

OPENING REMARKS

Good morning, good afternoon and good evening. It is at once a great honor and distinction, as well as a real pleasure for me to welcome all of you here at this Second Shotcrete Conference. We are happy that so many of you have been able to make this week available to share with others the exchange of ideas and experience in the informal setting of an Engineering Foundation Conference. Among the 100 participants registered so far, there are no less than 34 from overseas, including three from as far away as South Africa and one from Japan. We of the Organizing Committee are particularly pleased to have found such an enthusiastic response among our European colleagues.

On behalf of the Engineering Foundation, Dr. Cole its Executive Director and on behalf of the Organizing Committee, it is a pleasure to acknowledge at this time the co-sponsorship of the following organizations: The Department of Transportation, the American Concrete Institute, the Society of Mining Engineers of AIME, and the Canadian Geotechnical Society.

It is a great personal pleasure for me to recognize at this time the members of the Organizing Committee:

Bob Bates, Supervisory Mining Engineer at the Spokane Mining Research Center of the USBM in Spokane, Washington.

Jan Blanck, Shotcrete expert and troubleshooter with Di-Mambro Majestic. He lives in Lanham, Maryland.

Robin Mason, also a consultant on shotcrete and senior engineer with the A. A. Mathews Company from their project office in Montreal, Canada.

Harvey Parker, Parsons, Brinkerhoff, Quade and Douglas in New York, where he is Head, Geotechnical Department. Previously he was with the University of Illinois.

Tom Reading, Materials Engineer from the Missouri River Division of the Corps of Engineers in Omaha, Nebraska.

In addition, the committee has had the active participation of two associate members: Hans Egger of Meynadier in Zurich, Switzerland and my colleague, stand in, helper and coordinator, Ed Cikanek of Harza.

I wish to acknowledge the continuous, unflagging support of Dr. Cole's staff of the Engineering Foundation in New York.

Three years and three months ago, we met in South Berwick, Maine to discuss shotcrete as a means for stabilizing underground openings. Recently, I glanced through the 16-page report I prepared for my own company. I was impressed with the broad and wide ranging discussions of successful (and some not so successful) endeavors and also of the many problems that still remained. I hope to see some of these questions answered here, at least I trust that new light will be thrown on all of them, including such interesting controversies as wet vs. dry process, the effect of accelerators on durability, the bond at the shotcrete/rock interface, better control of rebound, quality control of shotcrete in place, improvements in delivery to the nozzle and last, but not least, improvement of specifications and contractual conditions.

I trust the exchange of experience with our many friends from abroad will be useful and beneficial for all, will contribute in particular to a more wide ranging application of shotcrete on the North American continent, and further clarify the ongoing discussions of the proper place for shotcrete applications. I hope this conference will contribute to the better use of shotcrete, to a better understanding among the sceptics and to the advancement of both the science and the art of its application.

J. A. Veltrop
Conference Chairman

KEYNOTE ADDRESS

by

E. E. Mason*

I was invited to speak here today as we were early users of coarse aggregate shotcrete on this continent. It might be of interest to tell you how we became involved.

I first heard of its use in tunneling from Helmut Kobler, a project superintendent of one of the mixed ground projects on the Montreal Subway in 1964. He had been in charge of construction of twin vehicular tunnels in Venezuela, one driven with conventional steel supports and the other with shotcrete. The latter was under the direction of Rabcewicz.

Kobler's lucid descriptions of the relative difficulties in both headings, supported by a collection of large photographs was impressive. It seemed evident a new and improved method had been developed of tunneling support in poor ground.

We had used guniting as a sealant against raveling in the arch of an unsupported section of a tunnel in downtown Vancouver in 1957. While the application proved effective, it did not suggest itself as a substitute for the steel supported sections underway.

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The 1965 Rabcewicz article in "Water Power" was the first documented account that came to us, of the use of coarse aggregate shotcrete for support in poor to very poor ground.

In particular, it offered a practical alternative to spiling or fore-poling, an unnecessarily slow and painful exercise usually in rock. An early personal experience had been the mining of 18 acres of a bed of gold-bearing gravels at 120 ft. depth. In corollary, the relative unskillfulness and slow tempo of work in unstable and caved rock obviously compounded the difficulties with increased loadings.

The use of shotcrete offered an immediacy of application and an increase in standing time, at least. The differentiation made between incipient and detrimental loosening also rang the bell of past experience. Descriptions of work done were impressive, having regard to the difficulties that could be expected with conventional support.

We proceeded to gather what we could in German and Italian literature for translation, with particular interest in guidelines used and product application and standards; of which probably the most useful were, in addition to Rabcewicz, the papers of Drogslers, Linder and Rotter. Among papers printed in English, Claes Alberts' represented proven practice that had become routine in the extent of its successful application.

In 1966, we were retained by Canadian National Railways for the construction of a tunnel under a dormitory area of Vancouver, B.C., through semi-indurated tertiary sediments and glacial tills. Ample surface exposures and drill cores indicated the necessity of support and lining throughout. We decided on shotcrete support as a viable option.

A visit by John Stewart and myself to the Craigmont Mine in B.C., where wet-mix shotcrete was under test, confirmed our decision. A highlight in the mine's experience was a 50 foot drift, just completed in finely shattered graphitic schist. It had been considered too dangerous to muck out without spiling. Shotcrete had been tried, applied above the muck pile, and stood as a monument of a simpler technique in caving ground.

Failing acceptance of bids, we participated in negotiating a contract with the low bidder, Northern Construction and J. W. Stewart Ltd., Noel Lambert, President. We remained responsible for construction and performance. Fred Langfeldt was project manager. The method, naturally, was received with scepticism.

Next, we both visited Switzerland, Austria and Italy, with the Aliva people, and in company of representatives of the contractors. Much of what we saw of interest chiefly in the mechanics of shotcrete application. An exception was a freeway tunnel in Italy advanced with shotcrete in soft,

wet, clayey rock, the perimeter being advanced with shotcrete first to receive steel ribs. The remainder of the cross-section formed a diminishing wedge-like shotcreted belly reaching some sets distant behind the face. There was some discussion as to the need of the steel.

Entry in the C.N.R. tunnel commenced August 1, 1966 with standard steel support in soft, water-logged, crushed and coaly shale. Converted quickly to top heading, forepoling was commenced. The surface drainage with the slow advance made quite a mess, slime pouring through the hay-packed lagging. It provided the opportunity to start shotcreting.

As normally, short sections of wall plates were advanced and a ring cut to accommodate the steel arch. The ring was excavated in short sections, each section being shotcreted before continuing with the next, the face also being held with shotcrete. As the ground improved the steel sets were extended to 4 foot centres. With the introduction of a stronger shale in the roof, two sets at 6 foot centres completed the steel rib and shotcrete supported section of the tunnel in rock. Certainly the crushed shale ground had improved with water control, but the shotcrete could be credited with limiting loosening at and beyond the face.

Subsequent advance of this heading was largely routine, such difficulties as occurred as could be expected of a newly introduced technique. The project was fortunate to

have its own mixing plant, with a quick drop to the tunnel, and grading was to Drogslers standards, and ready reference was had to Kobler's article in ACI Publication SP-14. Walter Lang of Aliva, Switzerland, initiated the shotcreting.

Control of shotcrete application and auxiliary reinforcement, and of blasting practice, was monitored on site throughout the working hours. Numerous photoelastic and three pressure cell stations were set up to monitor arch pressures. Perfo-bolted sections totalled 1,660 feet, emplaced radially in three, four and six-bolt patterns. Of this, 1,450 feet was required in the final 1,700 feet of brittle shales approaching break-through to the South Heading, which was driven with standard steel support in till. Total length supported with shotcrete was 10,742 feet. The section in till was 1,244 feet.

We early reached the conclusion the till section could very well have been done with shotcrete and bolting. Thus, in the construction of a 16-foot diameter ventilation shaft, it was sunk with shotcrete and fully grouted bolt support 70 feet through till, thence expanded in rock to form the Fan House at tunnel elevation. Shotcrete thicknesses were six inches on the arch and four inches on the walls, and without wire mesh reinforcement. Accelerators were used on the arch, and on the walls only when wet.

Following detailed inspection and mapping at the completion of the work, it was decided the shotcrete lining provided

an adequate final support. The rock shotcrete bond was examined by drilling at questionable locations, as we believed bond strength was of primary importance in gaining the composite structural rock mass-shotcrete effect. Instrumentation loadings had levelled off some time past.

A tendency towards minor cracking was found migrating from rock protruberances with a shotcrete thickness of one inch or less. A section of the arch where calcium chloride had been substituted when a satisfactory accelerator was unavailable had developed tension cracks soon after application. No further movement was found a year or more later.

At last inspection, April 1976, by John Stewart and C.N.R. engineers, no changes could be detected. Minor variations were found in the Gloetzl cells. Readings, however, showed pressure levels similar or lower than 1968.

We have worked since in a wide range of ground conditions, (though by no means the extremes documented in NATM literature) including participation in several instances of restoring caved ground in tunnels. On the whole such work as we took was confined to that we could direct and monitor performance. Major projects of lengthy duration were the 28 miles of Emisor Central outfall in Mexico City, and the central third of the 50 mile Orange and Fish rivers diversion tunnel in South Africa.

Shotcrete, properly mixed and applied, with or without fully grouted rock bolting, has been capable of coping with

any ground in our experience, including the restoration of caved ground, soft ground tunnelling, rock stabilization of canyon walls along existing railways and highways, and a number of shoring jobs in rock and soils.

We have preferred the fully grouted untensioned bolt, initially the perfo-bolt (which suffers from carelessness in drill hole tolerances); alternatively, a simpler rebar thrust into a fully grouted hole, essentially the Swedish SN bolt. Up-holes require a viscous grout, for which the Moyno pump is an effective placer. Bolting of this type was standard at one of our largest metal mines, used in spans up to 60 feet. We found this practice also existed in the South African gold mines.

Thus, in the Mexico project, our first work was in a transition from a 26 foot Boston horseshoe to two 20 foot spans at 150 foot depth. Maximum span reached was 56 feet, height 30 feet, the ground consisting of flatly-dipping, saturated sandy and silty tuffs of 28-70 psi compressive strength (2.0 - 5.0 TSF soil criteria). A top heading was advanced full face, once standing-time had been established, the core being lightly blasted, the perimeter hand trimmed, and the bench following in seven-eight days. An 18-24 inch bed of unconsolidated sand passed through the working, delivering surges of water to total deliveries of 350-400 gpm. Damage was caused by voiding at one springline, and massive voiding occurred approaching invert level. The invert reached a semi-liquid state complete with water boils.

The support consisted of eight inches of shotcrete without mesh and ten foot perfo-bolts without anchor plates at eight-foot centres, emplaced following the prime coat of four inches of shotcrete. Gloetzl cells registered 30 psi and 200 psi radial and tangential pressures respectively. At 250 days roughly, these increased to 100 psi and 400 psi as failure became evident in the walls. The walls had not been bolted. This deficiency being corrected, and some shotcrete and mesh pilasters added as a safety factor, the pressures levelled off. The work stood for three years before preparations began on the poured in-place concrete lining. The work was covered by R. E. Mason in a paper presented at your Shotcrete Conference in 1973.

Untensioned grouted rebars used without anchor plates are difficult to equate with the conventional anchor concept. In terms of bond strength it is possible to consider them as reinforcing elements. Their effectiveness in rock led us to test them in shoring vertical cuts in soil. In 1968, while the CNR was still under way, we used them successfully in clays ($q_u = 1.2$ to 4.4 TSF) to a depth of 35 feet. Movement was noted in the softer clays in one wall at an early stage of the work, obviously under-designed. Correction was made by increasing the reinforcement by decreasing its spacing, and stability was restored. We since have done 31 shorings in the Vancouver area, and a few outside.

The last completed August 15th last, was in Portland, Oregon, in dense sand with thin clay or silt lenses was up 44 feet depth. Initial construction was to tensioned tie-back standards. The repeated tensioning and relaxation against a four inch reinforced shotcrete wall in the first fifteen foot depth increased loosening of the soil measurably. The lower portion of the work was done to untensioned bond strength criteria.

Development of shotcrete usage on this continent has seemed to lean chiefly as accessory to conventional steel practice; as a substitute for lagging or embedment to stabilize steel ribs. A surprise was the early difficulties in reaching 4000 psi compressive strengths in the shotcrete in the early work in the United States, and the related flexurals that are probably of more importance in its use structurally.

Sweden for a number of years has used shotcrete and fully grouted bolts, often separately, divorced from steel supports, on an ad hoc basis. As such, operators with a background of underground experience are necessary, of which they are well stocked, in addition to engineers of related experience.

The shotcrete literature of mid-Europe suggests a gradual development from earlier underground practice, with the exception of the Austrian development. In Italy the change

in concept is sharper, no doubt impelled by difficulties encountered in Autostrada tunnel construction. No untypical terrain included masses of loam, clay and talus.

A visit with Art Chase to Quadrio Curzio's shotcreted advance through unconsolidated sands and gravels under old buildings in downtown Milan was an eye opener. Progress was slow by our standards, but settlement was the critical factor; less than that obtained by two competitive shields in alternative test sections. Shotcrete was used with rebar trusses as temporary and final lining.

The use of shotcrete structurally has lagged on this continent behind Europe. We did not experience the scale of steel shortages that occurred in post war Europe that impelled development of alternatives. Our long and satisfactory experience with steel supports, both in technical and contractual terms served to discourage a return to empirical on site determination.

Design of steel support can be quantified against rock loads in numerical terms. Pre-design could meet most ground conditions commonly encountered, and provide a measure of contractual costs. Spacing of the steel sets lends itself to meeting normal expectations in ground variables. To revert to on-site determination appears a backward step in design, and reopens an old can of worms between Owner and Contractor. Departure from proven technical and contractual practices requires a reasonable alternative.

Nor does shotcrete alone, or its use in a composite structure, offer the numerical evaluation equivalent to steel support. NATM is the most sophisticated approach to the use of shotcrete alone and in composite systems. It has been described as more than a design method, incorporating rather "design, construction and performance monitoring in an integrated manner". "Emperical dimensioning" is the founder's term to describe rationalization of pre-design by monitoring loads and displacements in the course of construction.

It would seem to provide a more rational and economically viable course than mathematical pre-design, particularly in difficult and very difficult ground projects. There is a broad range of ground conditions in which structural shotcrete and composite structures form a valid and often an economic substitute for steel supports; from the Swedish practice to the extremes of the Tauern and Tarbela tunnel structures. In the 80 feet x 70 feet transition in the latter, the employment of 130-200 lbs per linear foot steel sections were impractical. Included in the structure was a 15-25 foot bed of cohesionless sand in the quarter arch to be spiled and bolted in conjunction with shotcrete.

The primary function of shotcrete is to prevent loosening of an excavated surface, which at least provides gain in standing time. If its function also is to cope with the

rock in the latter's stress rearrangement, the shotcrete bond to the rock is of major importance even if the critical forces resisting displacement are provided by bolting. NATM practice uses light steel U-shaped sections (H.B. sections) as shotcrete reinforcement rather than to aid in supporting a loosening rock load. To quote Leopold Muller, "it is no longer necessary to consider the weight of de-strengthened rock which rests on the tunnel. It has only to consider the stress rearrangement which takes place after the rock is sealed".

There rests the anomaly in our current practice of using shotcrete as an appendage of standard steel support. It may be effective, but it hardly can be termed the best of two worlds structurally or economically.

However, application of the NATM concept or variations of same on this continent would tend to disrupt our formalized Owner-Contractor relationship. The engineering contribution if anything would increase, and become closer related to the actual ground conditions encountered in the course of the work. In Austrian practice, the Contractor co-operates and contributes to the insitu design. How such a relationship can be formalized contractually in our existing system remains to be seen.

RECENT RESEARCH ON SHOTCRETE BY THE
US ARMY CORPS OF ENGINEERS*

by

John M. Scanlon**

INTRODUCTION

During the July 1973 Engineering Foundation Conference on the "Use of Shotcrete for Underground Structural Support," Mr. Thomas Reading of the Corps of Engineers' Missouri River Division Office presented a paper entitled "Corps of Engineers Studies of Shotcrete." Mr. Reading is the Corps of Engineers' expert in this field, and due to his efforts, much interest has been generated within the Corps of Engineers to develop a greater knowledge of the potentialities of shotcrete. The Corps has conducted research and plans to continue to conduct research on shotcrete, especially in the areas of materials, protective structures, tunnel support, repairing poorly consolidated ("honeycomb") areas in new construction, and rehabilitating older structures. The Corps of Engineers has found shotcrete to be a very versatile material. An unofficial inventory was recently made of the number and type of concrete structures being maintained by the Corps and these results are indicated below.

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SHOTCRETE FOR GROUND SUPPORT

<u>Structure Age (years)</u>	<u>Multipurpose Projects</u>	<u>Flood Control Projects</u>	<u>Navigation Locks</u>
Under 10	15	54	39
10 to 20	15	52	36
20 to 30	17	54	13
30 to 40	5	45	46
Over 40	<u>1</u>	<u>0</u>	<u>111</u>
TOTALS	58	205	245

Since shotcrete is such a good repair and maintenance material, one can readily see why the Corps is interested in knowing more about it. My first experience with shotcrete came in 1966, when the Huntington District of the Corps of Engineers decided to use shotcrete to rehabilitate Charles Mill Dam in Ohio, which was built during the early thirties. The project construction specifications were based on information found in the American Concrete Institute's Recommended Practice for Shotcreting (ACI 506-66). Results of tests on this shotcrete caused the Corps to become increasingly interested in shotcrete for repair work. A number of panels were fabricated for evaluating the compressive strength of the shotcrete but in addition, panels were shot and sawed into 3-1/2 x 4-1/2 x 16 in. (8.9 x 11.4 x 40.6 cm) beams and at 28 days, these beams were subjected to the Rapid Freezing and Thawing Test (ASTM C 666). The results of these tests indicated that the shotcrete continued to gain flexural strength and the relative dynamic modulus of elasticity actually increased while being subjected to 300 cycles of freezing and thawing. There were probably two significant reasons for these results:

1. Manufactured fine aggregate was used since natural fine aggregate meeting the grade requirements was not readily available.
2. The use of the manufactured fine aggregate resulted to an excessive amount of rebound ($> 40\%$).

MATERIAL PROPERTIES

During 1974, Messrs. William Tynes and William McCleese of the Concrete Laboratory, US Army Engineer Waterways Experiment Station, investigated the two processes for producing shotcrete, the wet process and the dry process. The wet process consisted of adding all the ingredients of the mixture, including the water, and thoroughly mixing prior to application. The dry process consisted of thoroughly mixing all of the ingredients except the water in a mixing drum and then forcing the dry mixture through a hose where the water was added at the nozzle just prior to application. Specimens cored from panels of the shotcrete were tested to determine such physical properties as compressive strength, tensile strength, shear strength, permeability, bond pullout resistance of steel, density, absorption, and resistance to freezing and thawing. Significant results of this research program are:

1. Compressive strengths of shotcrete comparable to those of normal structural concrete were obtained for all four shotcretes evaluated (4000 to 8000 psi (27.58 to 55.16 MPa)).
2. The average shear strength percentages relative to the compressive strengths were 11.4 percent for the dry-mix, fine-aggregate shotcrete; 8.5 percent for the wet-mix, fine-aggregate shotcrete; 12.2 percent for the dry-mix, coarse-aggregate shotcrete; and 12.3 percent for the wet-mix, coarse-aggregate shotcrete. The

shear to compressive strength ratio of the shotcrete was approximately equal to a comparable ratio for normal structural concrete (8 to 12).

3. For the bonding of shotcretes to old shotcrete, the bond strength tests (shear and tensile) indicated approximately 100 percent bond efficiency. The bond efficiency for the shotcrete to old concrete was considerably lower than the bond efficiency of the shotcrete to old shotcrete.

4. The splitting tensile strength to compressive strength ratio of the shotcrete was approximately equal to the ratio of the shear strength to the compressive strength, i.e., 8 to 12. In general, this ratio is approximately equal to a comparable ratio for normal structural concrete.

5. The average resistance to freezing and thawing (DFE) values was 26 or below for all shotcretes tested. Since the shotcretes were not air-entrained, these low values were to be expected and were not lower than should be expected for nonair-entrained structural concrete.

6. The average bond pullout stress for the dry-mix, fine-aggregate shotcrete was 1200 psi (8.27 MPa); for the dry-mix, coarse-aggregate shotcrete, 1480 psi (10.20 MPa); for the wet-mix, fine-aggregate shotcrete, 790 psi (5.45 MPa); and for the wet-mix, coarse-aggregate shotcrete, 1050 psi (7.24 MPa).

7. The average permeability (K_c) values were 10.25×10^{-12} or less for all shotcretes tested. However, the dry process specimens were four times more permeable than the wet process specimens. These low permeability values indicate relatively impermeable shotcrete.

8. The addition of accelerators to the shotcrete did not result in any increase in the compressive strength at 7-days age; in fact, a slight decrease was noted.

9. In general, the epoxy-coated panels did not increase the bond strength of shotcrete to old shotcrete or old concrete.

10. The direct tensile test was the best test used in this investigation for evaluating the bond strength and uniformity of shotcrete. The shotcreting process results in a layering of the material as it is placed. Normally, placement of shotcrete is parallel to the direction in which test specimens are taken. In compressive testing, the layering effect does not significantly affect the test results. However, in the direct tensile tests, the layering effect is critical to the test results; hence, a better measure of the uniformity and quality of the shotcrete can be determined.

11. The percent rebound obtained on the dry-mix, fine-aggregate shotcrete was 36; on the dry-mix, coarse-aggregate shotcrete, 39; on the wet-mix, fine-aggregate shotcrete, 30; and on the wet-mix, coarse-aggregate shotcrete, 35.

12. The average water absorption values of 3.8 to 4.4 percent indicated normally dense shotcrete.

13. The average unit weights of the shotcretes were of magnitudes similar to those of conventional structural concrete (145 to 151 lb/ft³ (2323 to 2419 kg/m³)).

Reviewing these test results and by discussing application processes with the operators, the researchers preferred the wet

process over the dry process method due to more uniform results and slightly reduced rebound.

TUNNEL SUPPORT

In preparation for initiation of a research project to improve tunnel and rock cavity support systems, the Soils and Pavements Laboratory of the Waterways Experiment Station recently published Contract Report S76-4, which is a "State-of-the-Art Review on Shotcrete." It contains the results of four studies pertaining to shotcrete technology. The first part is an overview of American practices in shotcrete. The second is a discussion of the development and practice of shotcrete technology in Europe, including a description of the New Austrian Tunneling Method (NATM). The third describes the experiences of the US Bureau of Reclamation in using shotcrete for tunnel linings and their design and construction concepts. The fourth is a discussion of instrumentation currently employed for obtaining the data used in design of shotcreted tunnels being constructed by the NATM. The present research plans are to use a computer model, developed in 1975, together with actual documented failure cases, to create a geologic model of a typical tunnel failure and calibrate and verify the computer model. With the model calibrated to duplicate a field situation involving failures or impending failures, various tunnel support systems, i.e., steel sets, rock bolts, shotcrete, rockbolt-reinforced shotcrete, wire-mesh reinforced shotcrete, fiber-reinforced shotcrete, or various combinations will be evaluated for effectiveness in providing tunnel support. Additionally, using the calibrated computer model and a typical tunnel instrumentation plan, tunnel

instrumentation interpretative techniques will be investigated in an effort to assist the engineer in evaluating support requirements and anticipating possible failures.

REHABILITATION OF OLDER STRUCTURES

As mentioned in the introduction, the Corps of Engineers is very interested in economic procedures for rehabilitating older structures. Shotcrete is one of the most practical materials available since it does not require expensive forming. Very good laboratory results have been obtained with the use of short (1-in.)(25-mm) steel fibers in the shotcrete; the steel fibers increase spall resistance and since shotcrete has been shown to be relatively impervious, corrosion becomes a relatively insignificant problem. Shotcrete construction at the Dresden Island Lock and Dam, on the Illinois Waterway, has been one of the Corps' best experiences; the shotcrete was applied in 1953, and Figure 1 shows the appearance in 1976. Figure 2 is a picture of a 6-in. (15-cm) core taken from the lock wall during the summer of 1976. This shotcrete has been exposed to continuous wetting and drying, as well as freezing and thawing for the past 23 years. If research could assure such quality on all newly rehabilitated projects, there would be no question as to ability of shotcrete.

Three (3) representative shotcrete cores from the Dresden Island Project have been evaluated microscopically to determine the air-void content (ASTM C 457). The results of the evaluation are shown in Table 1.

Table 1
Micromatic Air Data^(a) for Shotcrete Cores from
Dresden Island Lock, Illinois Waterway

<u>Component</u>	<u>Location</u>		
	<u>North Wall</u>		<u>South Wall</u>
	<u>1</u>	<u>2</u>	<u>1</u>
Entrained Air, (%) ^(b)	2.3	1.8	1.6
Entrapped Air, (%)	1.5	0.9	1.4
Total Air, (%)	(3.8)	(2.7)	(3.0)
Paste, (%)	43.1	40.5	41.5
Aggregate, (%)	52.5	56.6	55.3
Wire Mesh, (%)	<u>0.6</u>	<u>0.2</u>	<u>0.2</u>
Totals	<u>100.0</u>	<u>100.0</u>	<u>100.0</u>
Air Void Spacing Factor (in.)	0.013	0.010	0.014

(a) ASTM C 457, examined at 125X

(b) Air voids 1 mm or less in diameter

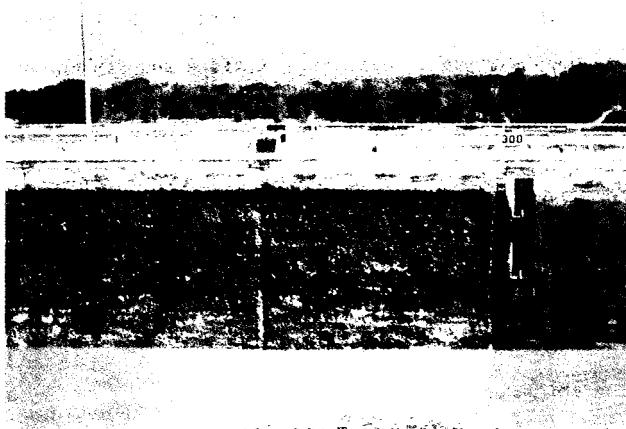


Figure 1. Dresden Island Lock and Dam



Figure 2. Dresden Island Lock and Dam Core



Figure 3. Fiber-Reinforced Shotcrete Domes
Prior to Placement



Whether the air was intended to be entrained or not, I do not know, but the resulting 2 percent average entrained air along with the relatively high strength has certainly provided the project with a relatively maintenance free surface.

PROTECTIVE STRUCTURES

The US Army Corps of Engineers' Construction Engineering Research Laboratory has been investigating various systems to develop field operational systems for constructing hardened shelters for use in theaters of operation. Their requirements are for a system that is rapid, economical, and of course requiring low skill to construct.

Fiber reinforced cellular shotcrete has been found to meet these requirements and domes measuring to 28 ft in diameter have been successfully constructed on the laboratory grounds in Champaign, Illinois. Figure 3 reflects the appearance of the placement prior to shotcreting. Figure 4 shows the completed 28-ft diameter by 14-ft-high dome with entrance. Figure 5 shows dome number 4, 18 ft in diameter by 10 ft high, being lifted from the construction platform by a truck mounted crane. This dome is 2-1/2 to 3 in. thick and weighs approximately 20,000 pounds. It is reinforced with 1 percent by volume of steel fibers; no other reinforcement was used. Only four lift points were required to successfully move the dome 75 ft into position for testing. This dome has been subjected to a simulated 4-ft underground burial by covering with 70 tons of sand. Dial gages monitor long-term deformations. As

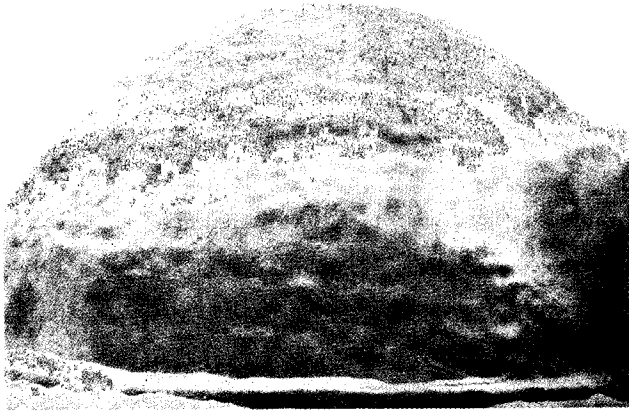


Figure 4. Completed 28-ft Diameter
Fiber-Reinforced Shotcrete Dome

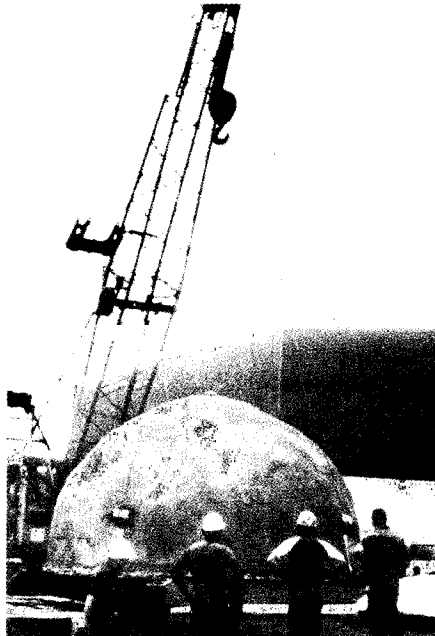


Figure 5. 18-ft Diameter Dome Being Lifted

of this time, no increased deformation over that experienced at the time of applying the load has occurred. The dome has been under load for 16 months.

SUMMARY

As you can see, the Corps of Engineers is very much interested in the use of shotcrete and envisions many economical and desirable uses of the system. We are very interested in the physical properties of shotcrete as well as its ability to be used in rapid construction, as a repair material, for tunnel supports, and for protective structures.

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PROPERTIES OF SHOTCRETE ON CONSTRUCTION PROJECTS

Philip T. Seabrook*

INTRODUCTION

The Author has been involved in the materials aspects of shotcrete construction since 1965. At that time there were no applicable codes or testing standards available on which to base conformance testing, so it was necessary to develop appropriate in-house procedures.

This paper relates to shotcrete properties as determined from tests conducted on samples prepared at construction projects. The Author's experiences relate to projects covering a range of shotcrete work in underground mining, restoration, channel lining, slope protection and foundation stabilization. However, in order to present shotcrete properties on a comparable basis, information here will be confined to those test results obtained from a series of ten foundation support projects recently constructed in the Vancouver area.

Test procedures referenced were developed by the Author in consultation with the staff of Dolmage, Mason and Stewart, who were the prime shotcrete consultants on the majority of the projects. The procedures have evolved since 1965 with periodic modifications to obtain evaluations of shotcrete properties in a form which would provide the quality control for the prime consultants supervision

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functions. Readers will appreciate that this has been a unique opportunity to objectively develop testing procedures which directly provide the information required for engineering a system without the necessity of strict conformance to conventional codes.

SHOTCRETE PERFORMANCE REQUIRED

Foundation stabilization systems are often demanding on the performance of the support materials, in this case, shotcrete. There are close parallels with the support of unstable ground in underground excavations.

The shotcrete is required to:

- have high early strength gain to prevent soil sloughing or weather slaking;
- bond to the soil system to permit "membrane" action with the ground system (and, in some cases, the soil anchors);
- be sufficiently flexible to accommodate localized strains and provide load redistribution - again a "membrane" action;
- be sufficiently impermeable and/or set rapidly to arrest ground seepage.

Ideal properties are therefore rapid set, early strength gain, bond to the native soil and flexibility.

Experience indicates that these ideal properties can be achieved using dry mix shotcrete with the addition of accelerating admixtures and with mix proportioning optimizing rapid and high level flexural strength gain and high in-place density.

CONSTRUCTION SPECIFICATIONS

Following is an outline of the materials portion of construction specifications which have been evolved by Dolmage, Mason & Stewart for foundation stabilization shotcreting.

(a) Aggregates

Aggregates are required to be provided in two sizes, sand and 3/8 inch x No. 4 stone. They are blended to achieve the combined gradation limits shown in Figure 1.

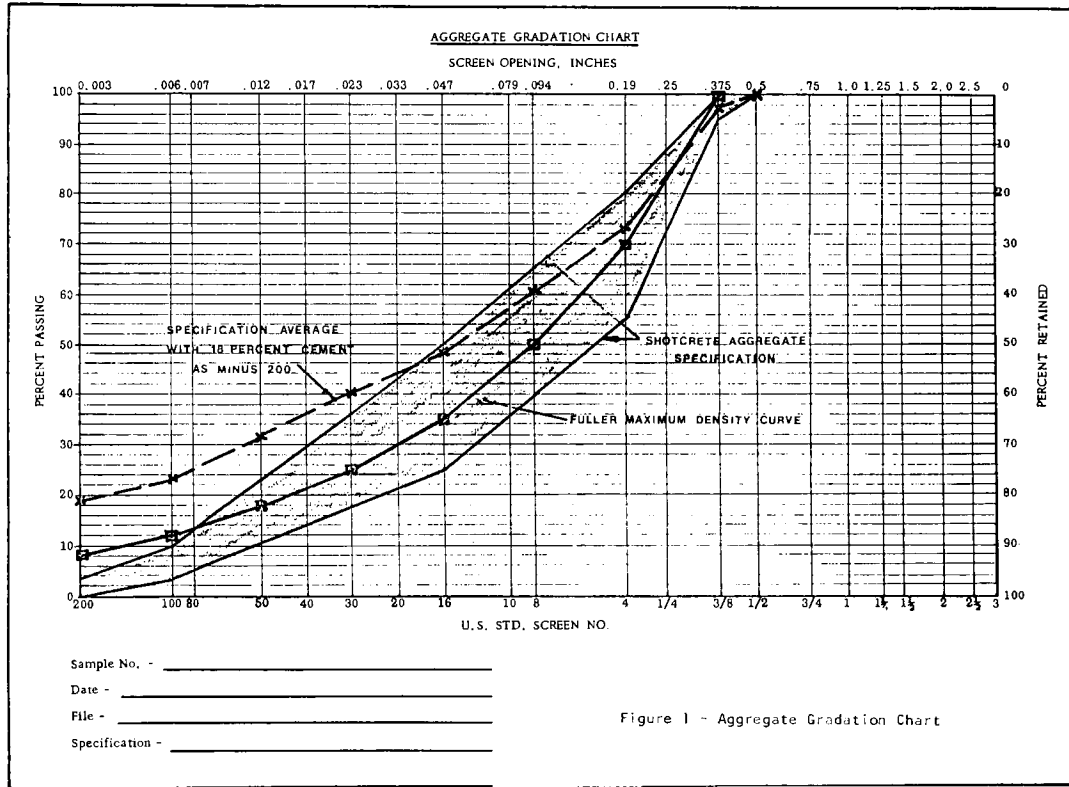
Superimposed on Figure 1 is the Fuller theoretical maximum density curve which was used to develop the original specification. The Figure also shows the overall particle size distribution that would result from aggregates blended to the median of the specifications with a normal cement content added.

Aggregate particles are required to meet conventional ASTM C33 limits for concrete aggregate with the following additional criteria:

Maximum passing No. 200 screen -

stone	-	0.25%
sand	-	2.0%

The stone limit was imposed to provide maximum particle bond and therefore flexural strength. The sand limit was originally intended to assist in minimizing workmen hazards from airborne dust. (Not a major problem in open excavation.)



(b) Shotcrete Strength

Minimum performance required is:

	Compression (psi)	Flexure (psi)
1 day	-	300
7 day	3000	450
28 day	4000	600

(c) Density

Properties required at 7 days:

boiling absorption	-	6-1/2% maximum
volume permeable voids	-	12% maximum

(d) Mix Design

A typical mix design used for the projects was:

Cement, Type 10	700 lb
Stone, 3/8" x No. 4	1350 lb
Sand, No. 4 x dust	1820 lb
Water, net through nozzle	245 - 275 lb
Accelerator, powder type	3 - 4% of cement

TEST PROCEDURES

Shotcrete properties are evaluated by the following procedure:

1. Sampling

A panel is prepared on the site during shotcreting by using a 2 ft x 2 ft x 4 inch plywood-backed form supported vertically. The panel is protected by a plastic sheet cover

until it can be transported to the laboratory.¹

Test samples are obtained by cutting with a diamond saw from the central portion of the panel. Normally, eight 3-inch cubes and three 3 inch x 3 inch section beams are obtained. Cubes are tested in pairs with the average constituting a test.

Two cubes are used for the density determinations and the remainder for:

- 2 at earliest possible test age, nominally one day;
- 2 at 7 days;
- 2 at 28 days

compressive strength determinations. For flexural strength, sufficient beam length is obtained (about 18 inch) to permit two tests on one sample. Test ages are as above.

2. Curing

Initial strength tests are conducted on samples as received (without any temperature or moisture conditioning). They therefore provide an appraisal of the in situ early age conditions. Samples are subsequently cured at $73 \pm 3^\circ\text{F}$ in a fog room.

Density samples are cured similar to strength samples. They are normally tested at about 7 days although experience indicates that there is no major improvement in void and absorption characteristics after three days.

¹ If the panel is shot early in the day, it can be moved without damage later the same day, otherwise it is transported early the next day.

3. Strength Tests

- Compression - ASTM C42 with the direction of loading perpendicular to the direction of shooting
- Flexure - ASTM C78 (One-Third Point Loading) with the direction of loading parallel to the direction of shooting

Loading directions have been selected to simulate the field conditions.

One-Third Point flexural loading has been found to give a better overall evaluation of shotcrete quality since the constant mid-span moment, to a degree, avoids testing the inevitable localized shotcrete structure variations.

4. Density Tests

Procedures of ASTM C642 are used.¹

PROJECT PARAMETERS

It was indicated above that a group of projects had been selected in order to provide comparable information. These projects span a three-year period and all involved the use of shotcrete to stabilize deep excavation walls (shotcrete shoring system). Construction was by a single shotcrete contractor and supply of the material from a single central mix plant. Following are the common elements:

¹Concrete technologists will appreciate that this test procedure does not provide an evaluation that directly correlates with concrete durability or permeability. However, experience indicates that it is a realistic evaluation of the density of the shotcrete structure.

- Shotcrete sprayed by dry mix method using a Mayco Gun.
- All mixes had powdered accelerators added at the mixing pot.
- Single type of aggregate - 3/8 inch maximum size, glacio fluvial origin consisting of hard sound particles predominantly granitic.
- Mix transported in ready-mix trucks.

Foundation wall areas shotcreted ranged from 6,000 to 70,000 sq ft.

TYPICAL SHOTCRETE PROPERTIES

Figures 2 and 3 present the age-strength curves for flexural and compression tests from the projects. A best fit line and 20% dispersion limit are shown. It is apparent that there is almost a linear relationship between both strengths and the log of the curing time - this indicates that the shotcrete is performing in accordance with the Plowman Maturity Principle for conventional concrete.

It will also be evident from Figures 2 and 3 that the specified strength levels have readily been met. Twenty-eight day compressive strengths in the order of 7000 psi and flexural strengths of 1000 psi were achieved.

Curves in Figures 2 and 3 have been extrapolated backwards to an approximate "zero" strength age as determined by previous tests.

Figure 4 provides more interesting information. It shows that flexural strength of shotcrete is a far higher percentage of compressive strength at early rather than later age.¹ It is thought that

¹After about 3 days the compressive : flexural ratio is about the same as would be expected from conventional concrete produced from the same aggregate and cement as used here.

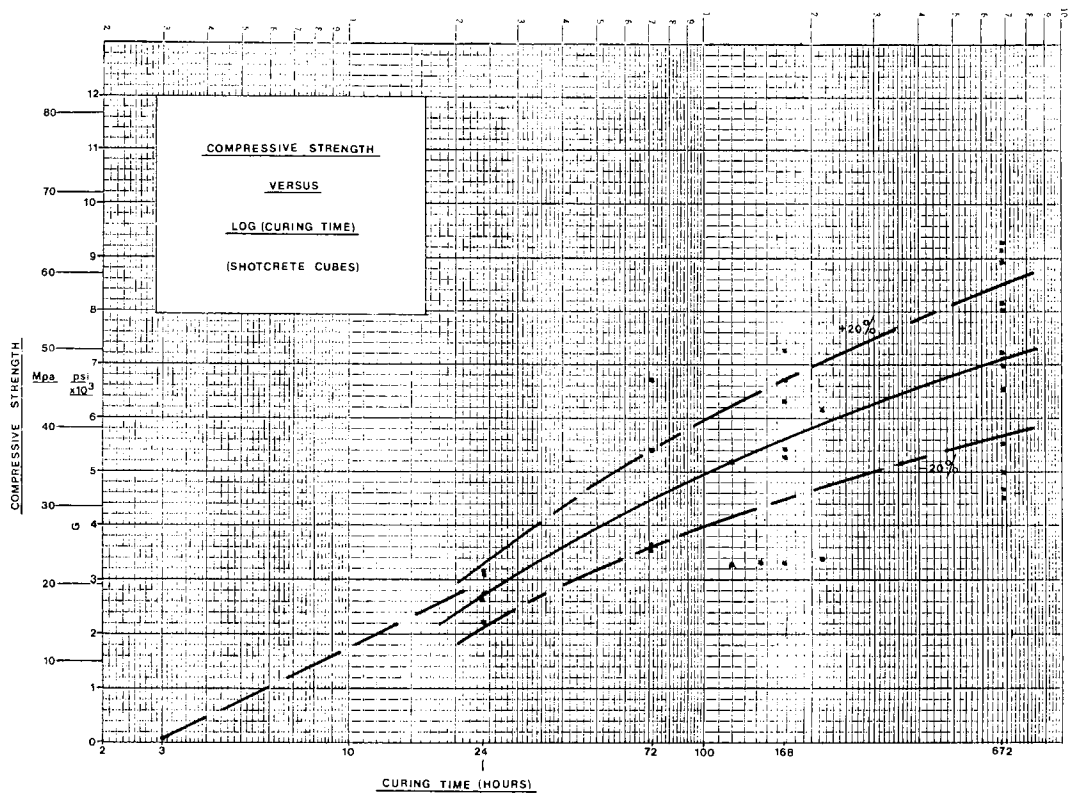


Figure 2 - Compressive Strength versus Curing Time

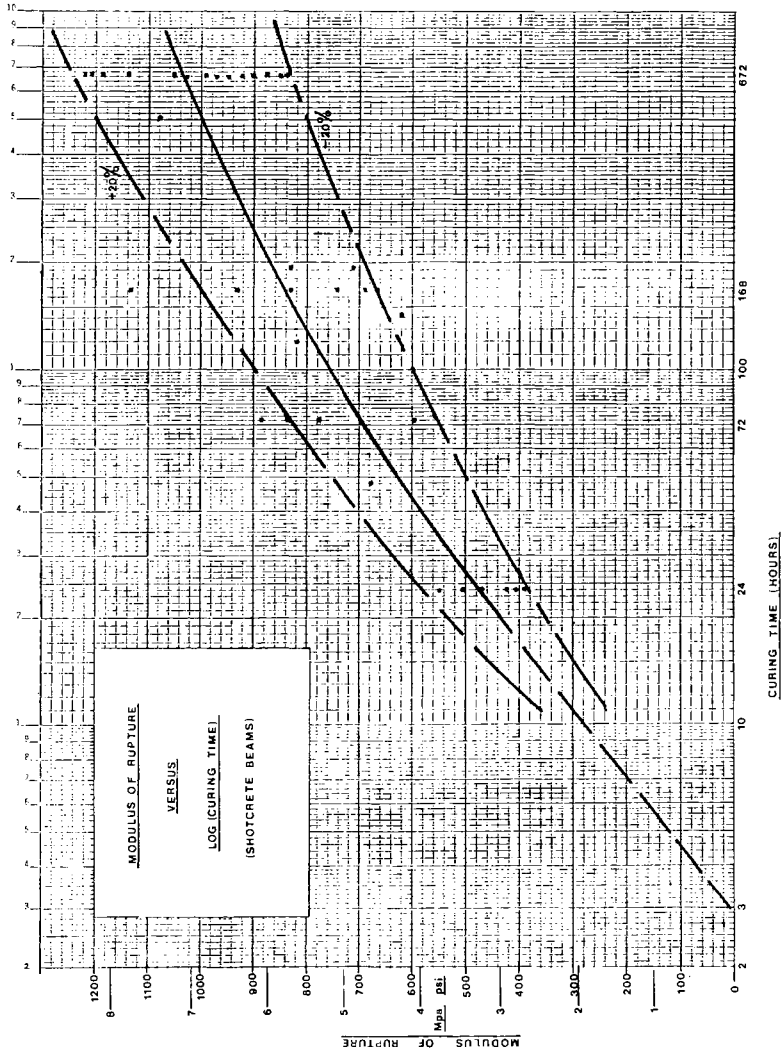
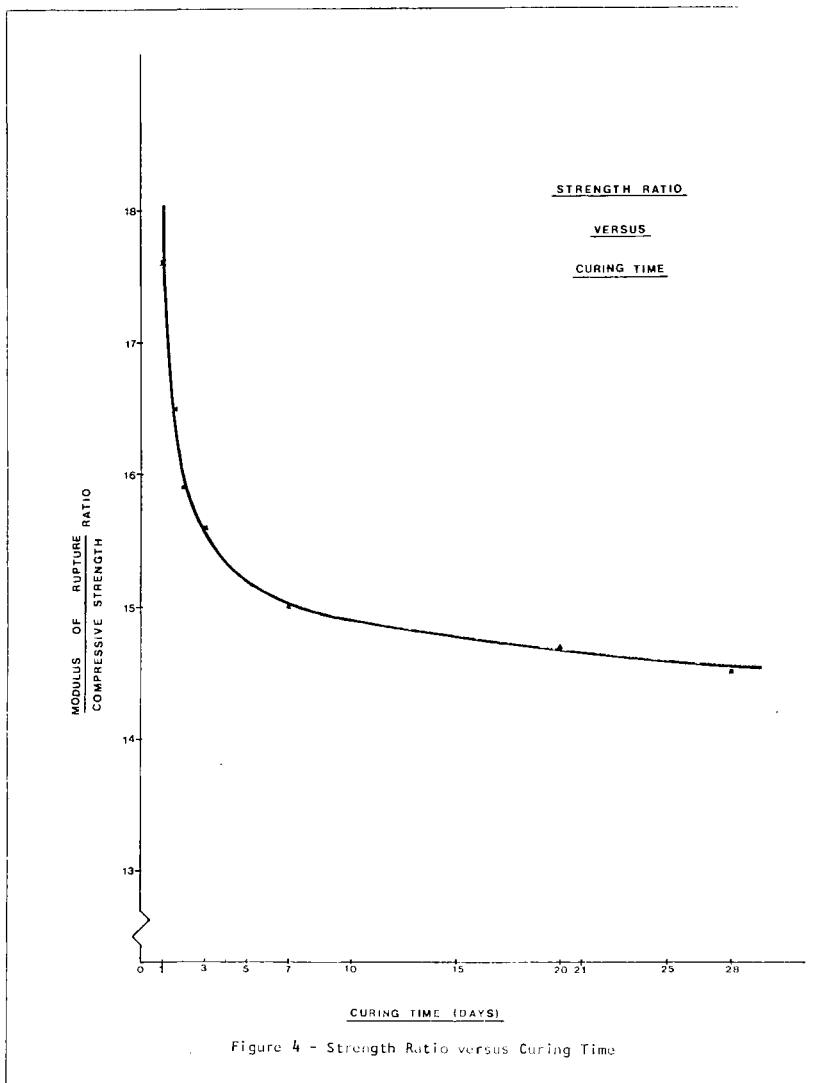


Figure 3 - Flexural Strength versus Curing Time



this phenomena is related to a combination of:

- the action of the accelerator;
- the high aggregate bond strength resulting from coating of stone particles in dry mix shotcrete;
- the lower stone content of an as-shot mix compared with conventional concrete.

Test results provide an interesting relationship between flexural and compressive strength. Figure 5 shows results obtained from the best fit curves of Figures 2 and 3 and indicates that the relationship can be generally represented by:

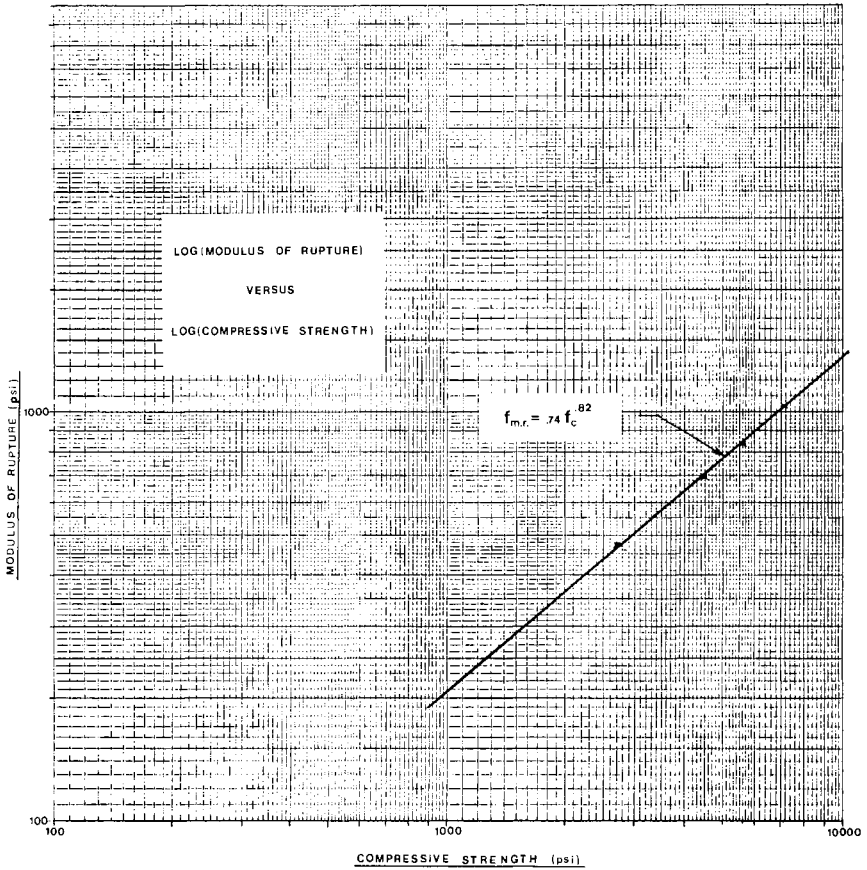
$$\begin{aligned}f_{mr} &= 0.74 f_c^{0.82} \\ \text{where } f_{mr} &= \text{flexural strength (modulus of rupture)} \\ f_c &= \text{compressive strength}\end{aligned}$$

as determined by test procedures here.

Tests on the shotcrete showed values of voids and boiled absorption generally within the 12 and 6-1/2% limits respectively of the specifications. Figures 6 and 7 show the correlation of these values with both forms of strength test. There is a general relationship between the absorption and strength, as represented by the best fit lines on the Figure, although the scatter for absorption versus flexural strength is appreciable.

High quality conventional concrete, in the order of 5,000 psi compressive strength and 0.45 water : cement ratio, would have voids and absorption values in the same order of magnitude as these results.

The fact that strength is generally a function of density is not surprising and not inconsistent with conventional concrete technology.

Figure 5 - Log Modulus of Rupture versus f_c

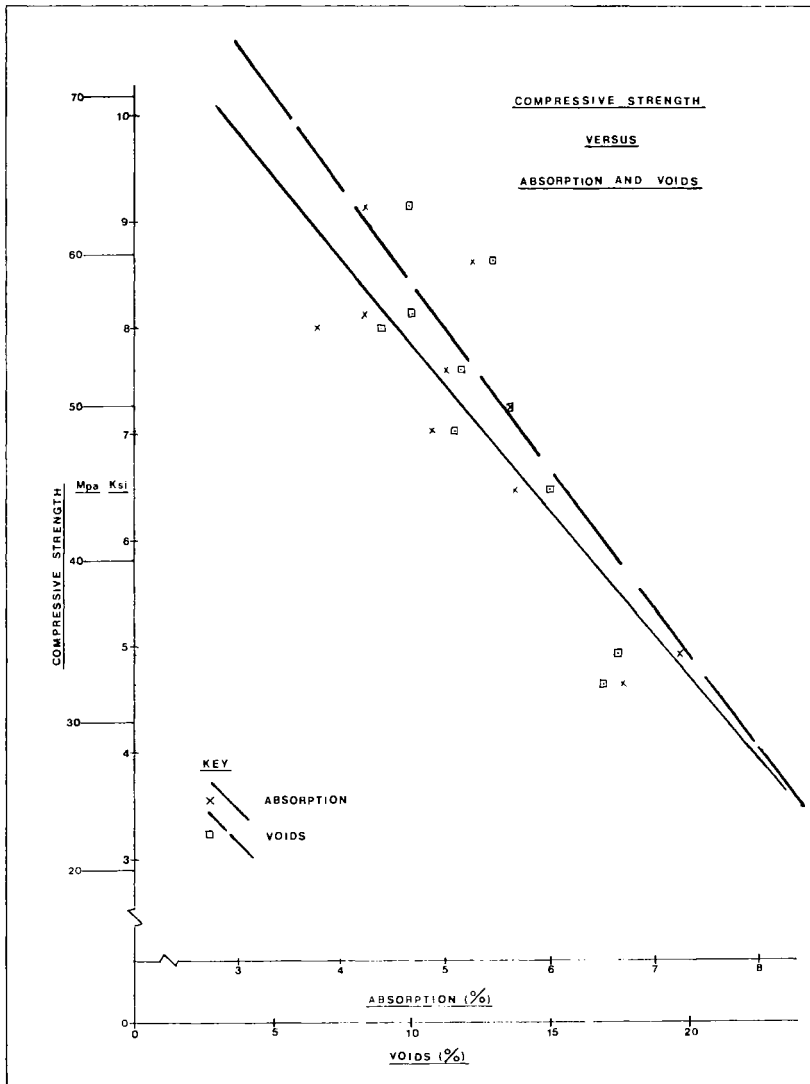


Figure 6 - Compressive versus Absorption and Voids

SUMMARY

This paper describes shotcrete test procedures which have been developed to provide materials quality control for foundation stabilization programs. Typical test results are presented.

The test procedures have been developed to specifically provide engineers with the type of information required to complement their design systems. Necessary information primarily includes evaluations of early age flexural strength and shotcrete density. Experience indicates that the procedures are practical, effective and economical.

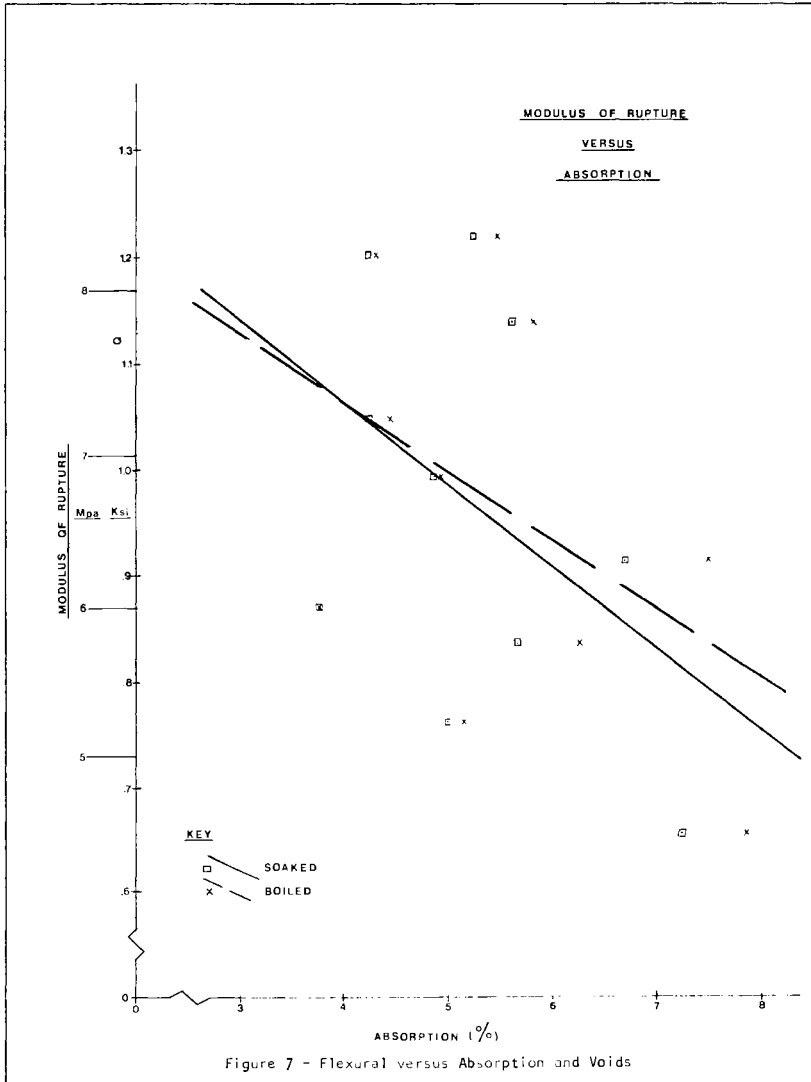
Test results show that shotcrete has properties such as:

- age : strength relationships;
- strength : density relationships

similar to conventional concrete. It is capable of being produced to high strength and density levels. A unique feature of shotcrete, and one that partially accounts for its effectiveness in early ground stabilization, is its high early age flexural strength.

There are no present generally referenced test procedures for shotcrete. It therefore presents a unique opportunity for engineers to develop procedures that will directly complement their design systems.

The Author suggests that more emphasis in this direction is preferable to expenditures of technical time and energy in debating refinements of test procedures.



PROPERTIES OF SHOTCRETE ADMIXTURES

by
Raymond J. Schutz¹

Accelerators are commonly used in both the wet and dry process Shotcrete to impart accelerated setting and accelerated strength gain. It is not the purpose of this paper to discuss why accelerated strength gain and setting time are of advantage in ground support, but rather to explain what these accelerators are and how they perform.

Shotcrete accelerators are composed of either soluble inorganic aluminates, carbonates, silicates, or combinations of these chemicals or proprietary organic chemicals. These four classes of chemicals act differently on the cement/water reaction.

The early reactions of Portland Cement and water may be described schematically as shown in Figure 1 and 2. Reaction of the tricalcium aluminate and water, as shown in Figure 1, can be considered to take place usually within approximately two hours. The rate of reaction of the tricalcium aluminate is controlled or retarded by the formation of ettringite which precipitates on the tricalcium aluminate thereby inhibiting the rate of reaction.

The reaction of tricalcium silicate (Figure 2) is also inhibited by the precipitation of lime and hydrates. It may be considered that the rate of early stiffening is controlled by the tricalcium aluminate, and the rate of early strength gain by the reaction of the tricalcium aluminate.

¹Vice President, Research and Development
Sika Chemical Corporation

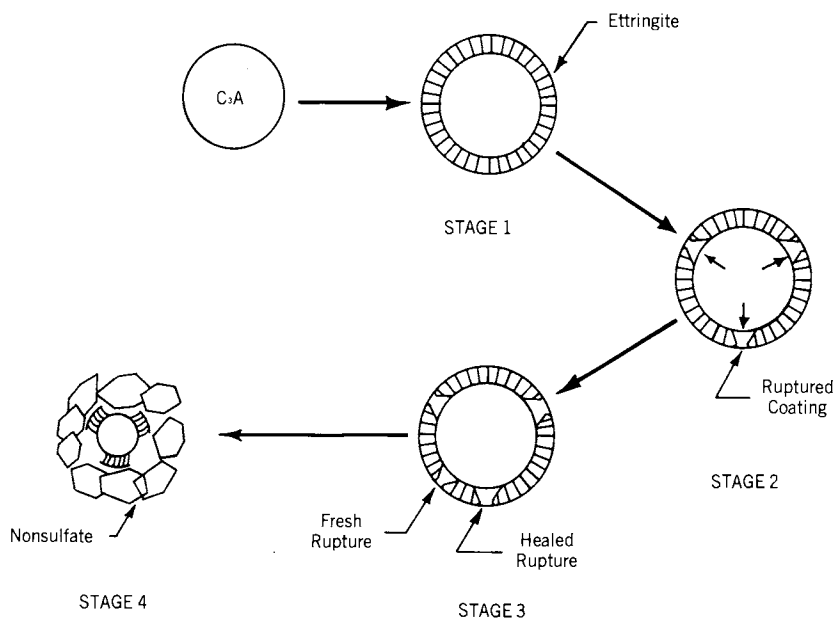


Fig.1. Schematic Representation of the Reaction of Tricalcium Aluminate and Water (Reaction Mechanics of Organic Admixtures with Hydrating Cement Compounds, J.F. Young)

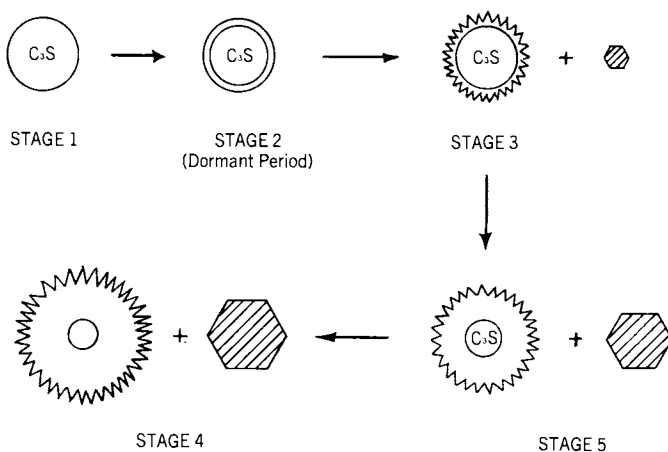


Fig.2. Schematic Representation of the Reaction of Tricalcium Silicate and Water (Reaction Mechanics of Organic Admixtures with Hydrating Cement Compounds, J.F. Young)

Action of Accelerators and the Cement/Water Reaction

The soluble aluminates - - when used in Shotcrete, or more correctly with Portland Cement - - will offset the balance shown in Figure 1. This occurs because the soluble aluminates combine with the gypsum, preventing the formation of ettringite around the cement grain. This will allow the tricalcium aluminate to flash set, providing the early stiffening desired in many Shotcrete applications.

The soluble carbonates mainly influence the reaction of the tricalcium silicate and water. The soluble carbonates will accelerate early strength gain but tend to have little effect on initial set. Actually, the soluble carbonates tend to retard the effect of the soluble aluminates on initial set. Since proportions of C_3A and C_3S will vary depending on the source of the cement, the percentage of aluminate and carbonate in the accelerator should be varied to suit the particular cement composition. Figure 3 illustrates the effect on setting time and strength of carbonates and aluminates.

Generally speaking, the formulator will manufacture a Shotcrete accelerator to fit the average cement composition as shown in Figure 4. However, this is an average, and the cement used in a particular mixture may vary tremendously from the average, as shown in Figure 5. This is an excellent cement, but the compound composition is such that a Shotcrete accelerator which is compatible with an average cement would be incompatible with this cement. Therefore, compatibility tests should be made before the job starts with the cement to be used. Both the carbonates and aluminates will decrease ultimate strength of Portland-Cement mixtures.

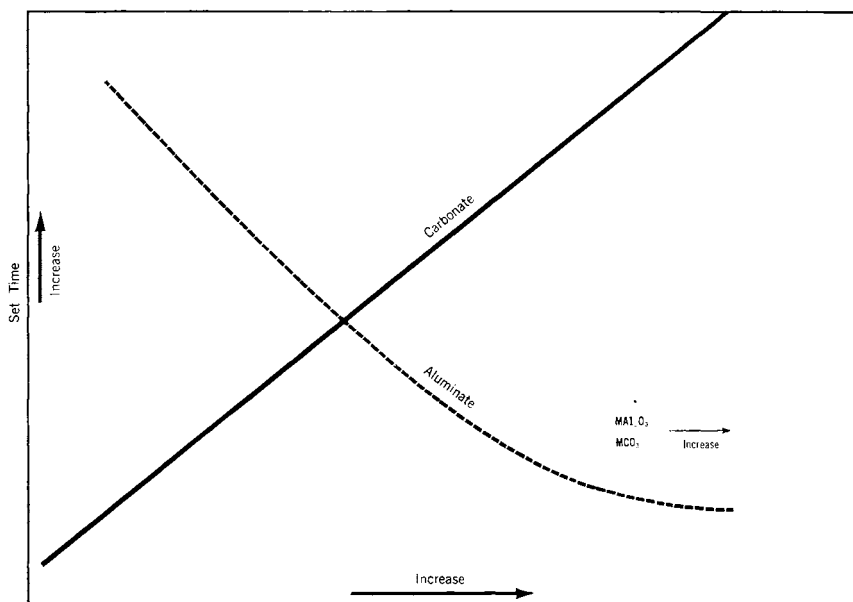


Fig.3. Effect of Carbonates and Aluminates on the Setting Time of Shotcrete

	Type I Standard	Type II Moderate Heat	Type III H.E.S.	Type IV Low Heat	Type V Sulphate Res.
C ₃ S	48	43	57	20	39
C ₂ S	27	30	20	52	33
C ₃ A	12	7.5	11	6	4.5
C ₄ AF	8	12	7	14	16

Fig.4. Average Compound Composition of Present-Day Portland Cement (Technology of Cement and Concrete, Blanks and Kennedy)

C ₃ S	46.9
C ₂ S	25.6
C ₃ A	4.2
C ₄ AF	16.7

Fig.5. Compound Composition of Cement Used at Morie Colorado Project

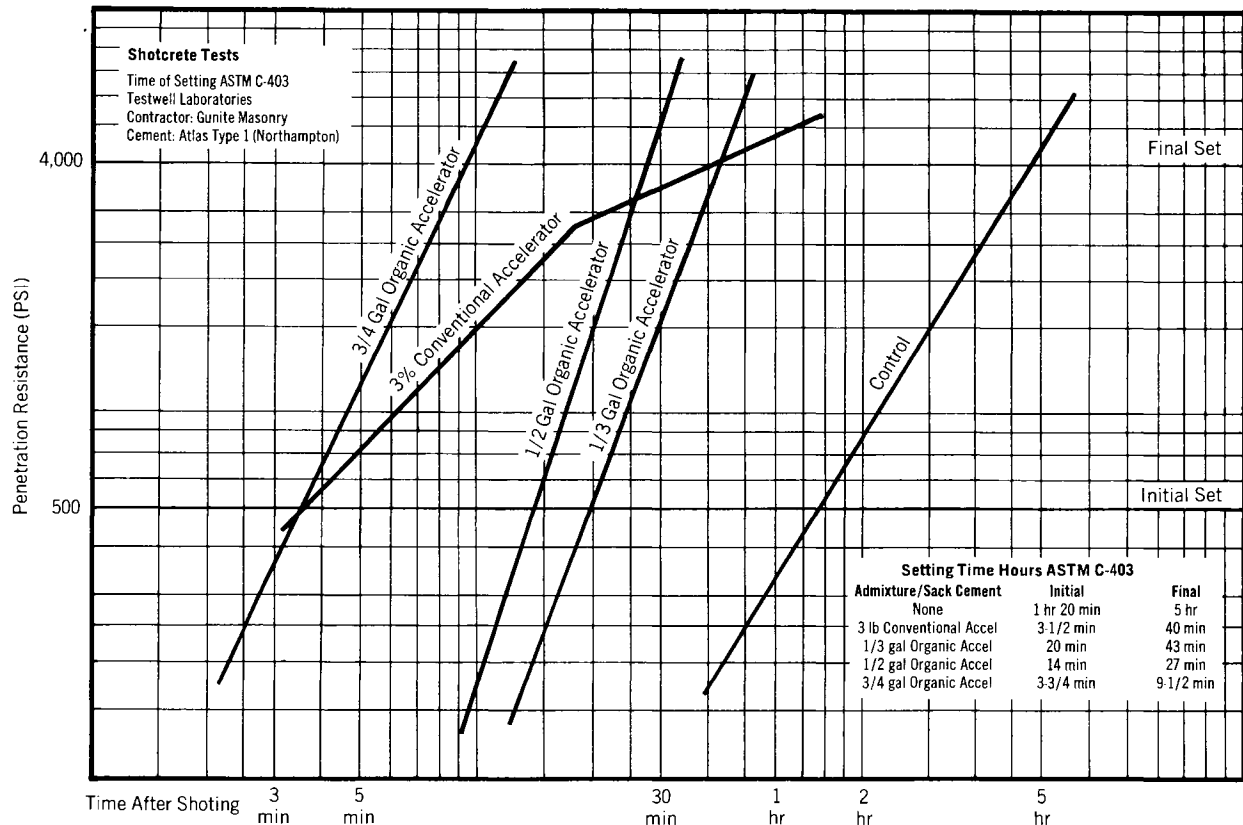


Fig.6. Setting Time of Shotcretes

ADMIXTURE PROPERTIES

The soluble silicates induce quick-setting by precipitating as calcium silicate. Although inducing very early set, the silicates will reduce ultimate strengths significantly by their interference on the cement/water reaction.

The organic accelerators accelerate the cement/water reaction by solubilizing the lime and ettringite which normally precipitates on the cement constituents thereby allowing the cement/water reaction to proceed unretarded. The organic accelerators generally produce no strength penalty, as they do not change the basic cement/water reaction. They only allow the reaction to proceed at an uninhibited rate.

Figure 6 shows the typical setting time characteristics of Shotcrete accelerated with a conventional carbonate/aluminate accelerator and with an organic accelerator. Note that the conventional accelerator induces early set, but strength gain after set is slow. The organic accelerator induces early set and also induces accelerated early strength. Figure 7 indicates the typical 28-day strengths obtained with conventional and organic accelerators. The strength penalty imparted by the carbonate/aluminate accelerators is very typical, and can be expected when using these accelerators.

The preceding data refers mainly to the dry-process Shotcrete. With wet-process Shotcrete, premature stiffening can be induced by the accelerators mentioned, but accelerated early strength is generally not obtained due to the fact that the cement constituents have already reacted with water at the time the accelerators are added. The reaction being partially complete, it cannot be accelerated significantly.

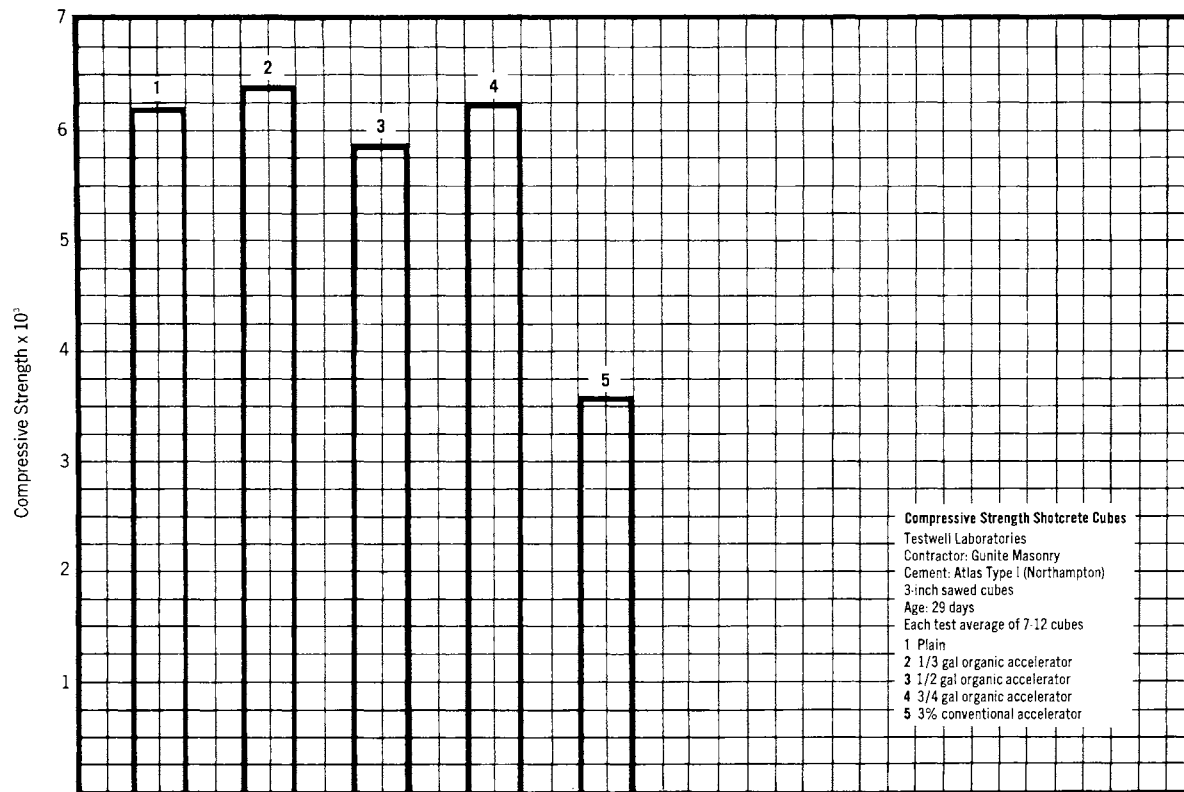


Fig.7. Compressive Strength of Shotcrete Cubes