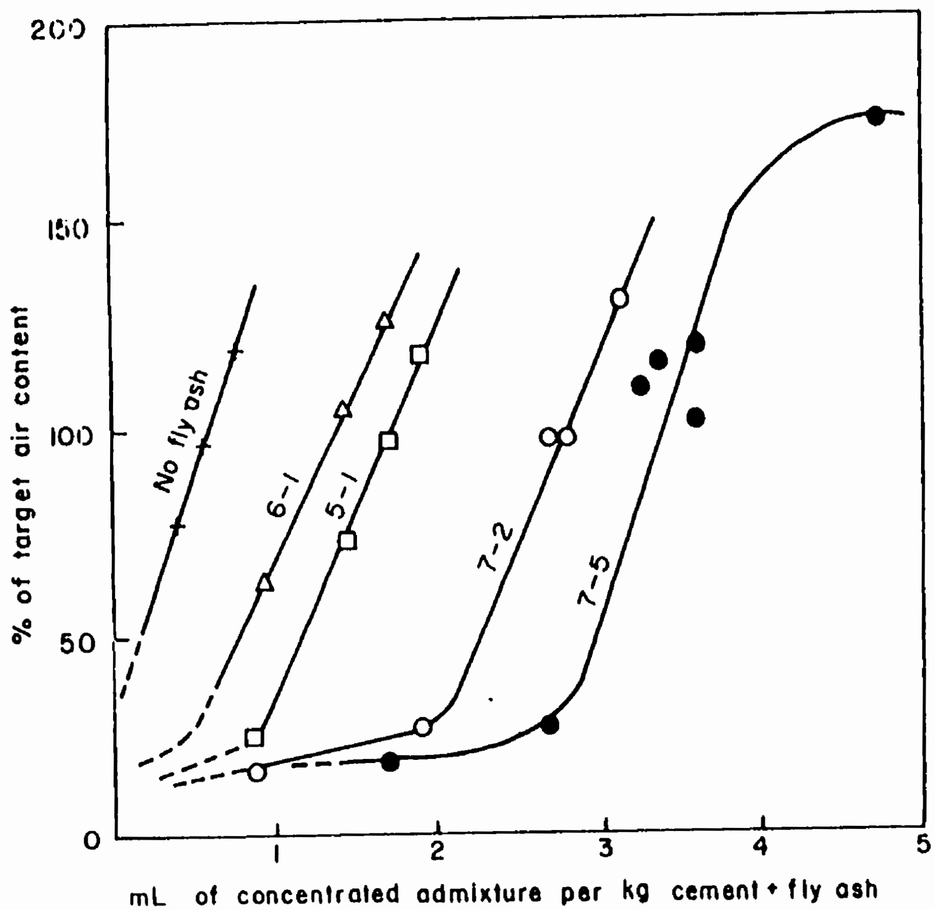


Figure 16 Typical Air Content Versus Air-Entraining Agent Dose Rate: Mortars With Four Different Fly Ashes (14)

APPENDIX C:

HIGH-PERFORMANCE CONCRETE MIXTURE DESIGN AND TESTING:

RESULTS (GRAPHS)

CONTENTS:

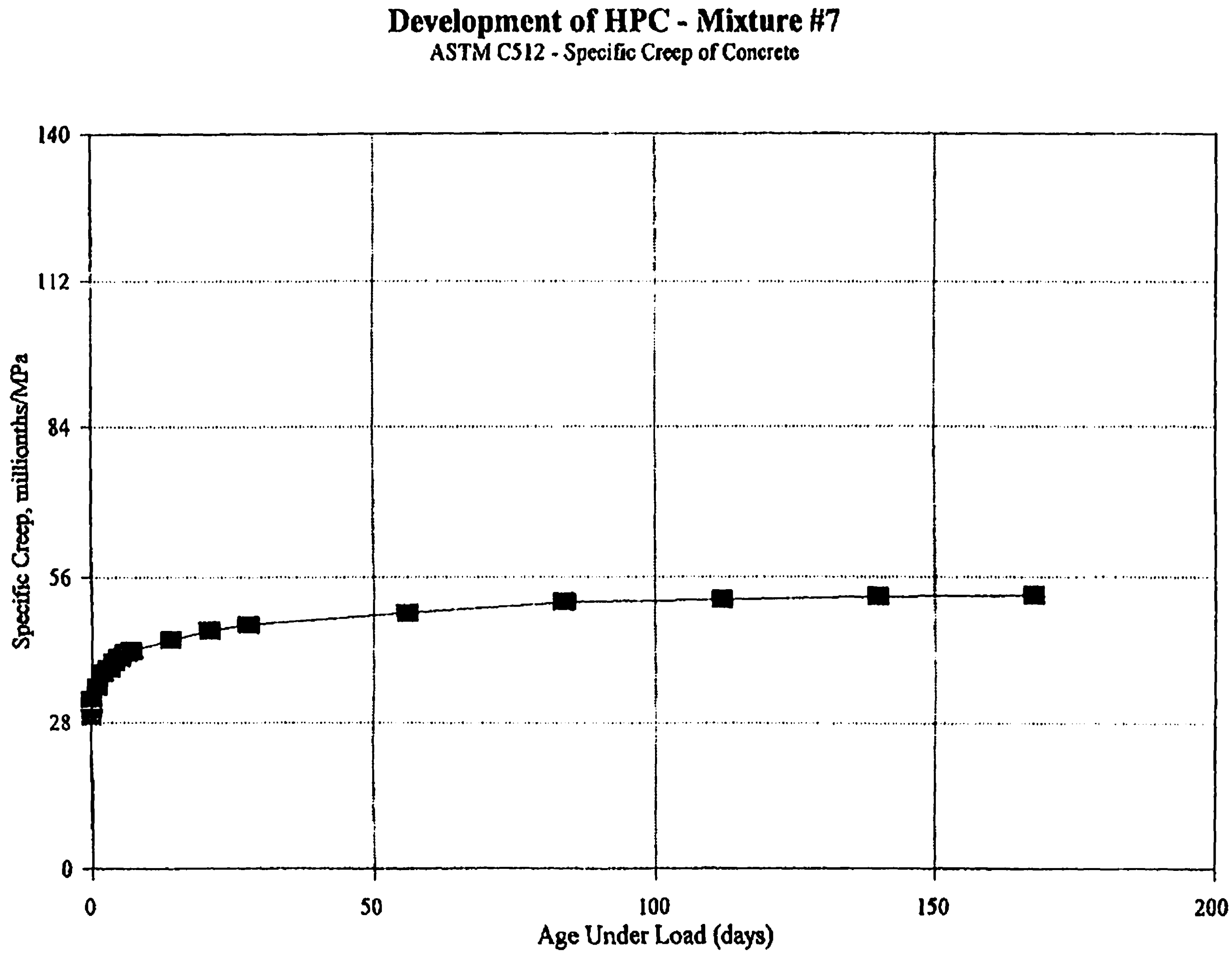
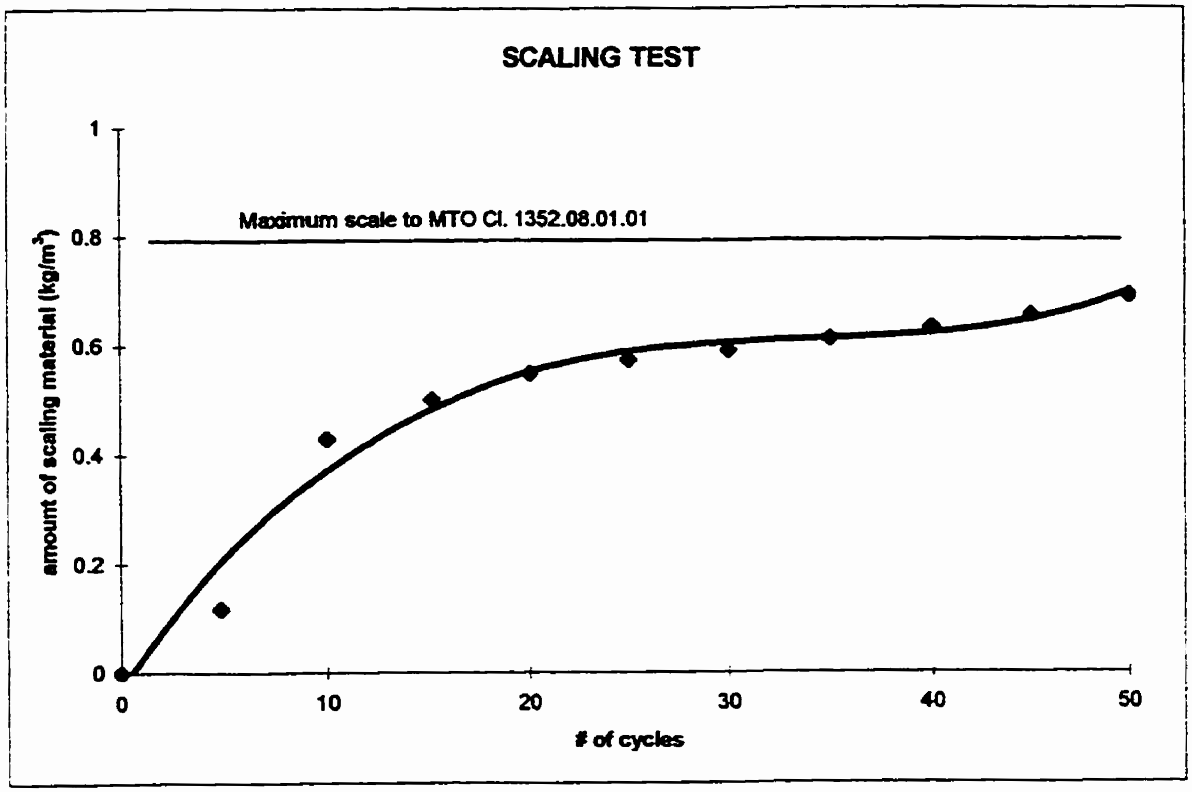


FIGURE C.1: Specific creep vs. age

RESULTS OF SCALING TEST ASTM 672 - 91a

HIGH PERFORMANCE BRIDGE - MIXTURE #1 (7)

Note: Values are based on an average of two plates and are rated to an amount in kg/m².



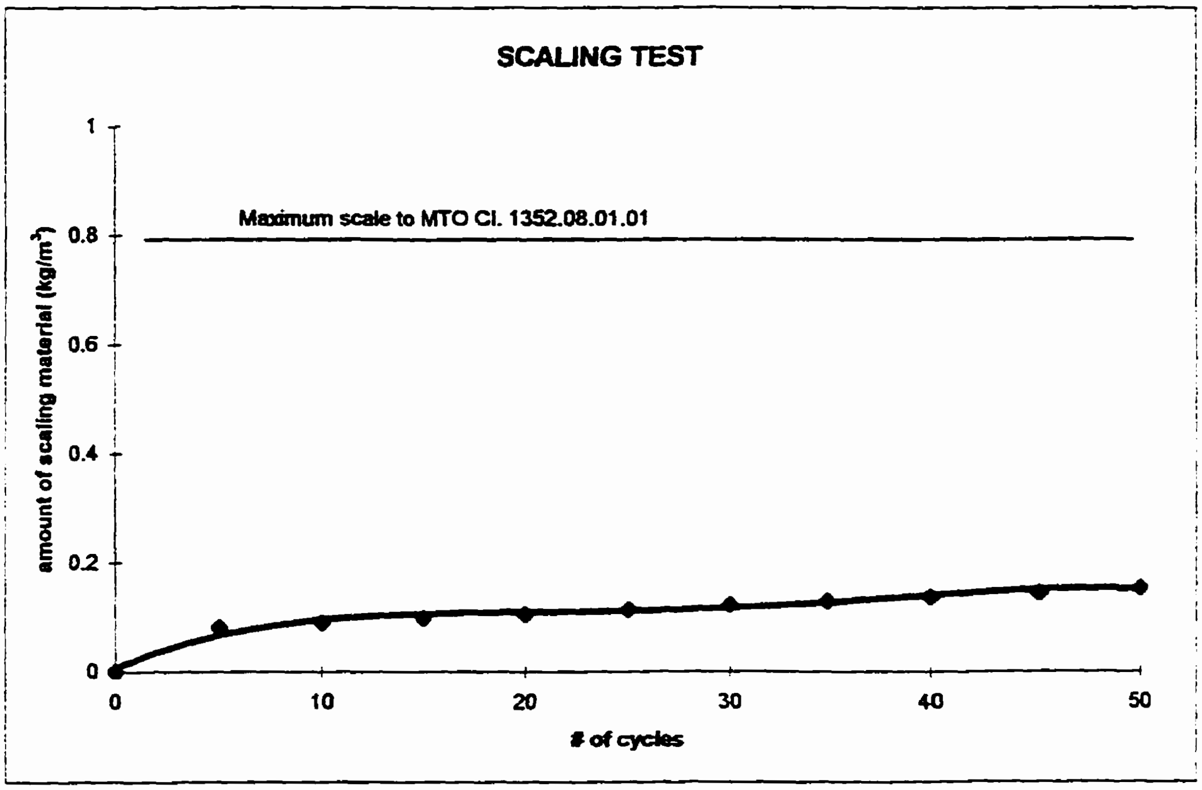
Scaling Ratings to ASTM Standard 672, Cl. 10.1.5		
Rating 0	-	no scaling
Rating 1	-	very slight scaling
Rating 2	-	slight to moderate scaling
Rating 3	@ 5 cycles	moderate scaling
Rating 4	@ 10 cycles	moderate to severe scaling
Rating 5	N/A	severe scaling

FIGURE C.2: Scaling test, Mixture #7

RESULTS OF SCALING TEST ASTM 672 - 91a

HIGH PERFORMANCE BRIDGE - MIXTURE #3 (10)

Note: Values are based on an average of two plates and are rated to an amount in kg/m².



Scaling Ratings to ASTM Standard 672, Cl. 10.1.5		
Rating 0	-	no scaling
Rating 1	@ 5 cycles	very slight scaling
Rating 2	@ 40 cycles	slight to moderate scaling
Rating 3	N/A	moderate scaling
Rating 4	N/A	moderate to severe scaling
Rating 5	N/A	severe scaling

FIGURE C.3: Scaling test, Mixture #10

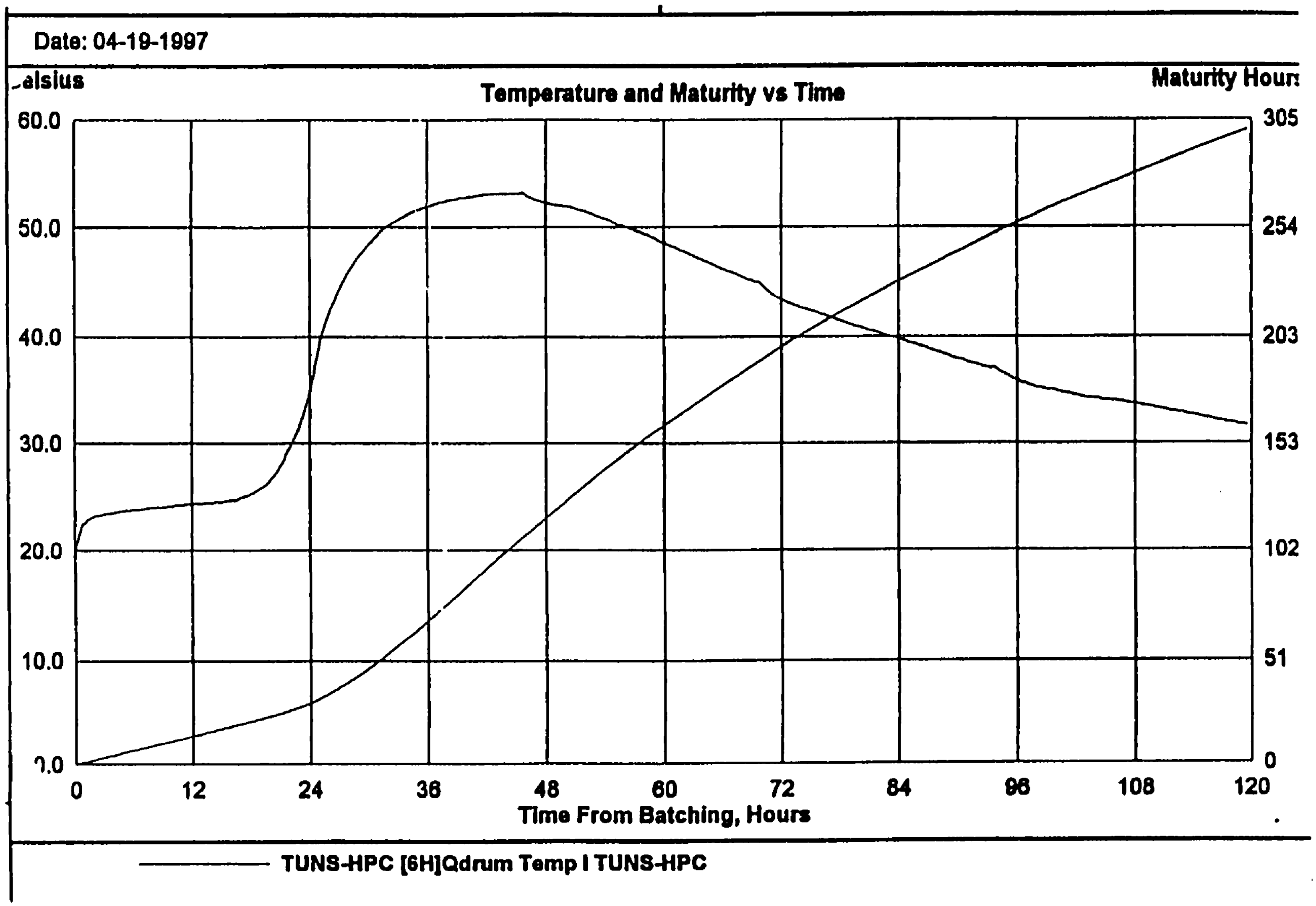


FIGURE C.4: Adiabatic heat development:
- temperature and maturity vs. time, Mixture #7

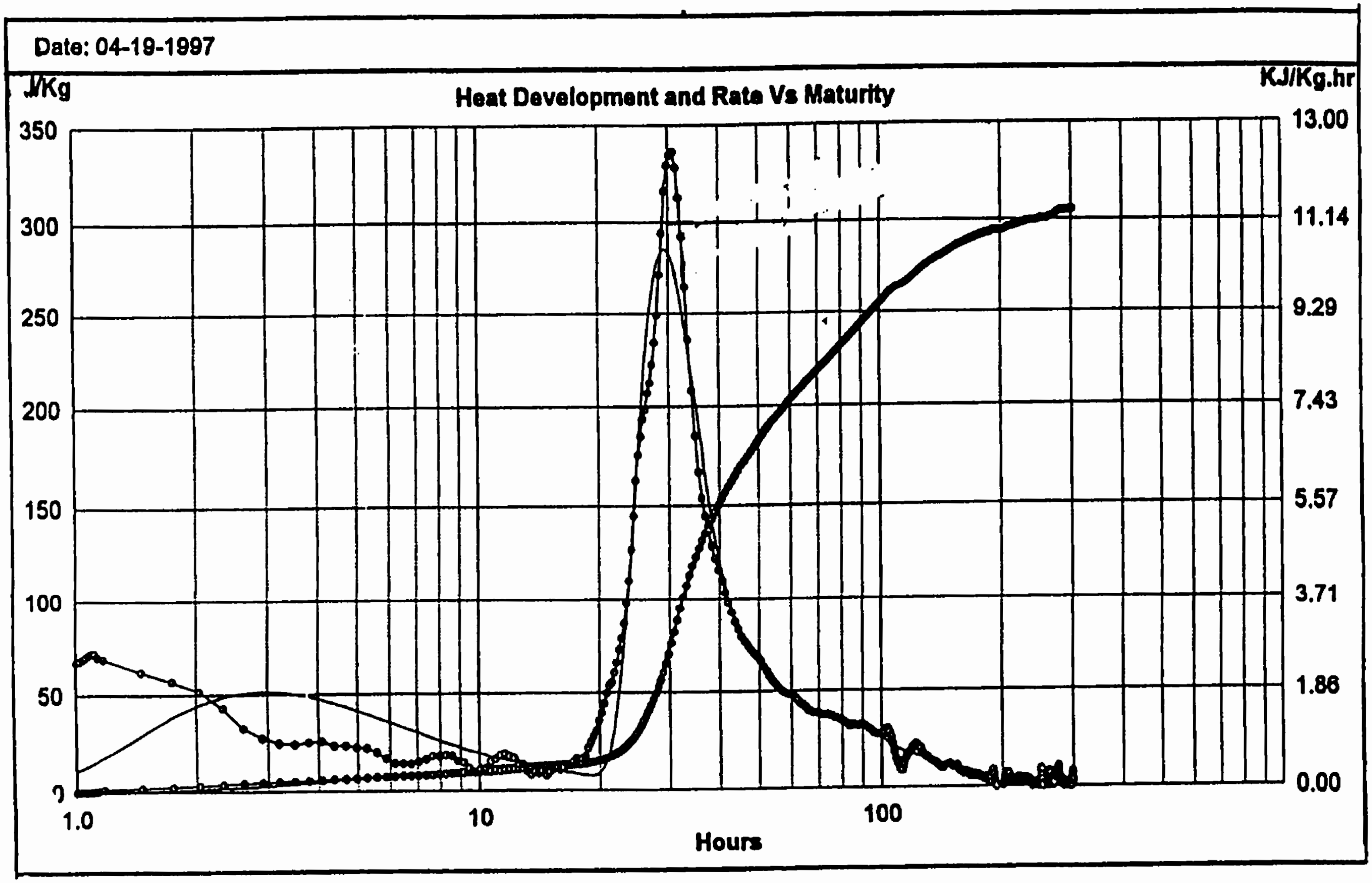


FIGURE C.5: Adiabatic heat development:
- heat development vs. time, Mixture #7

APPENDIX D:

BRIDGE SPECIFICATIONS:

SPECIAL PROVISIONS FOR
MATERIALS AND CONSTRUCTION
OF IN-SITU AND PRECAST CONCRETE

NSDoT PROJECT NO. 97-029

50 mm square metal posts as directed by the Engineer.

More than one small sign may be located on a single post and small signs may be located on posts of larger signs, but in both these cases, the unit bid price for erection of a particular type of sign will include all signs located on the particular sign post(s) in question.

Erection of Regulation Signs for this contract will be paid per sign installed and shall be full compensation, including all materials, labor, equipment and incidentals necessary to complete the work to the satisfaction of the Engineer.

ITEM 8.55:

REMOVE AND SALVAGE CORRUGATED METAL PIPE:

The contractor is advised that Division 5 Section 12 of the new standard specification shall be adhered to on this contract except as modified below.

The contractor will remove the corrugated steel pipe at locations determined by the Project Engineer. The pipe will be salvaged by the contractor and placed as per Division 5 Section 12 of the new standard specifications.

If the pipe is damaged by the contractor during the salvage operation, it will be the responsibility of the contractor to replace the pipe.

Unless otherwise approved by the Engineer the REMOVE & SALVAGE STEEL PIPE for this contract will be paid as lump sum of Steel Pipe Removed and Salvaged. The unit bid lump sum price of Remove and Salvage of Steel Pipe shall be full compensation, including all materials, labor, equipment and incidentals necessary to complete the work to the satisfaction of the Engineer.

ITEM 8.57:

TRANSPORTATION AND PLACEMENT OF PORTABLE JERSEY BARRIER:

Portable Jersey Barrier shall be used on this contract for traffic control purposes and/or for the protection of open excavations. The portable Jersey Barrier shall be supplied by the Department and shall remain the property of the Department at the conclusion of the contract. The Contractor shall make every effort to ensure that no units are damaged while being transported and/or relocated. Should any units be severely damaged or destroyed by the contractor during the transportation or relocation operations, they shall be replaced at the Contractor's expense.

The Portable Jersey Barrier shall be picked up at local DOT&PW Depots and returned to the same locations following completion of that part of the contract requiring its use. The Units may be at the Maclellans Brook Deot, and/or at the Amherst Depot.

Transportation and Placement of Portable Jersey Barrier will be paid for at the contract unit bid price per metre which shall be full compensation for the loading, unloading, transportation, placement, return and unloading of the barrier at its original location as directed by the Engineer. This price shall also include all equipment, plant, labour, tools and incidentals necessary to complete the work to the satisfaction of the Engineer.

ITEM 8.58

RELOCATION OF PORTABLE JERSEY BARRIER:

This item shall include the removal, temporary storage and relocation of existing portable jersey barrier where and as directed by the Engineer.

Payment for this item will be made at the contract unit price per metre for Relocation of Portable jersey barrier which price shall be full compensation for the supply of all equipment, plant, labour and incidentals necessary to complete the work to the satisfaction of the Engineer.

SECTION 5 SPECIAL PROVISIONS

The Contractor is advised that the following Special Provisions refer to Section 5 of the Contract.

Section 5.0: WEST RIVER EAST SIDE ROAD STRUCTURE, Construction of a two span structure with Prestressed Girders utilizing Highway Performance Concrete for all concrete elements of the structure.

ITEM 5.7.2 - CAST IN PLACE - HIGH PERFORMANCE CONCRETE

1.0 DESCRIPTION

This section details the requirements for materials and methods in the proportioning, mixing, transport, placement, finishing and inspection of Cast in Place High Performance Concrete (HPC).

2.0 REFERENCES

All reference standards shall be current issue or latest revision at the first date of tender advertisement. This specification refers to the following standards, specifications or publications:

- Division 5 Section 7, Cast in Place Concrete
- ASTM C1202, Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration

3.0 SUBMISSIONS AND DESIGN REQUIREMENTS

Submission and design requirements shall be governed by the provisions found in Division 5 Section 7 of the Standard Specification, except that additional requirements as specified herein shall apply.

3.1 Ready Mix Concrete Supplier. At tender closing the Contractor shall advise the Department of the qualified ready mix concrete supplier, sources of fine and coarse aggregate, cement, fly ash and admixtures proposed for the project. The Contractor shall not be permitted to change the concrete supplier or alter mixture proportions without written permission from the Department.

3.2 Mixture Proportions/Test Requirements. All mixture proportions shall be selected on the basis of an 80 year design life of the structure. In addition to the test requirements found in Division 5 Section 7, the Contractor shall perform all necessary tests indicated in this specification to demonstrate the long term performance and durability of the materials and concrete mixtures. The HPC mixture shall have a minimum compressive strength of 60 MPa in 28 days.

3.3 High Performance Trial. A trial slab shall be constructed prior to placing concrete in the deck. The Contractor shall cast a concrete slab 8 m wide by 9 m long by 215 mm thick at a site identified by the Contractor and acceptable to the Department. The slab shall be placed, finished, textured and cured as required by the contract documents using the same methods, personnel and equipment to be used in the work.

4.0 MATERIALS

4.1 Aggregates. The fine and coarse aggregates shall be normal density and conform to the requirements of CSA A23.1, except as modified herein. Upon acceptance of the aggregates, the source and method of manufacture shall not be altered for the duration of the contract. Aggregates shall be stored and maintained in such a manner to avoid the inclusion of foreign materials in the concrete and such that no equipment will be operated on the storage piles. The stockpiles shall be constructed to prevent segregation or contamination. Prior to the start of any concrete placement, the ready mix concrete supplier shall have at the place of production, sufficient quantity of aggregates to complete the entire concrete section scheduled for that day.

4.1.1 Fine aggregate. Fine aggregate shall be washed and classified to conform to the gradation limits specified in CSA except that the amount of material finer than 80 um shall not exceed 1.7%. The fineness modulus of fine aggregate shall be between 2.60 and 2.90.

4.1.2 Coarse Aggregate. Coarse aggregate shall consist of washed crushed stone having a nominal maximum size of 20 mm. Coarse aggregate shall be non-reactive when tested in accordance with CSA A23.2-14 or 25A. The maximum combination of flat, elongated and flat and elongated particles, as defined in CSA A23.2-13A, shall not exceed 10% of the mass.

4.2 Water. Water used in concrete production and curing shall conform to CSA A23.1 and be clean and free from injurious amounts of oil, acid, alkali soluble chlorides, organic matter, sediment or any deleterious substances.

4.3 Admixtures. Air entraining admixtures shall meet the requirements of ASTM C260. Other chemical admixtures shall meet the requirements of ASTM C494. Admixtures shall be stored above freezing temperatures at all times and in accordance with the manufacturer's recommendations. Calcium chloride or any admixtures containing chlorides shall not be used.

4.4 Cement. The cement shall be Type 10SF (low alkali) meeting the requirements of CSA A362. The silica fume content shall be between 7.5 and 9.5 percent of the total cement mass. The tri-calcium aluminate content shall be between 6 and 10 percent. The alkali content as Na_2O equivalent shall be less than 0.6 percent. Heat of hydration shall be less than 350 kJ/kg at 7 days.

4.5 Supplementary Cementing Materials. Fly ash shall be produced by the combustion of pulverized coal shall be Type F and meet the requirements defined in CSA A23.5. The total sum of the silica fume + fly ash must not exceed 25 percent of the total cementitious content.

4.6 Curing Materials. Curing compounds shall conform to ASTM C309. Burlap shall be free from holes, clay or other substances which would have a deleterious effect on concrete.

4.7 Evaporation Reducer. Evaporation reducer forming a monomolecular film which retards evaporation, shall be "Confilm" by Master Builders or an approved equal. The mixing ratio for Confilm shall not be less than one part of evaporation reducer to four parts of water.

4.8 Concrete. All mixture designs shall be proportioned as normal density concrete in accordance with CSA A23.1 , Alternative #1, and the Contractor shall accept responsibility for the concrete properties. Concrete shall be proportioned using Type 10 SF (low alkali) cement, Class F fly ash, fine and coarse aggregates and the following admixtures:

- air entraining
- water reducer and/or retarder
- superplasticizer

The Contractor may select the concrete mixture included within this specification or submit an alternate concrete mixture for use. Acceptance of the mixture design included within this specification does not relieve or reduce the Contractor's responsibility for the properties of the concrete mixture.

Concrete Mixture Proportions	Quantity
Cement Type 10SF (low alkali) (kg/m^3)	450
Fly ash Class F (kg/m^3)	30
Will-Kare fine aggregate (kg/m^3)	690
Will-Kare coarse aggregate (kg/m^3)	1045
Water (L/m^3)	144
Darex EH (mL/100 kg cement)as required	200*
Daratard 17 and/or WRDA-82 (mL/100 kg cement)	250*
WRDA 19 (mL/m^3)	3500
Water/cementing materials ratio	0.30

*Quantities of water air entrainment, reducer and/or retarder shall be adjusted to suit daily conditions.

Alternate Design:

If the Contractor submits an alternate concrete mixture, in addition to the tests outlined in Division 5 Section 7, additional test data as indicated below must be included prior to acceptance.

- Plastic Concrete Tests
  Slump (CSA A23.2-5C)
  Air Content of Plastic Concrete by Pressure Method (CSA A23.2-4C)
  Unit weight and Yield (CSA A23.2-6C)
- Compressive Strength Testing (CSA A23.2-9C)
  3 cylinders to be tested at 7 days
  3 cylinders to be tested at 28 days
- Air Void Analysis on Hardened Concrete(ASTM C457) tested at 7 Days
- Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration (ASTM C1202) tested at 28 Days

Concrete mixtures shall be proportioned to meet the following:

Minimum cementing materials content..450 kg/m^3

Maximum water/cementing materials ratio..0.34
Nominal size coarse aggregate..20 mm
Maximum slump before superplasticizer..60 mm
Slump after superplasticizer..190 +/- 30 mm
*Air content at point of discharge from truck....................................7 +/- 1%
Minimum 28 day compressive strength...60 MPa
Maximum spacing factor of hardened concrete not to exceed..........260μm
*Average spacing factor of hardened concrete not to exceed...........230μm
Rapid chloride permeability (91 days)...< 600 coulombs
Maximum concrete temperature(from delivery equipment)
thickness > 2 metres..18°C
thickness < 2 metres..25°C
Max. concrete temperature(insitu)..70 °C
Maximum temperature gradient...20 °C/metre

*It may be necessary to design the mixture to have a maximum spacing factor of 150μm at the point of discharge from the truck to accommodate air void degradation during placing and to meet the above limits.

5.0 CONSTRUCTION

Concrete shall be mixed, transported, placed and finished in accordance with Division 5 Section 7 except as modified herein.

Concrete shall not be placed when the air temperature exceeds 25°C or is likely to rise above 25°C during placement. The temperature of the formwork, reinforcing steel or other material on which concrete is to be placed shall not exceed 25°C. The maximum concrete temperature at the point of discharge shall not exceed the temperatures specified in Section 4.10 of this specification.

Superplasticizer shall be added on site and shall be used in all concrete.

Concrete placing methods and equipment shall be such that the concrete is conveyed and deposited at the specified slump, without segregation, and without changing or affecting the other specified qualities of the mixture. The maximum volume of concrete delivered to the site shall not exceed 75 percent of the mixers rated capacity. A minimum of two vibrators shall be used to consolidate the concrete in all phases of construction.

The deck shall be finished using a mechanical screed machine followed by bullfloating and final texturing. Final finishing, texturing and curing shall be completed within 1.5 metres behind the screed machine. A work bridge(mobile catwalk) shall be used following the screed machine for bullfloating and finishing operations. An evaporation reducer shall be used directly after initial screeding and/or between finishing operations as needed to aid in bullfloating and final texturing.

The finishing machine shall be self-propelled and travel on rails. It shall be fitted with a rotating cylinder screed, an adjustable powered screw auger and a vibrator mounted in front of the screed. It shall be capable of forward and reverse movement under positive control. There shall be provision for raising all screeds to clear the screeded surface without adjusting the legs. It shall also be provided with a locking device at each leg to prevent vertical adjustment. The finishing machine shall be capable of obtaining an acceptable surface texture without excessive additional hand finishing.

A work bridge riding on the screed rails behind the finishing machine with a working platform not higher than 0.4

m above the finished surface, shall be provided to facilitate hand finishing work, concrete inspection, and placing of curing materials. On placements longer than 40 m or wider than 10 m, a second work bridge shall be provided. When two work bridges are required, the trailing work bridge shall ride on the screed rails and shall be used for the purpose of placing the curing materials and shall have sufficient clearance to allow for the proper placing of the curing materials. Screed rail chairs shall be adjustable in height and made of metal.

After bullfloating, the deck shall be textured with a wire broom or comb having a single row of tines. The required texture shall be transverse grooves between 1.5 to 4.5 mm in width at centers between 15 and 20 mm respectively. The depth of grooves shall be between 3.0 to 4.5 mm.

Curing compound (water based) shall be applied at twice the manufacturers suggested rate to the deck surface immediately after texturing, within 20 minutes of initial screeding and prior to covering with burlap. The second application of curing compound shall be applied at a 90 degree angle from the first. Burlap shall be presoaked by immersing in water for a period of 24 hours prior to placing. Burlap shall be applied immediately after initial set of the concrete over the curing compound. Two layers of burlap shall be applied to the surface overlapping each strip by 150 mm. Burlap shall be maintained in a continuously wet condition for seven consecutive days. Burlap shall be covered with a layer of moisture vapour barrier immediately following the placement of the burlap.

If formwork is used to aid curing, it shall not be removed until five days after the concrete placement. In addition to the forms, areas exposed shall be cured in accordance with Division 5 Section 7. Where bonding is critical between finished surfaces, concrete sections shall be moist cured only as described above for the deck.

5.1 Instrumentation: Researchers shall have free access for the purposes of installing instrumentation and monitoring deck concrete at all times. These instruments will be used to monitor steel corrosion, the internal temperature and concrete deck deflection due to creep, shrinkage, and live loads.

6.0 QUALITY CONTROL / QUALITY ASSURANCE

The Department or it's representative shall have the right to sample and test all materials used in the mixture design and given access to the production facilities of the ready mix concrete supplier. Materials failing to meet this specification shall be immediately rejected.

Concrete shall be tested as per Division 5 Section 7 except as modified herein.

Concrete shall be tested for slump, air content and temperature prior to and after the addition of superplasticizer. Testing shall be carried out at the point of discharge from the truck and as close as possible to final deposit into the forms. Sufficient superplasticizer shall be added to produce a consistency as indicated in Section 4 of this specification. Superplasticizer shall be added on site and mixed a minimum of 5 minutes prior to retesting.

Concrete shall also be randomly tested for:

- Mass density and yield
- Compressive Strength (six cylinders per set) tested at the following ages:
  1 at 3 days
  1 at 7 days
  2 at 28 days
  2 at 91 days

- Microscopic Determination of Air Void Content and Parameters

→Rapid Chloride Permeability.

7.0 METHOD OF MEASUREMENT

7.1 General. Measurement shall be made at the contract unit price per cubic metre for HPC. The quantity of concrete for payment shall be the volume in cubic metres computed from the dimensions shown on the drawings or as revised by the authority of the Engineer. No deductions shall be made for the volume of concrete displaced by steel reinforcement, joint material, structural shapes, chamfers, tops of piles, or cylindrical voids of 110 mm diameter or less.

8.0 BASIS OF PAYMENT

8.1 General. Payment will be made at the contract unit price bid per cubic metre for HPC. The payment for HPC shall be considered full compensation for the high performance trial, cost of furnishing all materials, aggregates, cement, supplementary cementing materials, water, admixtures, including superplasticizers, and other materials, non-metallic expansion joint materials, tools, equipment, false work, forms, bracing, labour, curing, surface finish, damp-proofing and all other items of expense required to complete the concrete work as shown on the plans, and as outlined in the specifications.

8.2 Payment for Cold Weather Concreting. If it is considered necessary by the Department to place concrete in cold weather, additional payment for Cold Weather Concreting will be paid. Payment shall be as detailed in Division 5 Section 7.

9.0 WARRANTY

ITEM 5.8 - PRECAST/PRESTRESSED HIGH PERFORMANCE CONCRETE GIRDERS

1.0 DESCRIPTION

This section details the requirements for materials and methods in the proportioning, mixing, transport, placement, finishing and inspection of Precast/ Prestressed High Performance Concrete Girders.

2.0 REFERENCES

All reference standards shall be current issue or latest revision at the first date of tender advertisement. This specification refers to the following standards, specifications or publications:

- CSA/CAN3-A23.4 - Precast Concrete Materials and Construction
- Division 5 Section 8, Precast, Prestressed Concrete Girders
- Cast in Place High Performance Concrete
- CSA A251, Qualification Code for Manufacturers of Architectural and Structural Precast Concrete

3.0 SUBMISSIONS AND DESIGN REQUIREMENTS

Submission and design requirements shall be governed by the provisions found in Division 5 Section 8 of the Standard Specification, except that additional requirements as specified herein shall apply.

3.1 Fabrication of Girders. At tender closing the Contractor shall advise the Department of the girder supplier proposed for the project. Precast facilities are to be certified as per the requirements of CSA-A251. The girder supplier shall utilize the same mixture design proposed by the Contractor for Cast in Place High Performance Concrete. The Contractor shall not be permitted to change the girder supplier or alter mixture proportions without written permission from the Department.

3.2 Mixture Proportions/Test Requirements. All mixture proportions shall be selected on the basis of a 80 year design life of the structure. In addition to the test requirements found in Division 5 Section 8, the Contractor shall perform all necessary tests indicated in this specification to demonstrate the long term performance and durability of the materials and concrete mixtures. The concrete mixture shall have a minimum compressive strength of 65 MPa in 28 days for Precast/Prestressed High Performance Concrete Girders.

4.0 MATERIALS

All materials used in the manufacture of precast, prestressed concrete girders shall conform to the requirement found in Division 5 Section 8 except as modified herein:

4.1 Concrete

All mixture designs shall be proportioned in accordance with CSA A23.1, Alternative #1 and the Contractor shall accept responsibility for the concrete properties. Concrete shall be proportioned using Type 10 SF(low alkali) cement, Class F fly ash, fine and coarse aggregate, air entraining, water reducing and superplasticizing admixtures. Mixture proportions and all materials used in the production of concrete shall conform to the requirements found in Cast in Place High Performance Concrete.

4.2 Prestressing Strands

Prestressing strands shall comply to Division 5 Section 8 except that the prestressing strand shall be stabilized having a nominal diameter of 16 mm and an ultimate tensile strength of 1860 MPa unless otherwise specified.

5.0 CONSTRUCTION

The manufacture of the precast prestressed concrete girders shall be in accordance with CAN3-A23.4 "Precast Concrete Materials and Construction" except as modified herein:

5.1 Concrete Curing

All concrete shall be cured with water. If curing blankets are used, they must be covered with polyethylene sheets, keep saturated at all times and not allowed to dry-out. Girders shall be maintained continuously wet at a controlled temperature until release strength is obtained. The Contractor shall submit a curing plan/procedure to the Engineer for approval.

6.0 QUALITY CONTROL/QUALITY ASSURANCE

Quality control/quality assurance shall be conducted as indicated in Division 5 Section 8 of the Standard Specifications.

7.0 METHOD OF MEASUREMENT

8.0 BASIS OF PAYMENT

Payment shall be made at the contract lump sum bid price for High Performance Precast Prestressed Concrete Girders. This price shall be full compensation for all labour, materials, plant and services necessary to manufacture, deliver and erect the girders in the final position, as shown on the shop drawings, and to the satisfaction of the Engineer. This price shall also include any additional mixture designs and testing if applicable.

ITEM 8.1 - BRIDGE BEARINGS:

This Item is intended to cover both the supply and installation of all bearings as detailed on the Drawings or approved equal.

Bridge bearings shall be elastomeric bearings as shown on the Drawings. All bearings shall conform to the latest edition of C.S.A. S6-M88, Design of Highway Bridges. Steel components shall be to C.S.A. G40.21-300W.

All exposed steel surfaces shall be zinc metallized, (minimum thickness of 175 um) in the shop after fabrication in accordance with C.S.A. G189 - Sprayed Metal Coatings for Atmospheric Corrosion Protection, or hot dipped galvanized in accordance with C.S.A. Standard G164.

Payment for this Item will be made at the contract price tendered per bearing assembly which price shall include the installation and supply of all material and associated components for the bearing assemblies including lateral restraint angles, dowels, and all anchor bolts, nuts and washers.

Partial payment equal to one hundred (100%) percent of the invoice price for this Item shall be made to the Contractor when these units have been delivered and properly stored at the bridge site. The Contractor shall be responsible for the safe keeping of these units during transportation and after delivery at the site of work until the complete structure has been accepted by the Department. The Department shall not be liable for the replacement of any units or part thereof if such becomes necessary at any time during this contract.

ITEM 8.2 - JOINT SEAL ASSEMBLIES:

This Item shall cover the fabrication and delivery to the site of all joint seal assemblies required for the deck of the bridge as well as their installation, all as shown on the Drawings and as specified herein.

The gland shall be of the types known as the strip or pressure seal. It shall be fabricated of neoprene and shall be in accordance with ASTM D2628 modified. The gland is to be factory installed in one continuous length. Should a splice be necessary, it is to be located as directed by the Engineer as determined by the geometries of the bridge. Edge beams shall be either extruded, rolled formed or an approved equivalent. They, and other steel parts shall be equal to or better than C.S.A. G40.21M-300W. All steel surfaces shall be zinc metallized, (minimum thickness of 175 um) in the shop after fabrication in accordance with C.S.A. G189 - Sprayed Metal Coatings for Atmospheric Corrosion Protection, or hot dipped galvanized in accordance with C.S.A. Standard G164.

Proprietary systems which are considered by the Department to be in accordance with these requirements are the "ZT" system, manufactured by Z-Tech Incorporated, the Elastometal Acme Strip Seal or Pressure Seal Systems, the Honel "C" System and GSH System manufactured by Wercholoz Associates Limited, as well as the Steelflex System manufactured by D.S. Brown Company Limited.

Details of these systems shall be in accordance with the latest edition of the manufacturer's technical bulletin, unless otherwise approved by the Engineer.

Alternate systems of sealing the joints may be proposed by the Contractor provided they satisfy the requirements of these Specifications and are approved by the Engineer. The supplier must provide evidence of possessing expertise in the fabrication of joint seal assemblies. In addition, such alternatives must be accompanied by sufficient technical data and evidence of prior satisfactory performance to allow a proper assessment of their capabilities to be made.

Joint seal assemblies shall be firmly fastened to the traffic deck by means of a series of steel plates complete with a loop anchorage system while anchorage on the curbs and sidewalks can be provided by means of KSM studs.

The Contractor shall provide the Engineer, for approval prior to fabrication, drawings showing details of the assembly he intends to use. Drawings for the expansion joints shall contain a movement chart showing all anticipated movements of the structure and the required setting widths of the joint at various temperatures. The sealing units must extend the full width of the roadway and also provide a continuous watertight seal for the curbs and sidewalks. It is the responsibility of the Contractor to ensure that the sealing units are constructed to the exact dimensions necessary.

All welding required in these assemblies shall be carried out by a firm certified by the Canadian Welding Bureau in accordance with the requirements of C.S.A. Standard W47.1, "Certification of Companies for Fusion Welding of Steel Structures for Division 1 or for Division 2". They shall have the organization, personnel, welding procedures and equipment required to produce satisfactory welds consistent with good engineering practice.

Joint seals shall be water tight over the full range of movement of the joint opening from -29°C to +38°C. Installation of the seals shall be done under the supervision of representatives of the manufacturer and the Engineer and all movement due to drying, shrinkage and creep of the concrete as well as the ambient temperature at the time of installation shall be considered in setting the initial width of the joint opening.

The elastomeric parts (and steel screws and bolts, if required) shall be replaceable easily and quickly with a minimum of inconvenience to vehicular traffic, and without the cutting out or demolition of concrete.

The manufacturer of the seals shall provide the Department with a written certificate guaranteeing the proper functioning and the water tightness of all the assemblies installed for a two year period following completion of the bridge, and shall hold themselves available to repair, replace and made good any and all damages sustained in this period caused by normal use (including snow removal operations) and wear that may have occurred, at no cost to the Department.

No final payment will be made under this Item until the required certificate as described herein has been received by the Department.

PAYMENT

Payment for joint seal assemblies will be made under this Item at the contract bid price per metre which shall be full compensation for the furnishing of all materials, equipment, labour and services necessary to complete the work in accordance with the specifications and drawings and to the satisfaction of the Engineer.

The Contractor shall be responsible for the safe keeping of these units during transportation and after delivery at the site of work until the complete structure has been accepted by the Department. The Department shall not be liable for the replacement of any units or part thereof if such becomes necessary at any time during the execution of this Contract..

ITEM 8.4 - CRACK REPAIR:

At an age of 28 days, the concrete deck shall be surveyed by the Engineer to locate any visual evidence of cracking in the deck surface. The presence of any cracks will be recorded and evaluated. Those cracks identified from the survey and evaluation requiring remedial action shall be treated in accordance with the following methodology.

The cracks identified for repair shall be sealed with a low viscosity epoxy resin such as Sika Canada's Sikadur 52, or approved equal. The resin shall be applied by carefully pouring into the crack and allowing the sealant to be absorbed. A second application may be required, depending on the absorption and crack depth. The second application, if required shall be made as soon as possible after the first application has set. Wider cracks, as identified from the survey may require a higher viscosity resin for repair. The Contractor shall submit manufacturer's data for the proposed resin in this case. Excess resin in the vicinity of the crack may be sandblasted off the deck at the Engineer's direction.

Method of Measurement

Measurement shall be made at the concrete unit price per lineal metre of crack identified for repair.

Basis of Payment

Payment will be made at the contract unit price per lineal metre of crack repaired. The payment for remedial crack repair shall be considered full compensation for the surface preparation, initial sandblasting, crack filling with the specified sealant, and subsequent sandblasting, if required.

By my/our signature hereunder,	)	
I/we hereby identify this as	)	
the Special Provisions Speci-	)	Nova Scotia Department
fications referred to in this	)	of Transportation
contract executed by me/us and	)	and Public Works,
bearing date the.......day of.....	)	Halifax, Nova Scotia
...............19.........................	)	
	)	
....	)	
CONTRACTOR		

APPENDIX E:

HIGH-PERFORMANCE CONCRETE MIXTURE

CONSTRUCTION TESTING

RESULTS

CONTENTS:

NSDoT: HPC BRIDGE PROJECT - CONSTRUCTION TEST DATA

TEST RESULTS:

DATE CAST	LOCATION	CYLINDER STRENGTH (MPa)				SLUMP (mm)	AIR (%)
		3-day	7-day	28-day	91-day		
28/08	centre pier footing	30.0	41.8	55.9 56.5	60.9 64.5		
28/08		30.7	41.1	56.0 55.0	64.5 63.4		
29/08	retaning wall footing	27.7	38.5	52.4 52.1			
29/08		29.0	39.3	55.9 54.3			
02/09	centre pier column	24.7	31.8	45.3 44.8	52.4 50.5	160	8
05/09	retaining wall	31.2	44.0	61.5 60.0	65.2 64.5	180	7
05/09		29.3	38.2	55.4 54.9	63.2 63.0	170	8.2
08/09	retaining wall wing	34.2	44.7	58.7 61.5	66.8 70.8	160	7.4
08/09		32.2	43.6	59.5 55.6	67.8 70.3	205	7
10/09	pier cap	36.3	50.2	57.3 63.0	72.5 70.7	160	6
10/09	east abut. footing	32.2	48.0	64.4 59.5	73.8 76.9	190	7.1
15/09	east abut. beam seat	33.9	46.8	65.1 65.0	78.8 79.5	195	8
15/09		29.5	42.5	59.6 61.3	73.4 70.6	200	7
17/09	west abut. footing	24.8	48.8	71.4 69.2	78.1 80.4	220	7
17/09		32.1	48.4	65.9 66.2	77.2 77.4	200	7.2
22/09	west abut. beam seat	33.8	50.1	65.6 61.5		190	5.9
22/09		39.1	45.7	73.6 74.4		170	5.2
23/09	west abut. back wall	29.9	44.0	59.0 58.8		180	6.8

DATE CAST	LOCATION	CYLINDER STRENGTH (MPa)				SLUMP (mm)	AIR (%)
		3-day	7-day	28-day	91-day		
23/09		27.8	41.1	55.5 56.6		190	8.2
24/10	girder diagphrams	29.4	44.8	76.8 75.6		190	3
24/10		33.8	52.1	78.6 79.0		180	3.6
30/10	trial slab deck	27.7	36.9	58.4 58.2		190	4.6
30/10		33.0	46.0	70.4 68.5		95	6.2
03/11	girder diagphrams	32.2	42.2	65.6 61.9		170	8.5
03/11		41.3	51.7	74.5 72.3		140	6.4
10/11	abut. retaining wall	28.4	43.1	63.5 63.6		190	7.8
19/11		28.2	41.3	64.0 61.2		170	8
19/11		29.8	45.8	63.0 66.4		120	7.6
20/11	bridge deck	29.8	42.7			190	8
20/11		28.8	44.3			135	7.4
20/11			32.2			190	10.5
20/11		23.1	33.4			170	8.6

ANALYSIS:

DATE CAST day #	test #	AV. CYL. STRENGTH (MPa) - no. of days: 3	7	28	91	SLUMP (mm)	AIR (%)
1	1	30.0	41.8	56.2	62.7		
1	2	30.7	41.1	55.5	64.0		
2	3	27.7	38.5	52.3			
2	4	29.0	39.3	55.1			
5	5	24.7	31.8	45.1	51.5	160	8.0
8	6	31.2	44.0	60.8	64.9	180	7.0
8	7	29.3	38.2	55.2	63.1	170	8.2
11	8	34.2	44.7	60.1	68.8	160	7.4
11	9	32.2	43.6	57.6	69.1	205	7.0
13	10	36.3	50.2	60.2	71.6	160	6.0
13	11	32.2	48.0	62.0	75.4	190	7.1
18	12	33.9	46.8	65.1	79.2	195	8.0
18	13	29.5	42.5	60.5	72.0	200	7.0
20	14	24.8	48.8	70.3	79.3	220	7.0
20	15	32.1	48.4	66.1	77.3	200	7.2
25	16	33.8	50.1	63.6		190	5.9
25	17	39.1	45.7	74.0		170	5.2
26	18	29.9	44.0	58.9		180	6.8
26	19	27.8	41.1	56.1		190	8.2
57	20	29.4	44.8	76.2		190	3.0
57	21	33.8	52.1	78.8		180	3.6
63	22	27.7	36.9	58.3		190	4.6
63	23	33.0	46.0	69.5		95	6.2
67	24	32.2	42.2	63.8		170	8.5
67	25	41.3	51.7	73.4		140	6.4
74	26	28.4	43.1	63.6		190	7.8
83	27	28.2	41.3	62.6		170	8.0
83	28	29.8	45.8	64.7		120	7.6
84	29	29.8	42.7			190	8.0
84	30	28.8	44.3			135	7.4
84	31	0.0	32.2			190	10.5
84	32	23.1	33.4			170	8.6
AVERAGE:		**29.8**	**43.3**	**62.3**	**69.1**	**175**	**7.0**

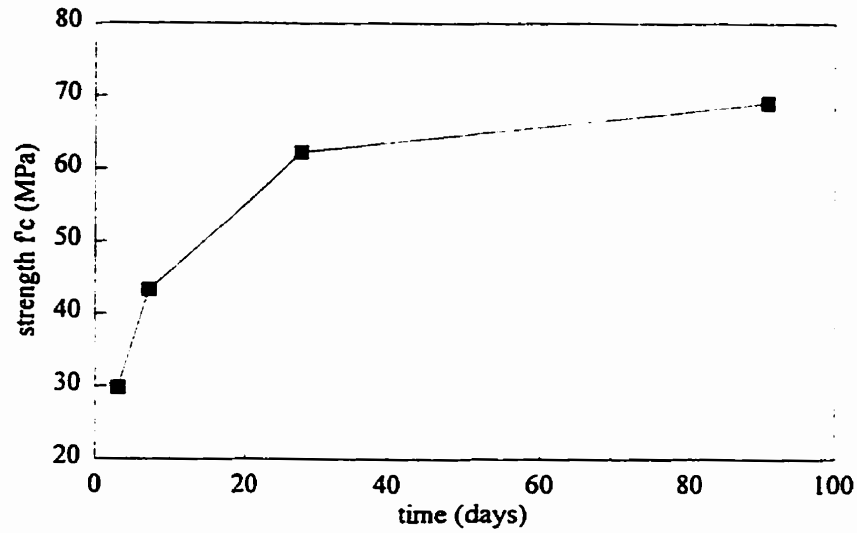


FIGURE E-1: average compressive strength vs. time

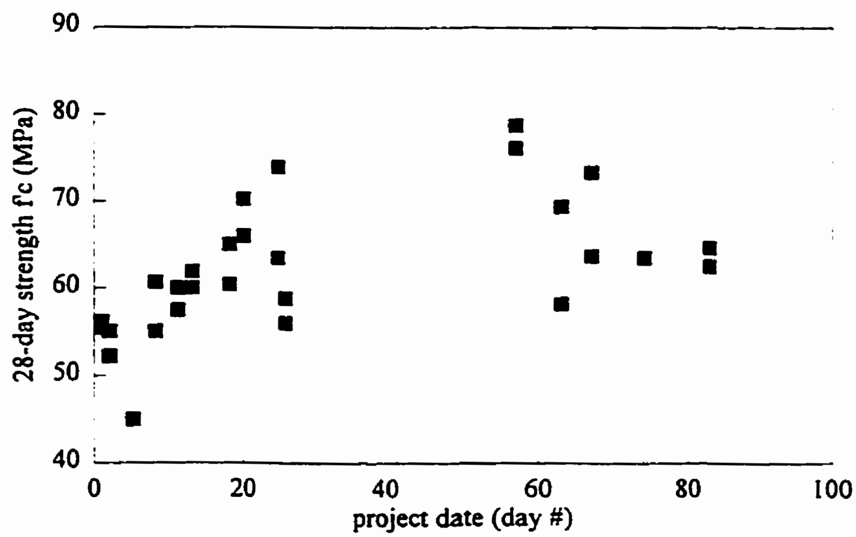


FIGURE E-2: 28-day compressive strength vs. project date

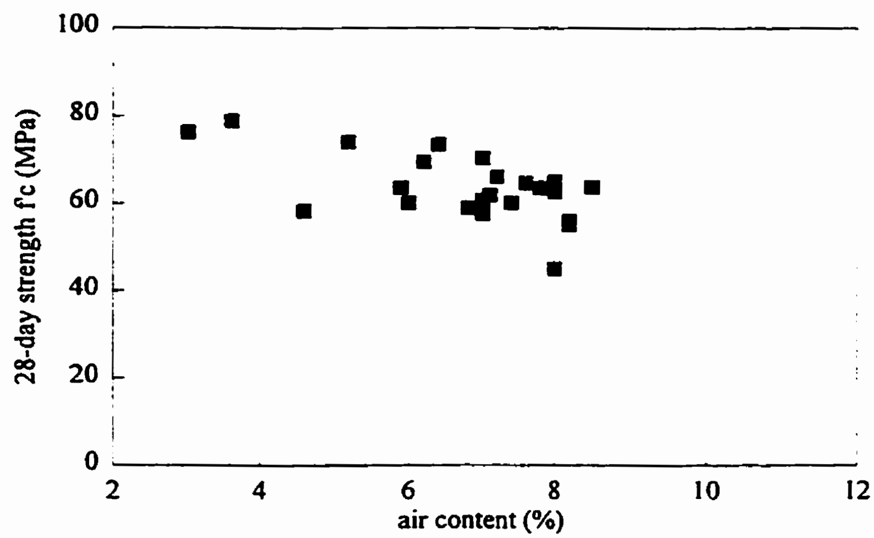


FIGURE E-3: 28-day compressive strength vs. air content

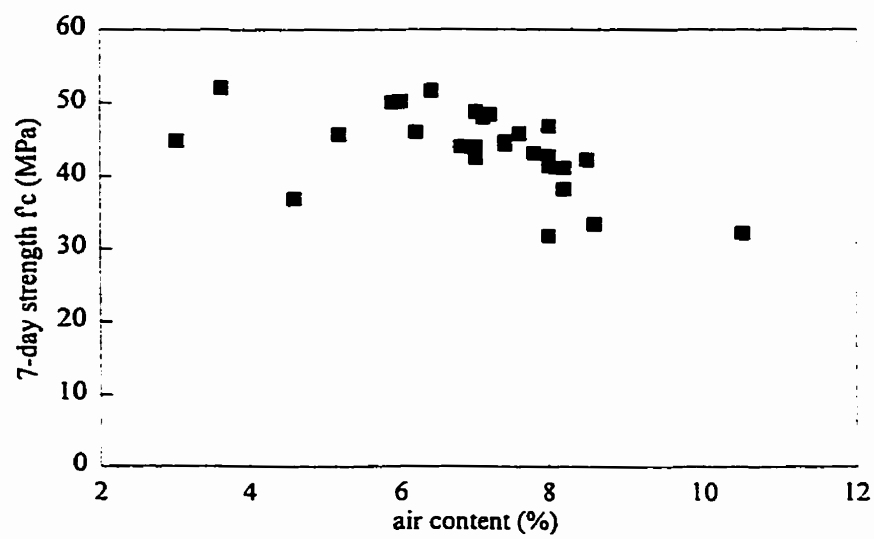


FIGURE E-4: 7-day compressive strength vs. air content

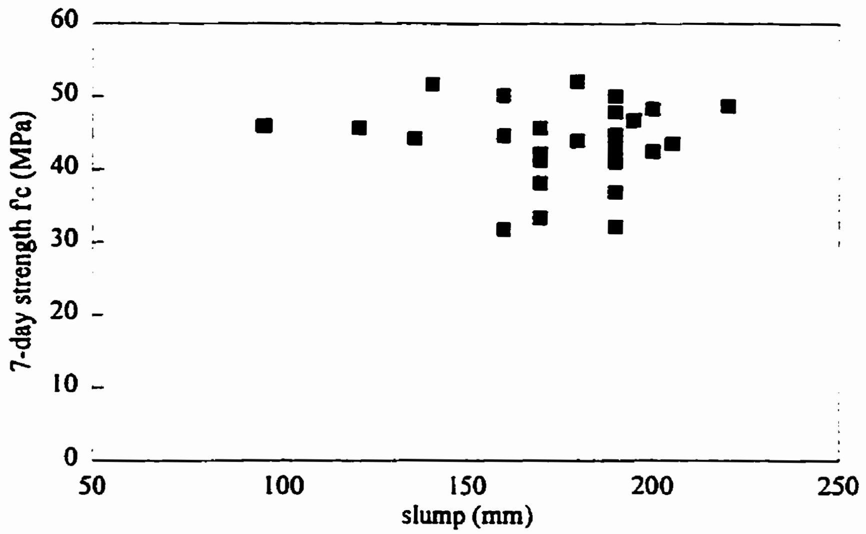


FIGURE E-5: 7-day compressive strength vs. slump

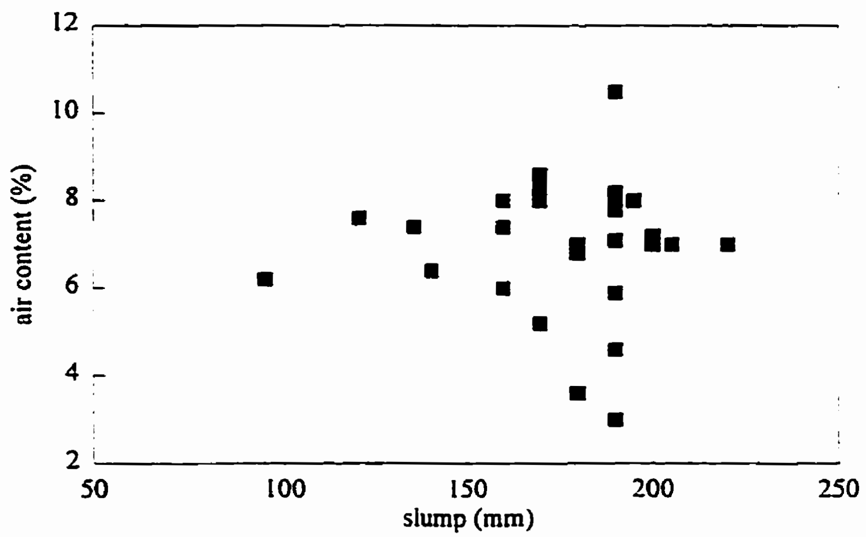


FIGURE E-6: air content vs. slump

APPENDIX F:

ECONOMIC EVALUATION

AND

SENSITIVITY ANALYSIS

OF

DESIGN OPTIONS

NSDoT: HPC BRIDGE PROJECT - ECONOMIC EVALUATION

<u>scope:</u>

The bridge is 2-span, skewed, 70.7 m x 8.85 m wide.
The bridge is a 2-lane overpass to a cut section of devided highway.
End supports are free-bearing and the central support is continuous for live load.
Design loading is CS600 (CSA 56-88)

<u>criteria:</u> **financial:**

discount rate = cost of borrowing =	7 %
escalation rate =	2 %
applicable taxes:	none

construction costs:

- construction quantities are based on conceptual/preliminary designs
- the foundations, abutments, piers, girders, deck, parapet wall and asphalt topping are considered for comparison of structural options
- only direct construction price estimates are compared (no testing, project management, administative overheads, interest during construction, etc)
- construction cost estimates are detailed below

durability:

- the expected life of the structure is 80-100 years (relates to the useful life of the bridge)
- the life for which the original design may be considered valid is considered to be 50 years (relates to changing traffic patterns and design loads)
- a life of 80 years is taken for life-cycle costing and for design related to durability
- major rehabilitation or repairs of joints may be required after about 35 years
- routine replacement of asphalt and waterproofing may be required every 12 years
- high performance concrete is expected to last for the design life without replacement.

high performance concrete (HPC):

- HPC with a design strength of 80 MPa, low permeability and controlled air-void spacing is proposed to improve the structural capacity of the prestressed girders and the durability of the deck, beam seats, walls, piers and abutments.
- a premium of $60 per cubic metre is estimated for HPC, based on $15 for low-alkali, silica fume blended cement, $20 for cement quantity, $10 for superplasticizer and $15 for field control and other overheads.

options: 1] normal strength concrete; asphalt topping (NSC)
- 4 prestressed concrete girders per span; epoxy-coated deck reinforcement

2] high performance concrete; asphalt topping (HPC-A)
- 3 HPC prestressed girders per span; HPC deck, piers and abutments; plain reinf.

3] high performance concrete; no asphalt topping (HPC-B)
- as for HPC-A except exposed HPC deck

results: **initial cost:**

- comparative initial costs include:

1] normal strength concrete; asphalt topping (NSC):	$484,697
2] high performance concrete; asphalt topping (HPC-A):	$470,317
3] high performance concrete; no asphalt topping (HPC-B):	$444,815

- HPC options have a significantly lower cost because of savings in number of prestressed concrete girders. HPC girders save in the order of: $33,441
This cost saving is directly attributable to the strength properties of HPC.

- the extra capital cost of HPC materials for all on-site concrete work (abutments, piers, deck and parapets) is approximately: $27,171

- the epoxy-coating of NSC deck reinforcement is estimated to cost: $8,110

- the use of an exposed HPC deck results in reduced structural weight and no waterproofing or asphalt. Exposing the concrete deck may save approx.: $25,502
However, construction experience has not been published to date.

life cycle costs:

- comparative life costs include:

1] normal strength concrete; asphalt topping (NSC):	$578,827
2] high performance concrete; asphalt topping (HPC-A):	$525,070
3] high performance concrete; no asphalt topping (HPC-B):	$454,587

- the improved durability of HPC concrete elements (excluding prestressed girders) results in life cycle savings of approximately:

- with asphalt topping:	$20,317
- without asphalt:	$90,799

- the results are reasonably insensitive to variations in depreciation, escalation or the schedule for repairs, or the cost of repairs (see sensitivity analysis).

construction costs:

item	NSC	HPC-A	HPC-B
SUMMARY:			
footings	$31,500	$37,800	$37,800
abutments, piers, beams, walls	27,217	31,300	31,300
girders	298,200	264,759	264,759
girder diaphrams	7,200	8,280	8,280
deck concrete, haunches	52,102	60,438	60,438
deck reinforcing	24,330	20,198	16,220
parapet walls	22,624	26,018	26,018
waterproofing	12,514	12,514	0
80 mm asphalt topping	9,010	9,010	0
TOTAL:	**$484,697**	**$470,317**	**$444,815**

DETAIL:

earthwork	- same for all options; not applicable			
	- structural depth is the same in all cases			
footings	- HPC used to check constructability and strength			
	- approximate volume of concrete:			
	30+44+31	=	105	m^3
	- approximate cost ($/m^3) for installed reinforced concrete:			
	300	360		360
	- total cost of item:			
	$31,500	**$37,800**		**$37,800**
abutments, piers, beams, walls				
	- approximate volume of concrete:			
	piers 3*6*3.14*0.4^2	=	9.04	m^3
	pier cap 1*9*1		9.00	
	abutments, wing walls 25+25		<u>50.00</u>	
	total:		<u>68.04</u>	m^3
	- approximate cost ($/m^3) for installed reinforced concrete:			
	400	460		460
	- total cost of item:			
	$27,217	**$31,300**		**$31,300**
girders	- AASHTO bulb T: cross-sectional area = 0.54 m^2			
	- base cost for NSC ($/m):	supply:	800	
		install:	<u>250</u>	
		total:	<u>1050</u>	
	- add increments for HPC: $30/m for concrete; $50/m, say, for prestressing			
	- add a further 10% for HPC "learning curve"			
	- total lengths:			
	- 8 girders @ 35.5m: 284 m	- 6 girders @ 35.5m: 213 m		- 6 girders @ 35.5m: 213 m
	- unit cost installed ($/m):			
	1050	1243		1243
	- total cost of item:			
	$298,200	**$264,759**		**$264,759**
girder diaphrams				
	- approximate volume of concrete:	=	18.00	m^3

- approximate cost ($/m^3) for installed concrete:			
	400	460	460
- total cost of item:			
	$7,200	**$8,280**	**$8,280**
deck concrete, haunches			
- includes formwork, concrete placement and finishing; excludes reinforcement			
- approximate volume of concrete:			
	8.85*70.7*0.2+13.8	= 138.94	m^3
- approximate cost ($/m^3) for installed concrete:			
	375	435	435
- total cost of item:			
	$52,102	**$60,438**	**$60,438**
deck reinforcing	- #20 @ 300 B + #15 @ 300 T + #15 @ 300 T&B: 23.57 kg/m^2 - epoxy coated	- #20 @ 250 T&B + #15 @ 300 T&B: 29.35 kg/m^2 - uncoated	- same as NSC 23.57 kg/m^2 - uncoated
- approximate area of deck:		8.85*70.7 = 625.70	m^2
- approximate cost ($/kg) for installed reinforcing:			
	1.65	1.10	1.10
- total cost of item:			
	$24,330	**$20,198** **$0**	**$16,220**
parapet walls			
- approximate volume of concrete:		2*70.7*0.4 56.56	m^3
- approximate cost ($/m^3) for installed reinforced concrete:			
	400	460	460
- total cost of item:			
	$22,624	**$26,018**	**$26,018**
waterproofing			
- approximate area of deck:		8.85*70.7 = 625.70	m^2
- approximate cost ($/m^2) for installed waterproofing:			
	20	20	not applicable
- total cost of item:			
	$12,514	**$12,514**	**$0**

80 mm asphalt topping			
- approximate volume: 8.85*70.7*0.080 =		50.06	m^3
- approximate cost ($/m^3) for installed asphalt topping:			
	180	180	not applicable
- total cost of item:			
	$9,010	**$9,010**	**$0**

repair costs:

Rehabilitation work being carried out at present typically includes:

1] expansion joint replacement:
   $1,200 - $1,800 per metre every 10-30 years
2] deck repairs - replace delaminated concrete; replace waterproofing and asphalt:
   $160 - $220 per square metre every 15-30 years
3] repair damage to other concrete surfaces, e.g.. abutment walls, beam seats:
   $50,000 - 200,000 after 20-30 years
4] replace asphalt topping and waterproofing:
   $12 - 20 plus $16-20 per square metre every 8-12 years
5] related traffic control:
   $15,000 for asphalt replacement; $25,000 for joint or deck repairs

Consider that improvements in conventional technology will extend the time period between major rehabilitation operations: consider future rehabilitation projects will be required after 30-40 years of service.

Then rehabilitation after, say, 35 years includes:

NSC:	1] expansion joint replacement:	1400*2*8.85 =	$24,780
	2] deck repairs:	190*8.85*70.7 =	$118,882
	3] repair other concrete surfaces:		$125,000
	5] related traffic control:		$25,000
			$293,662
HPC-A:	1] expansion joint replacement:	1400*2*8.85*1.5extra =	$37,170
	4] asphalt topping, waterproofing:	34*8.85*70.7 =	$21,274
	5] related traffic control:		$25,000
			$83,444
HPC-B:	1] expansion joint replacement:	1400*2*8.85*1.5extra =	$37,170
	5] related traffic control:		$15,000
			$52,170

And routine repairs after, say, 12 years includes:

NSC and HPC-A:			
	4] asphalt topping, waterproofing:	34*8.85*70.7 =	$21,274
	5] related traffic control:		$15,000
			$36,274

HPC-B: no cost

life cycle cost estimates:

discount rate: 7.00%
escalation rate: 2.00%

item	year	NSC $ (PC)	NSC $ (NPV)	HPC-A $ (PC)	HPC-A $ (NPV)	HPC-B $ (PC)	HPC-B $ (NPV)
bridge construction:							
	0	$484,697	$484,697	$470,317	$470,317	$444,815	$444,815
replace asphalt topping if applicable:							
	12	$36,274	$20,426	$36,274	$20,426	$0	$0
	24	$36,274	$11,502	$36,274	$11,502	$0	$0
rehabilitation or major repairs:							
	35	$293,662	$55,007	$83,444	$15,630	$52,170	$9,772
replace asphalt topping if applicable:							
	47	$36,274	$3,826	$36,274	$3,826	$0	$0
	59	$36,274	$2,155	$36,274	$2,155	$0	$0
	71	$36,274	$1,213	$36,274	$1,213	$0	$0
total, net present value:			$578,827		$525,070		$454,587
comparison of net present costs:							
compare all variable costs with HPC-A:							
			$53,758		$0		($70,483)
subtract:comparative cost of prestressed girders (primarily related to strength of materials):							
			($33,441)		$0		$0
then costs related to durability, compared with HPC-A:							
			$20,317		$0		($70,483)

sensitivity analysis:

summary:

base condition:

discount rate:	7.00%
escalation rate:	2.00%

compare all variable costs with HPC-A:

		NSC	HPC-A	HPC-B
	$ (NPV):	$53,758	$0	($70,483)
a] vary discount rate:				
	5.00%	$90,597	$0	($101,026)
	difference:	$36,840	$0	($30,544)
	9.00%	$34,975	$0	($54,947)
	difference:	($18,783)	$0	$15,535
b] vary escalation rate:				
	1.00%	$42,273	$0	($61,096)
	difference:	($11,485)	$0	$9,387
	3.00%	$69,784	$0	($83,553)
	difference:	$16,027	$0	($13,071)
c] vary time frame for repairs:				
year factor:	75.00%	$74,235	$0	($87,229)
	difference:	$20,478	$0	($16,747)
year factor:	150.00%	$31,423	$0	($51,832)
	difference:	($22,335)	$0	$18,650

sensitivity analysis / ...

a] vary discount rate: **discount rate:** **5.00%**
escalation rate: 2.00%

item	year	NSC $ (PC)	NSC $ (NPV)	HPC-A $ (PC)	HPC-A $ (NPV)	HPC-B $ (PC)	HPC-B $ (NPV)
	0	$484,697	$484,697	$470,317	$470,317	$444,815	$444,815
	12	$36,274	$25,617	$36,274	$25,617	$0	$0
	24	$36,274	$18,091	$36,274	$18,091	$0	$0
	35	$293,662	$106,470	$83,444	$30,253	$52,170	$18,915
	47	$36,274	$9,288	$36,274	$9,288	$0	$0
	59	$36,274	$6,559	$36,274	$6,559	$0	$0
	71	$36,274	$4,632	$36,274	$4,632	$0	$0
total, net present value:			$655,353		$564,756		$463,730
comparison of net present costs with HPC-A:			$90,597		$0		($101,026)

discount rate: **9.00%**
escalation rate: 2.00%

item	year	NSC $ (PC)	NSC $ (NPV)	HPC-A $ (PC)	HPC-A $ (NPV)	HPC-B $ (PC)	HPC-B $ (NPV)
	0	$484,697	$484,697	$470,317	$470,317	$444,815	$444,815
	12	$36,274	$16,356	$36,274	$16,356	$0	$0
	24	$36,274	$7,375	$36,274	$7,375	$0	$0
	35	$293,662	$28,769	$83,444	$8,175	$52,170	$5,111
	47	$36,274	$1,602	$36,274	$1,602	$0	$0
	59	$36,274	$722	$36,274	$722	$0	$0
	71	$36,274	$326	$36,274	$326	$0	$0
total, net present value:			$539,848		$504,873		$449,926
comparison of net present costs with HPC-A:			$34,975		$0		($54,947)

sensitivity analysis / ...

b] vary escalation rate: **escalation rate:** **1.00%**

discount rate: 7.00%

item	year	NSC $ (PC)	NSC $ (NPV)	HPC-A $ (PC)	HPC-A $ (NPV)	HPC-B $ (PC)	HPC-B $ (NPV)
	0	$484,697	$484,697	$470,317	$470,317	$444,815	$444,815
	12	$36,274	$18,149	$36,274	$18,149	$0	$0
	24	$36,274	$9,080	$36,274	$9,080	$0	$0
	35	$293,662	$38,964	$83,444	$11,072	$52,170	$6,922
	47	$36,274	$2,408	$36,274	$2,408	$0	$0
	59	$36,274	$1,205	$36,274	$1,205	$0	$0
	71	$36,274	$603	$36,274	$603	$0	$0
total, net present value:			$555,106		$512,833		$451,737
comparison of net present costs with HPC-A:			$42,273		$0		($61,096)

escalation rate: **3.00%**
discount rate: 7.00%

item	year	NSC $ (PC)	NSC $ (NPV)	HPC-A $ (PC)	HPC-A $ (NPV)	HPC-B $ (PC)	HPC-B $ (NPV)
	0	$484,697	$484,697	$470,317	$470,317	$444,815	$444,815
	12	$36,274	$22,963	$36,274	$22,963	$0	$0
	24	$36,274	$14,537	$36,274	$14,537	$0	$0
	35	$293,662	$77,396	$83,444	$21,992	$52,170	$13,750
	47	$36,274	$6,052	$36,274	$6,052	$0	$0
	59	$36,274	$3,831	$36,274	$3,831	$0	$0
	71	$36,274	$2,425	$36,274	$2,425	$0	$0
total, net present value:			$611,902		$542,118		$458,565
comparison of net present costs with HPC-A:			$69,784		$0		($83,553)

sensitivity analysis / ...

c] vary time frame for repairs: **year factor:** **75.00%**
discount rate: 7.00%
escalation rate: 2.00%

item	year	NSC $ (PC)	NSC $ (NPV)	HPC-A $ (PC)	HPC-A $ (NPV)	HPC-B $ (PC)	HPC-B $ (NPV)
	0	$484,697	$484,697	$470,317	$470,317	$444,815	$444,815
	9.00	$36,274	$23,580	$36,274	$23,580	$0	$0
	18.00	$36,274	$15,328	$36,274	$15,328	$0	$0
	26.25	$293,662	$83,614	$83,444	$23,759	$52,170	$14,854
	35.25	$36,274	$6,714	$36,274	$6,714	$0	$0
	44.25	$36,274	$4,364	$36,274	$4,364	$0	$0
	53.25	$36,274	$2,837	$36,274	$2,837	$0	$0
total, net present value:			$621,134		$546,899		$459,669
comparison of net present costs with HPC-A:			$74,235		$0		($87,229)

year factor:	150.00%
discount rate:	7.00%
escalation rate:	2.00%

item	year	NSC $ (PC)	NSC $ (NPV)	HPC-A $ (PC)	HPC-A $ (NPV)	HPC-B $ (PC)	HPC-B $ (NPV)
	0	$484,697	$484,697	$470,317	$470,317	$444,815	$444,815
	18.00	$36,274	$15,328	$36,274	$15,328	$0	$0
	36.00	$36,274	$6,477	$36,274	$6,477	$0	$0
	52.50	$293,662	$23,807	$83,444	$6,765	$52,170	$4,229
	70.50	$36,274	$1,243	$36,274	$1,243	$0	$0
	88.50	$36,274	$525	$36,274	$525	$0	$0
	106.50	$36,274	$222	$36,274	$222	$0	$0
total, net present value:			$532,299		$500,877		$449,044
comparison of net present costs with HPC-A:			$31,423		$0		($51,832)

IMAGE EVALUATION TEST TARGET (QA-3)

150mm

6"

APPLIED IMAGE . Inc
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Rochester, NY 14609 USA
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Fax: 716/288-5989