TABLE 6--BUILDING DURABILITY STUDIES IN SYDNEY -- DATA ON CORROSION OCCURRENCE AND BUILDING COASTAL PROXIMITY

Building Groups		Building Location		
		Over 2 km from water	Between 1-2 km from water	Less than 1 km from water
All Buildings	Total No.	12	19	105
	Corroded No.	4	7	43
	% Corroded	33	37	41
Group (i) Buildings	Total No.	5	3	10
	Corroded No.	2	2	3
	% Corroded	40	67	30
Group (ii) Buildings	Total No.	7	16	17
	Corroded No.	2	5	8
	% Corroded	29	31	47
Group (iii) Buildings	Total No.			79
	Corroded No.			32
	% Corroded			41

TABLE 7--CARBONATION STUDIES -- COMPARISONS BETWEEN SITE OBTAINED RESULTS AND BRE LABORATORY STUDIES (3, 20, 21)

Mix Reference Number	Water Content (kg/m^3)	Cement Content (kg/m^3)	Fly Ash Content (kg/m^3)	Mean Carbonation Depths: Air Stored 10 yr exp (mm) (X)	Mean Carbonation Depths: Water Cured/ Site Exp. (8 yr exp) (mm) (Y)	Calculated Carbonation (Ref. 3) f(Age,C,W:C)(mm) (Z)
C2R	195	227	0	31.0	4.0	7.2
P3R	169	186	67	31.0	5.0	8.6
C3R	211	230	0	39.0	8.5	8.8
P4	184	190	70	39.0	7.0	10.4
C4R	180	340	0	14.0	0.5	1.8
P5R3	171	277	89	14.0	0.5	2.8
C5	203	338	0	14.5	1.5	2.5
P6R	182	276	91	16.5	1.0	3.3
C6	182	228	0	28.5	5.0	5.9
P7R	151	187	67	27.0	8.0	6.2
C7R	202	227	0	34.5	4.0	8.0
P8	171	188	72	33.5	5.0	8.7
C8R2	174	335	0	16.0	1.5	1.7
P9R	167	270	89	17.0	2.5	2.8
C9R	193	344	0	15.5	1.5	2.1
P10R	188	276	91	18.5	2.5	3.7
C10R	191	225	0	41.0	3.0	7.0
P11	160	184	70	26.0	4.0	7.6
C11R	216	227	0	45.0	3.5	9.7
P12	187	189	72	46.0	6.5	11.0
C12R2	180	341	0	14.0	1.0	1.7
P13R	180	276	88	18.0	2.0	3.2
C13R	203	338	0	17.5	1.0	2.5
P14R	187	275	91	18.0	2.0	3.6
3C2R	193	225	0	46.5		7.2
3P15	166	186	67	33.0		8.2
3C3R	213	231	0	44.5		8.9
3P16	180	189	70	39.5		9.9
3C4R	180	340	0	12.0		1.8
3P17	170	275	88	12.5		2.8
3C5	203	339	0	16.5		2.5
3P18	162	276	91	15.5		2.4

Summary of obtained relationships

X = 3.50(Z) + 7.04 - R Squared (Significant to 1% level) - 0.87

Y = 0.62(Z) + 0.01 - R Squared (Significant to 1% level) - 0.66

X = 3.46(Y) + 13.94 - R Squared (Significant to 1% level) - 0.57

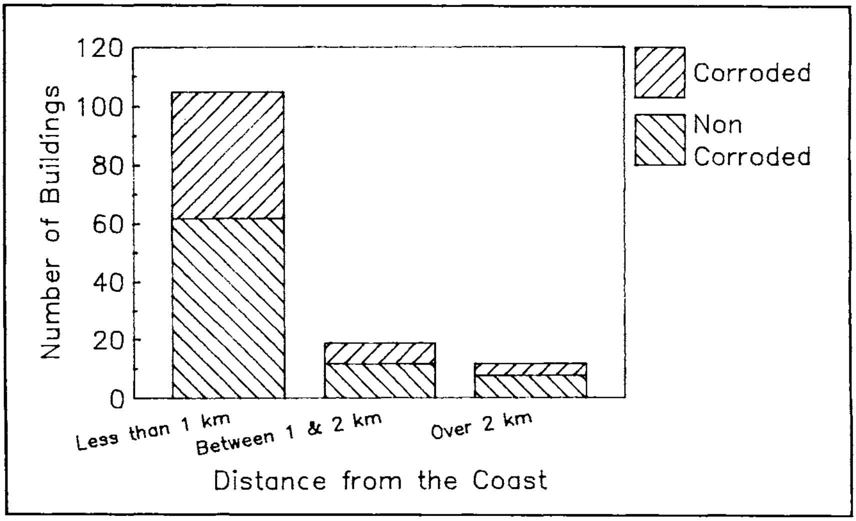
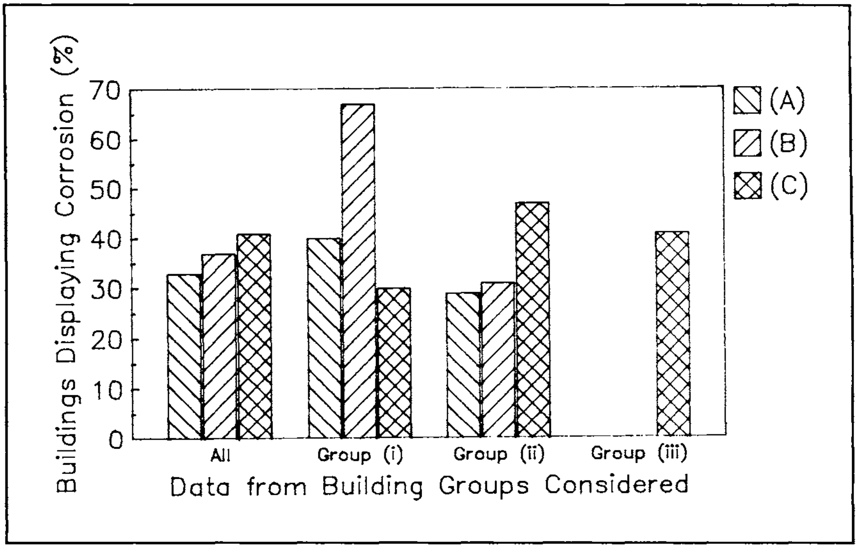


Fig. 1--Building survey results corroded and non-corroded buildings



(A) Data for Buildings Located Over 2 km from the Coast
(B) Data for Buildings Located Between 1 km and 2 km from the Coast
(C) Data for Buildings Located Less Than 1 km from the Coast

Fig. 2--Building survey results distribution of corroded buildings (percent)

Comparison of Creep and Shrinkage of High-Strength Silica Fume Concretes with Fly Ash Concretes of Similar Strengths

by M.D. Luther and W. Hansen

Synopsis: The specific creep and shrinkage of five high-strength concrete mixtures were monitored for 400 days at The University of Michigan, Ann Arbor, during research sponsored by Elkem Materials Inc., of Pittsburgh. One 52.6 MPa (7630 psi) silica fume (SF) concrete was compared with a fly ash concrete of similar compressive strength, and SF concrete and fly ash concrete having compressive strengths of approximately 69 MPa (10,000 psi) were compared. A 106.6 MPa (15,450 psi) SF concrete was also studied.

The creep of the SF concretes was not significantly different from that of the fly ash concretes. Furthermore, the relationship between creep and compressive strength was consistent with that reported in the open literature for high-strength portland cement concretes. The shrinkage of the SF concretes was either equal to or less than that of the associated fly ash concretes.

Several other concrete properties were studied, including: slump retention, time of setting, compressive strength development for one year, split-tensile strength, modulus of rupture, and for the nominal 69 MPa concretes only, rapid freezing and thawing durability and the hardened concrete air-void system.

Keywords: creep properties; flexural strength; fly ash; freeze-thaw durability; high-strength concretes; modulus of elasticity; shrinkage; silica fume; splitting tensile strength

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INTRODUCTION

This paper presents the results of a cooperative laboratory investigation conducted by The University of Michigan, Ann Arbor, and Elkem Materials Inc., of Pittsburgh, Pennsylvania. The study focused upon the creep and shrinkage properties of high-strength concretes, comparing silica fume (SF) mixtures with fly ash mixtures of similar compressive strengths. One very high-strength SF mixture was studied also.

SCOPE

Five high-strength concretes were produced. Two were "off the shelf" fly ash mixtures that were provided by ready mix concrete suppliers. One (batch 1) achieved a 54.1 MPa (7850 psi) 28-day compressive strength, and the other (batch 3) obtained 72.1 MPa (10,460 psi). Two alternate SF mixtures (batch 2 and batch 4) were developed. These were designed to match the 28-day compressive strengths of the two fly ash concrete mixtures. The last mixture (batch 5) was a very high-strength SF concrete that achieved 106.6 MPa (15,450 psi).

The tests included air content, slump, unit weight and yield, time of setting, compressive strength development for up to one year, split-tensile strength, modulus of rupture, specific creep (ASTM C512-82), drying shrinkage, and for the two nominal 69 MPa mixtures only, rapid freezing and thawing durability (ASTM C666-84, procedure A), and the determination of the hardened concrete air-void system parameters (ASTM C457-82a).

CONCRETE MATERIALS

The concretes were made with the following materials:

Cement

The same ASTM C150 type I portland cement was used for all the concretes. The physical and chemical properties of the cement are shown in Table 1.

Fly Ash

The fly ash was an ASTM C618 class C material. The physical and chemical properties of the fly ash are listed in Table 1.

Silica Fume and Silica Fume-Based Admixtures

The silica fume (SF) that was used in this investigation came from the production of elemental silicon. Typical physical and chemical properties for the SF are presented in Table 1. All of the SF concretes used the same SF. The SF was added to the concrete in the form of two commercially available slurried products, which are referred to in this paper as SF-A and SF-B. Each of these two products contained, by weight, 45% SF, 51% water, and a proprietary combination of refined sodium lignosulfonates and sodium naphthalene condensate admixtures.

Coarse Aggregate

The coarse aggregate was crushed dolomite having a specific gravity of 2.83 and a 0.40% absorption. Two coarse aggregate sizes were used. One, for batch 3 only, was an ASTM C33-86 No. 7 size, and the other was a No. 67 size. The gradation analysis for each of the coarse aggregate sizes is shown in Table 2.

Fine Aggregate

The fine aggregate was dredged from the Ohio River. The specific gravity of this material was 2.60, the absorption was 1.40%, and the gradation analysis is presented in Table 2.

Water-Reducing Admixture

The chemical water-reducing admixture (WRA) was a sodium salt of a hydroxylated carboxylic acid, 40% solution in water. The WRA was an ASTM C494-86, class A material.

Water-Reducing and Retarding Admixture

The chemical water-reducing and retarding admixture was an ASTM C494-86, class D material. This admixture was a 44% solution of a hydroxylated polymer in water.

High-Range Water-Reducing Admixture

The high-range water-reducing admixture (HRWRA) was an ASTM C494-86, class F material. This admixture was a 33% solution in water of a sodium melamine condensate.

CONCRETE PRODUCTION AND PROPORTIONS

Only mixtures suitable for use in the field were chosen for this study. Batches 2 and 4 (Table 3) were proportioned to achieve similar 28-day compressive strengths and initial slumps as batches 1 and 3, respectively.

Batch 5 (Table 3) was patterned after the SF concrete mixture that was used for the Kinzua Dam Stilling Basin Rehabilitation project of 1983 (1), although small proportioning adjustments were made to achieve the desired strength. Many high-strength applications do not require the use of air-entrained concrete. Therefore, the concretes produced for this investigation were not air-entrained.

Concrete was manufactured in a 0.17 cubic meter (6 cu. ft.) capacity 24 RPM drum mixer. For each mixture two equally-sized batches were produced, and these were combined by hand in a receiving tray. Admixtures were added separately after the other materials were combined in the mixer.

TEST METHODS AND PRACTICES

The test methods and practices are described as follows:

Slump: ASTM C143-78.

Air Content: ASTM C231-82.

Unit Weight and Yield: ASTM C138-81. The bucket volume was 7.05 liters (0.249 cu. ft.).

Compressive Strength: ASTM C39-86. A Forney (model LT-806D) 272 metric ton (600,000 lb) capacity machine was used.

Molds: ASTM C470-81. Plastic molds were used.

Casting and Curing Specimens: ASTM C192-81. Specimens were cured in lime-saturated water.

Capping Cylinders: ASTM C617-856. A high-strength sulfur-mineral filler capping compound was used.

Modulus of Rupture: ASTM C78-84. Nominal beam dimensions were 152.4 by 152.4 by 533.4 mm (6 by 6 by 21 in.). The beams were tested on a Forney (model BT-30) flexural testing machine.

Split-Tensile Strength: ASTM C496-85. Nominal 101.6 mm (4 in.) diameter cylindrical specimens were tested in the compressive strength testing machine (above) using 3.2 mm (1/8 in.) thick plywood bearing strips.

Time of Setting: ASTM C403-85.

Rapid Freezing and Thawing (batches 3 and 4 only): ASTM C666-84, procedure A. Three nominal 76.2 by 101.6 by 406.5 mm (3 by 4 by 16 in.) prisms were tested for each mixture in a Logan Freeze-Thaw Mfg. Co. (model H-3185) testing machine. Concretes were placed in the machine at 14 days of age.

Air-Void System (batches 3 and 4 only): ASTM C457-82a, modified point count method.

Creep: ASTM C512-82 (1983). Two specially manufactured Soil Test (model CT-180) 40.8 metric ton (90,000 lb) capacity creep jigs were used. The 152.4 by 304.8 mm (6 by 12 in.) cylindrical specimens were arranged, listed by specimen number, in the creep jigs as follows: jig 1 - 1C (top), 2E, 1B, 2C, and 3A (bottom); jig 2 - 3D (top), 4D, 5A, 4E, and 5E (bottom). The jig 1 concrete was stressed to 14.6 MPa (2130 psi), and the jig 2 concrete was stressed to 20 MPa (2900 psi). The creep stresses were 27%, 28%, (20% and 28%), 30% and 19% of the 28-day compressive strengths for batches 1, 2, 3 (for specimens 3A and 3D), 4, and 5, respectively.

Two sets of brass inserts with steel contact points were cast into opposing sides of the test cylinders. The nominal gauge length was 152.4 mm (6 in.). The length-measuring apparatus was a Soil Test (model CT-171) multi-position strain gauge which was capable of measuring in 0.00254 mm (0.0001 in.) increments through a 5.08 mm (0.2 in.) range.

The creep specimens were cured under wet burlap for 20-24 hours, then immersed in lime-saturated water for three weeks. The specimens were then packed individually in lime-saturated water and driven from Pittsburgh to The University of Michigan where they were cured in a fog room until the 28th day. The creep and shrinkage room environment was 23° ± 1.7°C (73.4° ± 3°F) at 50 ± 5% RH.

Static Modulus of Elasticity: The static modulus of elasticity (E) was estimated from the initial loaded creep specimen stress and strain conditions. Unlike ASTM C469-83, which requires a determination of E for a stress level at 40% of ultimte, the E results were calculated as follows: E = initial creep specimen stress/initial creep specimen strain.

Drying Shrinkage: Unloaded 152.4 mm (6 in.) diameter cylinders served as the shrinkage specimens. The contact points, gauge length and strain gauge were the same as those employed for the creep specimens. The cylinders were also prepared, transported, and cured in the same manner as the creep specimens.

TEST RESULTS AND DISCUSSION

Fresh Concrete Properties

The slump results were plotted and smooth curves were fitted to the related points (Figure 1). Reasonably similar slumps were achieved for the comparison pair concretes (Table 3). Batches 1 and 2 showed similar slump retention behavior, and the same held true for batches 3 and 4 for 30 minutes. After 30 minutes this SF concrete showed better slump retention than its fly ash concrete counterpart. Batch 5, made with SF-B, showed the best slump retention even though it had the lowest initial slump. It is believed that the slump rentention properties may be attributed to the chemical admixtures.

The SF-A concretes showed somewhat higher air contents than their fly ash concrete counterparts (Table 3), while the air content of the SF-B concrete, which used the highest SF dose, was 0.9% lower than that of batch 4. This suggests that factors other than the SF dose also affect the air content.

The SF concretes showed lower unit weights (Table 4) than their fly ash concrete counterparts, an observation attributed primarily to air content differences. The SF-B concrete showed the highest unit weight, as one might expect, considering that this mixture had the highest aggregate volume (Table 3).

All of the concretes were retarded (Table 3). However, the SF mixtures were significantly less retarded than their fly ash concrete counterparts. In fact, batch 1 showed more retardation than batch 5, and batch 5 was made with SF-B, a product known for its strong retarding effect.

Compressive Strength

Considering the 152.4 mm (6 in.) diameter cylinder results (Table 4), batches 1 and 2 achieved similar 28-day compressive strengths. The compressive strengths of batches 3 and 4 were not as close to each other as the batch 1 and 2 results, although either batch the 3 or batch 4 mixture could satisfy a 55.2 MPa (8000 psi) specified compressive strength requirement. Having achieved 106.6 MPa (15,450 psi) at 28 days, the batch 5 mixture could provide concrete at the highest known specified compressive strength level (1).

The results for the 101.6 mm (4 in.) diameter cylinders are presented in Table 4, and the strength development curves are shown in Figure 2. All concrete mixtures continued developing increasing strength after 28 days, with batches 1, 2, 3, 4, and 5 showing 26.4, 12.5, 31.0, 8.9, and 15.9% strength increases from 28 days to one year. The concretes containing fly ash showed the greatest long-term strength gains. It should be noted that the SF-B concrete achieved 119.5 MPa (17,330 psi) at one year.

Split-Tensile Strength

The split-tensile strength results (Table 4, Figure 3) for all five mixtures appear to follow the same curve. Also, the magnitude of the split-tensile strength-to-compressive-strength ratio becomes smaller as strength increases (Table 4). Both the magnitude and the trend of this ratio agree with previously reported observations for portland cement concretes (2). It is not known why the use of a nominal 12.7 mm (1/2 in.) aggregate in batch 3 did not appear to influence this split-tensile strength result.

Modulus of Rupture

The modulus of rupture (MR) results (Table 4, Figure 3) for all the SF concretes showed essentially the same relationship between modulus of rupture and compressive strength, which may be modeled by the following expression: $MR = 1.021(fc(MPa))^{1/2}$ $(MR = 12.3(fc(psi))^{1/2})$.

Batch 1 showed somewhat less flexural strength than its alternate SF mixture. This may be due to a stronger aggregate-to-paste bond in the SF concrete. Batch 4 developed higher flexural strength than its alternate SF mixture. This result may be attributed to the use of smaller aggregate in this fly ash mixture.

Rapid Freezing and Thawing Durability and Hardened Concrete Air-Void System

Both the batch 3 and the batch 4 concretes showed clearly acceptable freezing and thawing durability. The 300 cycle durability factors were 97.6 and 95.5 for batches 3 and 4, respectively, and the prism weight changes were very small, at much less than 0.1%. When testing was finally discontinued at 634 cycles, the two mixtures continued to demonstrate acceptable performance (Percent dynamic modulus of elasticity values were 83.7 and 92.2 for batches 3 and 4, respectively.). Neither concrete showed a traditionally acceptable hardened concrete air-void system (Table 4). For example, the spacing factors exceeded 0.200 mm (0.008 in.). Knowing this, it appears that the concretes were not critically saturated during testing - a condition that may occur in low water-to-cementitious materials ratio concretes.

Static Modulus of Elasticity (E)

The E results (Table 4, Figure 4) for all of the concretes appear to follow the same curve. However, the magnitudes of the results are somewhat lower than predicted by applicable ACI equations. This observation is consistent with the findings of Carrisquillo, Nilson and Slate (3), Ahmad, Shuaib and Shah (4), and Hansen (5).

Specific Creep

The specific creep results (Table 5) up to 400 days were plotted in Figure 5 and smooth curves were fitted through these points. From these curves the specific creep stress at one year of age was estimated (Table 5). The relationship between specific creep at one year and E (Figure 6) (6) appears to be best described by a single hyperbolic-shaped curve that is similar to one reported by McDonald (7) and by Reinhold et al. (8). This suggests that the specific creep of SF concrete is similar to that of concretes that do not contain SF when compared at similar E.

The relationship between specific creep and compressive strength is shown in Figure 7 for not only the data from this study, but also data from Wolsiefer (9), Perenchio and Klieger (10), Saucier (11), Ngab, Nilson and Slate (12), and Neville (13). The Ngab, Nilson and Slate, and the Neville data have been adjusted to one-year specific creep values using the ACI equation $Vt = (t^{0.6}/(10 + t^{0.6}))\ Vu$ (14). The data from this study appear to show a hyperbolic relationship between specific creep and compressive strength. Also, these data nestle between the Neville data (dashed curve on the left) and the Perenchio and Klieger results (dashed curve on the right), and they agree with the applicable Ngab, Nilson and Slate results. Furthermore, these data near the high-strength levels are close to the Wolsiefer and Saucier results. Thus,