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COMPOSITE REINFORCED SHOTCRETE AND
COST REDUCED SHELL CONSTRUCTION

by

• David William Large, B.Eng.

A THESIS SUBMITTED TO THE FACULTY OF GRADUATE STUDIES
AND RESEARCH IN PARTIAL FULFILLMENT OF THE REQUIREMENTS
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ABSTRACT

Construction of reinforced concrete shells is avoided in Canada because of higher costs relative to other long span roof systems. Expensive labour intensive temporary formwork is a major cause. With the intent of reducing the need for temporary formwork, this thesis proposes a composite reinforced shotcrete shell construction system and evaluates its strength and costs.

It is assumed that an arbitrary industrial building plan provides an accurate basis for an evaluation of budget construction costs. A series of industrial buildings with various long span roof systems, including hypar shells, is designed and costed. Results of the investigation show that the use of fibreglass fabric/steel grid composite reinforced shotcrete reduces the cost of shell construction, compared to traditional methods, by two means; the strength contribution of the woven fibreglass fabric decreases the amount of reinforcing steel required, and the system as a whole decreases the need for temporary formwork. Furthermore, with modifications that would completely eliminate temporary formwork, the system has the potential to be cost-competitive with today's commonly used long span roof systems.

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- I. Rigid Cable Net Concept

LIST OF SYMBOLS

a	plan dimension of hyper element
A_g	gross area of concrete cross section
A_{st}	steel area
b	plan dimension of hyper element
c	rise of hyper element
C	center, compressive load
c/w	complete with
δ	delta, change in dimension
D	delta, change in dimension
(E)	expected value
f_c	compressive strength
f_c'	28-day compressive strength
GCA	Gunite Contractors Association
H	horizontal
K	kip, one thousand pounds
ksi	kips per square inch
$\frac{1}{4}$	plan dimension of square test plate
L	left
O.D.	outside diameter
P	load, strength
P_u	ultimate load
psi	pounds per square inch
psf	pounds per square foot
ϕ	capacity reduction factor
R	right
RC	reinforced concrete

s reinforcement spacing
t thickness
T thrust, ton
v shear stress
V vertical
w uniformly distributed load

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CHAPTER 1

INTRODUCTION

1.1 Problem Definition

The problem addressed by this thesis is that construction of reinforced concrete shells is avoided in Canada because of higher costs relative to other long span roof systems.

Concrete shell construction reached a peak around the world in the 1950's and 1960's, witnessed by no fewer than fifteen major international conferences (1-15). Applications for reinforced concrete shell roofs in the building industry ranged from houses, restaurants, schools and churches to pavilions, grandstands, auditoria, arenas, factories, warehouses, terminals and hangars. Unfortunately, the number, size and variety of shells built in Canada were not as great as those in other countries. Canadian contractors preferred familiar long span roof systems and were reluctant to bid on projects requiring a largely indeterminate cost for labour intensive temporary formwork (19). The advent of industrialized modular space structures (16, 17, 18) further discouraged reinforced concrete shell construction, so that only special projects such as the Toronto City Hall dome (20) and the Montreal Olympic Velodrome (21) have recently been built.

Today a designer in Canada has two choices when faced with a project where reinforced concrete shells could be applied; either ignore shells and select a less expensive long span roof system, or accept shells and specify a construction system which reduces requirements for labour intensive temporary formwork.

This thesis will explore the second alternative in an attempt to find a solution for the problem of non-competitive shell construction costs arising from excessive requirements for labour intensive temporary formwork.

1.2 A Solution

Using traditional methods, a reinforced concrete shell is essentially constructed twice; a temporary wooden shell is overlaid by a steel reinforcement grid and completed with concrete or shotcrete (air-placed concrete). A major labour cost reduction could be achieved by eliminating the need for the initial temporary structure.

Consider this hypothesis; if a structural fabric with small openings is attached to a freestanding reinforcement grid, and the combination is shotcreted on both sides, then the need for a temporary shell surface has been eliminated. The fabric acts as both a form and permanent supplementary reinforcement, and labour costs have been reduced compared to traditional construction methods.

Therefore, a composite reinforcement shotcrete system represents a plausible solution for the defined problem of non-competitive shell construction costs arising from excessive requirements for labour intensive temporary formwork.

1.3 Scope of Thesis

With the goal of furthering the state of the art of reinforced concrete shell construction in Canada, it is the main objective of this thesis to investigate the hypothesis that costs of reinforced concrete shells can be reduced by the use of a composite reinforced shotcrete system. A specific application to hypar shell construction is examined.

The scope of this thesis is defined by three sub-objectives:

- (1) Illustrate clearly the derivation and details of the proposed reinforced concrete shell construction system.
- (2) Determine the structural properties of the proposed system by a lab test program.
- (3) Determine the cost reducing potential of the proposed system by a building construction cost program.

CHAPTER 2
COMPOSITE REINFORCED SHOTCRETE SHELL
CONSTRUCTION SYSTEM PROPOSAL

2.1 Reinforced Concrete Shells

The selection of reinforced concrete shells for study must be defended. First, consider these defining properties of structural thin shells (22, 23):

- (1) A shell is a three dimensional load carrying element of small thickness (as small as two inches or less) and of large span (as large as two hundred feet or more) which combines architectural and structural form and function.
- (2) A shell derives its load carrying capability from a curved surface or combination of surfaces.
- (3) A shell may be singly curved or doubly curved (either synclastically like a dome or anticlastically like a saddle).
- (4) A shell transfers its loads primarily through the action of in-plane tension, compression and/or shear stresses because flexural stress is intentionally minimized by the absence in most cases of an axis of bending. A shell is therefore thin because bending moment need not be resisted. However, caution must be paid to local bending and buckling (15).
- (5) A shell may be formed by rectilinear or curvilinear generators by moving the generators in translation, rotation or revolution, or by any combination thereof.

The definition of shells can be extended to include folded plates, flexible membranes (tents, cable nets and bubbles) and ribbed or skeletal surfaces, but, unless otherwise stated, the term "shell" in this thesis is

limited to a reinforced concrete rigid continuous surface of uniform thickness.

Besides the general features of combined architectural and structural function, and efficient use of structural material when compared to other long span roof systems, reinforced concrete shells have a number of particular features:

- (1) Concrete has excellent architectural flexibility because of its moldability.
- (2) A reinforced concrete shell has good vibration, acoustic, fire, insulation and weatherproofing characteristics when compared with flexible membranes.
- (3) Reinforced concrete shells have good market diversity (plates 2.1.1 to 2.1.6).

Because of these features, the study of reinforced concrete shells is justified.

PLATE 2.1.1

Short Single Span RC Shell; Bubble House,
Florida, 1954 (source - ref. 46, p. 137)

PLATE 2.1.2

Short Multiple Span RC Shell; Restaurant,
Mexico, c.1955 (source - ref. 47, p. 31)

PLATE 2.1.3

Medium Single Span RC Shell; Church, Mexico,
c. 1955 (source - ref. 47, p. 33)

PLATE 2.1.4

Medium Multiple Span RC Shell; Priory Chapel,
Missouri, c. 1964 (source - ref. 48, p. 12)

PLATE 2.1.5

Long Single Span RC Shell; Garden State Arts
Center, New Jersey, c. 1970 (source - ref. 49,
p. 198)

PLATE 2.1.6

Long Multiple Span RC Shell; Grandstand, Spain,
1935 (source - ref. 46, p. 204)

2.2 Reinforced Concrete Hypar Shells

The selection of hypar shells for study must also be defended.

Consider these properties of hypars:

- (1) A hypar is a synclastically doubly curved surface formed by moving straight line generators in translation and rotation. A vertical intersecting plane defines a parabola and a horizontal intersecting plane defines a hyperbola (24) (figure 2.2.1).
- (2) For a small (a and b less than 50 feet) and shallow (c less than a or b) hypar element subjected to a uniformly distributed load, the resulting uniform in-plane shear stress can be approximated by a simple equation (25) (figure 2.2.2). The shear stress is then "collected" by axially loaded edge beams and carried to points of support. Stress analysis of larger and non-shallow hypars, including bending and buckling considerations, has been facilitated by theoretical research (26) and many numerical techniques including finite element analysis (27).

The use of the doubly curved hypar element has therefore been very popular because of its requirement for simple straight line construction formwork and its straightforward stress analysis. Furthermore, the hypar has an unlimited number of useful configurations (figure 2.2.3).

Because of these features, the study of reinforced concrete hypar shells is justified.

$$z = k_1 y^2 - k_2 x^2 = k_3 x^* y^*$$

FIGURE 2.2.1

Fundamental Hypar Geometrical Properties

$$v = \frac{w a b}{2 c t}$$

$$P_1 = \frac{w a^2 b}{2c} \quad (P_2, P_3, P_4 \text{ similar})$$

FIGURE 2.2.2
Fundamental Hyper Element Stress Condition

FIGURE 2.2.3
Fundamental Hypar Element Configurations

2.3 State of the Art of Construction Systems

A review of the state of the art of reinforced concrete shell construction will illustrate the traditional techniques as well as the more recent attempts to reduce requirements for temporary formwork. Here is a brief summary of the three main components of shell construction:

- (1) One of two basic shell materials is used; either concrete or shotcrete. Shotcrete, also known as gunite, is identical to concrete except for its lack of aggregate larger than 3/8" and its method of application. A number of references describing shotcrete are available (28, 29), and a major portion of the Gunite Contractors Association brochure G-76 is reproduced in Appendix A. Shotcrete has been successfully used for decades in shell construction and possesses two main advantages over concrete. First, shotcrete can be easily applied to steep, vertical and overhead surfaces where standard double forming for concrete would make costs prohibitive. Second, shotcreting is a capital intensive rather than labour intensive system (plates 2.3.1 to 2.3.5). One potentially hazardous characteristic of shotcrete, rebound, is easily controlled by good workmanship. "Rebound" is a term for sand particles which separate from the mix on surface contact and tend to collect in low-strength pockets. An experienced nozzleman will always attempt to apply new shotcrete in a direction towards shotcrete already in place, so that rebound cannot accumulate in areas yet to be covered.
- (2) One or more of three reinforcement concepts is used; rods seated and tied, ferro reinforcement, or fibre additives. The rods can be steel or glass (30), the ferro reinforcement can be chicken wire (31), woven wire fabric (32), welded wire fabric (33) or expanded metal (34), and the fibre can be acrylic, asbestos, cotton, glass, nylon, poly-

ster, polyethylene, polypropylene, rayon, rock wool or steel (35).

- (3) One of three forming concepts is used; traditional temporary forms (plates 2.3.6 to 2.3.9), permanent forms not serving as shell reinforcement (plates 2.3.10 to 2.3.13), or permanent forms serving as reinforcement (plate 2.3.14). One method of temporary forming not illustrated is to pour a shell on grade and excavate the soil from underneath when the shell has cured.

The forming concepts shown in plates 2.3.7 to 2.3.14 represent progressive steps toward elimination of the labour intensive temporary formwork evident in plate 2.3.6. The "Lift-Shape" process (plate 2.3.14) is the most advanced concept because all of the materials used perform a dual function of forming and reinforcement. However, there remain two large disadvantages with the concept. Expanded metal is difficult to manipulate, especially on non-developable surfaces, and the size and variety of shell surfaces available from lifting an assembly is limited.

The ideal shell construction system would therefore be similar to the "Lift-Shape" process, but would use a material other than the expanded metal in order to be applicable to all or most shell surfaces. Further investigation is justified.

PLATE 2.3.1

Labour Intensive Poured in Place RC Shell;
Church, Kentucky, c.1960 (source - ref. 50,
p. 167)

PLATE 2.3.2

Capital Intensive Shotcrete Shell in Progress;
Church, Ottawa, 1952 (source - courtesy Adje-
leian and Associates, Ottawa)

PLATE 2.3.3

Shotcrete Shell Complete (source -
courtesy Adjeleian and Associates,
Ottawa)

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PLATE 2.3:4

Capital Intensive Shotcrete Shell in Progress;
Industrial Roof, England, 1975 (source - ref.
51, p. 12)

The "shoe" that sits by the M6

PLATE 2.3.5

Shotcrete Shell Complete
(source - ref. 51, p. 12)

PLATE 2.3.6

Traditional Labour Intensive Temporary Wooden
Formwork; Bottle Factory, Mexico, c.1955
(source - ref. 47, p. 27)

PLATE 2.3.7

Traditional Labour Intensive Temporary Steel
Scaffolding c/w Wooden Form; Water Tank,
Morocco, c.1960 (source - ref. 52, p. 338)

PLATE 2.3.8

Temporary Wooden Formwork, Ground Mold; Pavilion,
Germany, 1977 (source - ref. 53, p. N13)

PLATE 2.3.9

Temporary Pneumatic Formwork; Bubble House,
Florida, 1954 (ref. 46, p. 139)

PLATE 2.3.10

Permanent Formwork not as Reinforcement, Wires,
Burlap and Thin Cured Shotcrete Application;
Church, Ottawa, 1952 (source - courtesy Adjeleian
and Associates, Ottawa)

PLATE 2.3.11

Permanent Formwork not as Reinforcement, Wires,
Styrofoam and Mortar Crust; Pavilion, Indiana,
c. 1960 (source - ref. 54, p. 457)

PLATE 2.3.12

Permanent Formwork not as Reinforcement, Cable Net,
Plywood Backing; Industrial Roof, England, 1975
(source - ref. 51, p. 14)

PLATE 2.3.13

Permanent Formwork not as Reinforcement, Cable
Net for Precast Elements; Industrial Roof,
Baltimore, c. 1970 (source - ref. 55, p. 322)

PLATE 2.3.14

Permanent Formwork as Reinforcement, "Lift-Shape Process" c/w Flexible Bars, Expanded Mesh and Shotcrete; Test Structure, Texas, c. 1960 (source - ref. 56, p. 449)

2.4 Proposed Construction System

A proposal for a reinforced concrete shell construction system must conform to the following criteria:

- (1) Minimize the system's requirements for labour intensive temporary formwork.
- (2) Maximize the system's applicability to various shell surfaces.
- (3) Minimize the system's number of different materials.
- (4) Minimize the system's resultant thickness.
- (5) Maximize the system's cross-sectional symmetry.
- (6) Maximize use of materials with proven construction performance.
- (7) Maximize use of materials with handling ease and placement flexibility.
- (8) Maximize use of commercially available materials.
- (9) Minimize cost.

The shell material, the reinforcement materials, and the forming concept must now be individually specified.

Shotcrete can be specified immediately for the shell material of the proposed system. It conforms to all of the criteria, whereas poured in place concrete does not.

The proposed forming concept, of the "permanent formwork serving as shell reinforcement" category, is a logical consequence of the selection of shotcrete, and an extension of the "Lift-Shape" process. Figure 2.4.1 shows the proposal applied to a hyper shell surface. It is similar to the "Lift-Shape" process except that the main reinforcement grid is assembled in its final position rather than on the ground. Elements of the main reinforcement grid are attached to the edge beams and correspond to the straight line generators of the hyper. A fabric, whose purpose is to first trap the top application of shotcrete and then serve as permanent additional reinforcement,

is placed between the two layers of the main reinforcement grid. Obviously, the fabric must have an opening small enough to trap the first shotcrete application without too much waste, but large enough to allow handling flexibility and good bond between the top and bottom shotcrete applications. An opening of between one eighth and three eighths of an inch is estimated to be satisfactory. The reinforcement materials could have other configurations (figure 2.4.2), but any alternative violates at least one of the criteria.

The last step is to specify the reinforcement materials themselves. Consult Appendix B for details of the following discussion. Glass rod should not be considered for the main reinforcement grid because of lack of proven performance in shells, unknown expense, and connection problems to the edge beams. The main reinforcement grid can therefore be specified as steel, which might be re-bar, wire, 7-wire strand or 133-wire rope depending on the project. The specification of the fabric proves to be more difficult. A full range of commercially available fabrics could be considered, including non-woven and woven polyethylene fabric, non-woven polypropylene fabric, non-woven chemotextiles, expanded steel mesh, chicken-wire, woven steel wire fabric, and woven fiberglass fabric. A comprehensive computer search of the Engineering Index and Dissertation Abstracts, done by the Carleton University Library in June 1978, and discussions with manufacturers concerning their fabric's concrete reinforcement capabilities, narrowed the possibilities to steel and fiberglass. Further, the expanded steel mesh and chicken wire can be rejected because of poor handling and construction characteristics. Either of the remaining two might therefore be specified, though the fiberglass fabric has both handling and cost advantages over the wire fabric.

With the completion of the specification of the shell material, reinforcement concept and possible reinforcement materials, a testing program for the proposed shell construction system can now be considered. Because a particular application to hypar shells is being studied, and because hypars

transfer loads primarily through in-plane shear, the test program will concentrate on the determination of the in-plane shear strength of the proposed shell construction system. Details of the test program formulation, apparatus, procedure and results are discussed in Chapter 3.

FIGURE 2.4.1

Proposed Shell Construction System

- A. Minor fabric stretched over major fabric over cable grid - rejected because of too many materials

- B. Minor fabric stretched over cable grid - rejected because it's not symmetrical and too much shotcrete must be applied from bottom

- C. Minor fabric stretched between layers of cable grid - accepted

- D. Minor fabric stretched below cable grid - rejected because it's not symmetrical and construction costs would be high

- E. Minor fabric hung below cable grid - rejected because construction costs would be high, and fabric could not be considered as reinforcement

FIGURE 2.4.2

Alternative Reinforcement Materials Configurations
for Proposed Shell Construction System

CHAPTER 3

PLATE (SHELL ELEMENT) SHEAR STRENGTH TEST PROGRAM

3.1 Program Formulation

The main objective of the test program, as stated in the Introduction, is to "determine the structural properties of the proposed composite reinforced shotcrete shell construction system". The determination of the most essential properties can be accomplished by answering these specific questions:

- (1) Does the fabric of the composite reinforcement system possess adequate construction handling characteristics?
- (2) Does the fabric contribute to the shear strength of the composite reinforcement system? If so, does it correspond to predictions?
- (3) Does the shear strength of the total reinforcement system correspond to predictions?
- (4) Are there any apparent weaknesses of the proposed reinforcement system?

In order to answer these questions a series of test samples was fabricated using actual construction methods and materials, including the shotcreting process, and the samples were tested by simulating actual loading conditions.

It had been first decided that small elements of a shallow hypar should be tested instead of scale models of a complete hypar. Note that a 32 inch by 32 inch square element of a 50 feet by 50 feet by 10 feet hypar has one corner out of plane by only 6.5 inches. It was therefore assumed that small hypar elements could be represented for testing purposes by flat plates. If test loads simulate only the in-plane shear stress of the actual hypar stress condition, then the assumption is justifiable.

The test program consisted of 16 plates with different reinforcement

combinations being loaded to failure in planar shear. The first four plates, Series A, consisting of plain shotcrete only, were meant to provide a benchmark. The next four, Series B, consisting of shotcrete and the main steel reinforcement grid, were meant to demonstrate the increase in strength of the plate due to the presence of the steel, and to provide another benchmark for the evaluation of the fabrics. The third batch of four, Series C, consisting of shotcrete, the reinforcement grid and the woven fiberglass fabric, was meant to demonstrate the structural properties of the woven fiberglass fabric. The fourth batch, Series D, was similar to the third, but woven wire fabric was substituted for the fiberglass. See figure 3.1.1.

Two supplementary plates of plain shotcrete were also required. One was to be used for a dry run of the entire test procedure, and another was to be cut into prisms for compressive strength tests.

Resource limitations restricted the choice of the steel reinforcement to one of the four possibilities (re-bar, wire, 7-wire prestressing strand and 133-wire rope). Only the prestressing strand and wire rope were judged to be capable of meeting all of the envisioned applications of the system, and wire rope was judged to have the most potential applications. Therefore, wire rope was specified for the main reinforcement grid.

With the completion of the formulation of the test program and material specification of the test samples, testing could begin. A discussion of the design of the test apparatus and test plates will be given before the test results are presented.

FIGURE 3.1.1

Perspectives of Test Plate Series A-D

3.2 Test Apparatus

3.2.1 Test Concepts

The test program formulation specified that small flat plates were to be tested for planar shear strength. A number of testing concepts were considered (figure 3.2.1). There was little available documented evidence of similar testing programs on which to base a decision. An exhaustive search through Carleton University's library facilities disclosed only two articles which illustrated testing methods for planar shear strength of plates. Tanaka (36) in 1962 tested a series of plates in a manner similar to Concept 3 of Figure 3.2.1, and achieved the centrally located vertical cracking pattern such as one might expect from a split cylinder test. Leonhardt and Walther (37) in 1966 successfully tested plates in the course of their deep beam studies using Concepts 1, 2 and 5. Because the emphasis of this thesis concerns the comparison of different materials' performance rather than the optimization of testing processes, a decision was made to forego further research and select the concept which appeared the most adequate. Concepts 1 and 2 were rejected because of the presence of flexural stress, which is not present in the actual stressed state of a hypar shell. Concept 3 was rejected because Concept 5 provides a more uniform, hence more realistic, stress application, and Concept 4 was rejected because Concept 5 is easier to load. Concept 5 was therefore selected for the testing procedures, corresponding to the failure model of figure 3.2.2 (38). The one remaining decision concerned the attachment of the test apparatus to the test plate.

3.2.2 Apparatus Edge Detail

Two edge details were considered (figure 3.2.3). Detail 2 was selected for 3 reasons:

- (1) If the shear pins of Detail 1 were cast-in-place, there would be

a danger of shotcrete rebound accumulating around the pins.

Cured rebound has little to no strength and the pins' ability to transfer shear would be greatly reduced. Indeed, when the forms were later removed from the shotcrete plates before testing, the corners and edges of the plates crumbled away.

- (2) If the shear pins of Detail 1 were drilled in place after curing, there would be a danger of cleaving the plates along their central planes of potential weakness during the drilling process. The behaviour of the wire fabric composite reinforced plates during later testing showed that the cleavage probably would have occurred.

- (3) Even if the shear pins were successfully implanted, there remained the probability of cleaving the plates during the testing process.

By selecting Detail 2 there was no danger of cleaving the plate.

However, the arm loads could not be permitted to transfer their loads through bearing of the bolts against their shotcrete holes, since ultimate load would then be determined by crushing failure of the shotcrete. Therefore, the arm bolts of the apparatus must be tightened enough to transfer the loads through surface friction.

Design drawings and costs for the final test apparatus can be found in Appendix C. The completely assembled apparatus is shown in plate 3.2.1. Recalling that the apparatus was fabricated with time and money of great concern, note that a double pin was designed for each corner instead of a single pin, for two reasons:

- (1) The width dimension of the apparatus arms could be substantially reduced.
- (2) A single hinge was judged to be too costly to have machined. The double hinge is very simple and required no tooling other than flame-cut holes. There was also some question as to the immediate availability of the larger round high-strength stock required for a

single hinge.

The double pin divides the applied load into halves as well as a single pin, and for small deflections the deflected shape of the plate in the double pin apparatus would be virtually the same as in the single pin apparatus. For these reasons the double pin apparatus was assumed to be a fair representation of the chosen Concept 5 (figure 3.2.1). In general, the size of the apparatus was limited by the size of the plates in the test series, which were in turn limited by the handling capacities of two lab assistants.

Concept: 1

Concept: 2

Concept: 3

Concept: 4

Concept: 5
(selected)

FIGURE 3.2.1
Plate Planar Shear Strength Testing Concepts

- (1) for testing unreinforced plates, $\sqrt{2} \frac{P_u(E)}{2\ell t} = 0.11 f_c'$
- (2) for testing reinforced plates, $\sqrt{2} \frac{P_u(E)}{2} = \sum_{i=1}^n P_n$
- (3) for shell design, $s = \frac{P_n}{vt}$

FIGURE 3:2.2
Reinforced Concrete Plate Failure Model

FIGURE 3.2.3

Alternative Edge Details for Shear Test Apparatus

PLATE 3.2.1

Assembled Shear Test Apparatus

3.3 Plate Test Program

3.3.1 Plate Forming and Fabricating

Eighteen test plates, each 32 inch x 32 inch x 2 inch, were formed and fabricated according to the descriptions given in paragraph 3.1 (plate 3.3.1). The main reinforcement grid for Series B, C and D was quarter inch wire rope on six inch centers in two directions. The fibreglass fabric for Series C was FRC 1027, the wire fabric for Series D was Tyler 7-054, and the shotcrete was applied by Benson Pools of Ottawa.

The wire rope proved to be very flexible, as was the fibreglass fabric. Both would easily conform to non-developable surfaces. In contrast, the wire fabric was very stiff and difficult to handle, and would be completely impractical for any real shell application. A finer, more widely spaced wire fabric would be an improvement, but it would still not be able to conform well with non-developable surfaces.

The shotcreting operation was continuous for two hours at -1°C (plate 3.3.2). Both fabrics responded well to the first application of shotcrete. There were no local failure and an eighth inch finite thickness of shotcrete passed through before the fabrics were completely plugged. Each plate was stored immediately after fabrication in a heated room, and all plates were cured in a moist environment at 15°C for 10 days. The forms were stripped, the plate edges were ground to remove edge irregularities, and the plates were stacked with only their edges exposed at 20°C until their time of testing. Complete drawings and costs for the forming operations can be found in Appendix D.

3.3.2 Shotcrete Quality Control

For quality control, the GCA brochure in Appendix A outlines methods for casting cylinders and extracting cores from gunite in place. Neither method was judged appropriate for this project. Cylinders were considered

too large and rebound-free to be representative of a shell element, and cylindrical cores would have to be tested in an out-of-plane orientation rather than in the actual in-plane compressive stress situation. Therefore, one plain shotcrete plate was cut into 2 inch x 2 inch x 4 inch prisms by diamond-tipped circular saw for compressive testing along their 4 inch in-plane axes. The prisms were stored in a wet cabinet and kept moist until their time of testing. They were then capped with hydrostone and tested to failure in batches of ten to twelve every seven days from Day 14 to Day 56 (plate 3.3.3). No correction factors were applied to the resulting test values (39). The resulting time-strength curve is shown in figure 3.3.1, and full details can be found in Appendix E. Figure 3.3.2 shows a bar chart of a typical batch results. The observed 28 days prism strength of 2.5 ksi. was below the GCA prescribed minimum of 3 ksi., but the plate testing program did not proceed until constant 3 ksi. strength was achieved after Day 56.

3.3.3 Plate Testing

In turn, each plate was mounted in the test apparatus, placed in a Tinius Olsen test machine, and loaded incrementally in the compression mode to failure. Total vertical deflection of the machine platform was monitored throughout the loading process. Plates 3.3.4 to 3.3.7 show typical failed conditions of Series A to D respectively. Failure for Series A was defined as the load at which the first complete vertical crack appeared. Failure was expected when the applied shear stress reached the split cylinder tensile strength of the shotcrete, given by $0.11 f_c'$ (40). Failure for Series B, C and D was defined as the maximum load carried by the plate, regardless of the number of cracks. Failure was expected when the reinforcement yielded in tension (figure 3.2.2). The resulting average load-deflection curves are shown in figure 3.3.3, and a bar chart of actual and expected loads

is shown in figure 3.3.4. Full details can be found in Appendix F.

The following critical observations were made:

- (1) As the plates were being mounted into the test apparatus, one or two fine random straight cracks frequently appeared on the plate surfaces when the apparatus arm bolts were tightened.
- (2) Failures of the plain shotcrete plates were characterized by a single, large brittle crack.
- (3) Failures of all three series of reinforced plates were similar to each other, and were characterized by a fairly uniform set of narrow vertical cracks. All of the cracks originated at the bolt holes drilled through the plates.
- (4) At ultimate and advanced loads, in all cases the main reinforcement grid did not yield; instead the ends receded an eighth to a quarter of an inch into the plate edges, indicating failure by pullout.
- (5) At ultimate load the fibreglass fabric reinforcement failed in tension. At advanced deflection some surface spalling occurred. A close-up photograph of a fibreglass reinforced plate surface at a deflection far past ultimate load clearly shows both of these features (plate 3.3.8).
- (6) At ultimate load, the steel wire fabric reinforcement did not yield. Instead, the fabric caused the plate to cleave along its central plane. At a deflection far past ultimate load, large pieces of the plate surface either fell away from, or could be pulled away from, both sides of the fabric, leaving the fabric exposed and intact (plate 3.3.9).
- (7) For every plate in each series, the observed ultimate load was substantially below the expected value.

PLATE 3.3.1

Sample Plate Formwork, Woven Wire Fabric and
Wire Rope Composite Reinforcement

PLATE 3.3.2

Shotcreting in Progress, Fibreglass Fabric and
Wire Rope Composite Reinforcement Shown

PLATE 3:3.3

Sample Shotcrete Prism Compressive Strength
Test - Typical Failed Condition

FIGURE 3.3.1

Prism Time-Compressive Strength Curve

FIGURE 3.3.2

28-Day Prism Test Bar Chart.

PLATE 3.3.4

Sample Plain Shotcrete Plate Shear Strength
Test - Typical Failed Condition

- PLATE 3.3.5

Sample Steel Grid Reinforced Shotcrete Plate
Shear Test / Typical Failed Condition

PLATE 3.3.6

Sample Fibreglass Fabric Composite Reinforced
Shotcrete Plate Shear Strength Test - Typical
Failed Condition

PLATE 3.3.7

Sample Wire Fabric Composite Reinforced Shot-
crete Plate Shear Strength Test - Typical
Failed Condition

FIGURE 3.3.3
Plate Average Load-Deflection Curves

FIGURE 3.3.4

Plate Test Bar Chart

PLATE 3.3.8

Close-up Showing Local Spalling and Ruptured
Fibreglass Fabric at Advanced Deflection of a
Fibreglass Fabric Composite Reinforced Shot-
crete Plate

PLATE 3.3.9

Close-up Showing Central Plane Cleavage After
Failure of a Woven Wire Fabric Composite Re-
inforced Shotcrete Plate

3.4 Discussion of Results

3.4.1 Shotcrete Quality Control

The 28-day prism strength was 2.5 ksi. with a standard deviation of 0.7 ksi. This strength was below the acceptable 3 ksi specified in the GCA brochure, and far below the usual strength of 4.5 to 5 ksi. claimed by the shotcrete contractor. The low strength was due to a higher than usual water/cement ratio, which had been requested in order to minimize air pockets close to the reinforcement and along the formwork edges. The intent had been to lower the compressive strength to 3.5 or 4 ksi. Despite the effort, between 5% and 10% of the prisms were still rejected for testing because of obvious defects from air bubbles.

CSA specifications recommend that for a 28 day standard deviation of greater than 600 psi for 30 test specimens the specified concrete strength should be at least 1200 psi greater than the design strength (41). Although there were only ten or twelve test specimens per batch, it can nevertheless be concluded that quality control of shotcrete can be poor even with an experienced nozzleman. Therefore, for any shell project requiring shotcrete, the specified compressive strength should be at least 1200 psi. beyond the design strength. Further, to assist the nozzleman in quality control, there should be a clearly marked calibration device for the flow of water at the nozzle.

In general, the process of casting a plate and cutting and testing the prisms was simple and indicative of the true condition of the shotcrete in place. Therefore the process has some merit, and any shell project requiring shotcrete could use some variation of this method in conjunction with either cylinder or core tests.

3.4.2 Plate Shear Strength

In general, the strength trends in the results were very close to expectations, and the results within each different plate series were consistent. However, the major difficulty is to determine why the average observed results were so much lower than the expected results.

Recall that the plain shotcrete plates were expected to fail when the applied shear stress reached approximately the split cylinder tensile strength of the shotcrete. The unexpected appearance of surface cracks when the arm bolts were tightened provide the apparent answer. If the plates were formed and cured slightly out of plane, then a flexural stress would be applied to the plates as the arms were tightened. Therefore, the initial prestress coupled with initial cracks would cause premature plate failure. It illustrates one shortcoming of the apparatus design, and unfortunately, no conclusion can be reached concerning the validity of the failure assumption.

Note that the ultimate load of the simply reinforced plates should not be affected by the initial flexural prestressing and cracking, since the failure model assumes that the ultimate load is governed only by the tensile strength of the main reinforcement. The low average ultimate load was therefore caused solely by the pullout of the main reinforcement before yield in tension. This illustrates another shortcoming of the apparatus, and therefore no conclusion can be reached about the failure model or the reinforcement capabilities of wire rope. However, a consistent benchmark was provided by this series of plates. The performance of the fibreglass and steel wire fabrics could now be adequately assessed, because the fabrics should not be sensitive to pullout versus yield in tension by the main reinforcement.

For the fibreglass fabric composite reinforced plates, the observed average increase in strength over the simply reinforced plates of 6.2 K was close to the predicted increase of 7.4 K. The full increase did not occur

probably because the glass fibers did not all yield simultaneously. Nevertheless, the failure model appears essentially correct, and the strength of the fabric could be incorporated into the design resistance of a shell. The performance of the fiberglass is important in another respect - at advanced deflection there was local spalling, but no dangerous massive cleavage of the central plane.

The steel wire fabric composite reinforced plates did not perform nearly as well as those of fibreglass. At ultimate load the fabric did not yield, but instead caused a total brittle cleavage of the central plane. Evidence of the brittle failure can be seen on the load-deflection curve in figure 3.3.3. The steel wire fabric plates sustained only a slightly higher vertical deflection than the simply reinforced plates, and not as much as that of the fibreglass. This was undoubtedly caused by a poor combination of the large screen wire thickness and close wire spacing, which prevented a good bond between the top and bottom applications of shotcrete. There is room to suggest that a wire fabric with thinner wires and larger openings might perform as well as the fibreglass fabric, especially if the shotcrete was up to normal strength. However, none of Tyler's fabrics that might be applicable can compete on a cost basis (Appendix B), and any steel product including welded wire fabric or expanded mesh would always have more troublesome handling characteristics than fibreglass. Also, because of a steel product's thickness compared to the fibreglass, there would be a greater planar cleavage hazard. Therefore, on the basis of cost, handling, and structural performance advantages, the fibreglass fabric is superior to steel wire fabric. Further, the building construction phase of this thesis should use the fibreglass fabric in the proposed shell construction system, and the additional strength provided by the fabric should be incorporated into the shell design.

Testing of the proposed fibreglass composite reinforced shotcrete

system should by no means be considered complete. Flexural testing should be done to determine local bending strength, which might also disclose more information concerning the central cleavage plane. Shock testing might reveal a tendency for bottom delamination. Tests should be done with 7-wire prestressing strand, a material less expensive than wire rope and adequately flexible, and effects of varying the weave in the fibreglass fabric might prove useful. The best experiment would be to construct and load test a series of full scale hypars using actual methods and materials.

3.4.3 Testing Apparatus

The apparatus was successful in a number of respects. The evenly distributed vertical crack patterns at ultimate load, and random location of initial vertical cracking, indicate an evenly distributed shear load on the plates. The lack of local crushing around the arm bolts, apparent after apparatus disassembly, showed that the loads were transferred to the plate by surface friction rather than by bolt bearing. Finally, all components of the apparatus were non-destructible and re-usable.

The apparatus was not successful in a number of respects. Flexural prestressing of the plates during mounting into the apparatus ruined the study of the plain shotcrete plate series. Pullout of the main reinforcement grid, which could have been prevented by attaching the ends of the reinforcement by some means to the arms of the apparatus, prevented an assessment of the reinforcement capabilities of wire rope, and prevented verification of the failure model. Considerable expense and effort went into the plate forms, which were eventually wasted. Hundreds of manhours were required for plate preparation, and apparatus assembly and disassembly. The double pin at each corner is at best a close approximation of a true single hinge.

Therefore, despite its small number of advantages, the apparatus used in this test program is not adequate for further testing of plates in

shear. If for any reason further plate shear testing is to be undertaken, a different apparatus should be used, one in which the test plate is cast monolithically with the arms, and the ends of the reinforcement grid are firmly anchored. Temporary plate formwork, assembly labour, plate prestressing and pullout would all be considerably reduced if not eliminated. A single large pin should be used at each corner, despite the extra machining costs. A proposed edge detail is found in figure 3.4.1. There are two drawbacks with the revised apparatus. If only one apparatus is fabricated, then only one plate can be tested per month. Alternatively, if many tests are required simultaneously, then the capital investment for many apparatus becomes very high. It should be emphasized that if the experiments are judged necessary, then the investment is justified.

3.4.4 Summary

The questions posed in paragraph 3.1, concerning the structural performance of the proposed fabric/steel grid composite reinforced shell construction system, have been answered by the plate planar shear test program.

In the course of the fabrication of the test plates, it was determined that woven fibreglass fabric has adequate shell construction handling characteristics, whereas woven steel wire fabric does not. Both fabrics can resist the abuse from a direct shotcrete application. The amount of shotcrete allowed through is small, and probably contributes a beneficial effect by allowing the passage of rebound.

The testing program proved that the fibreglass fabric predictably increased the planar shear strength of the composite reinforced plates compared to the steel grid simply reinforced plates. The steel wire fabric failed to do so. Unfortunately, the observed strength of the fibreglass fabric composite reinforced plates did not correspond to the predicted

strength because of a design error in the testing apparatus which prevented the steel grid from reaching full strength. Nevertheless, the proposed fabric/steel grid composite reinforced shell construction system has demonstrated real potential, and further investigation by a building construction cost program in this thesis has been fully justified.

The only apparent weakness of the system was exposed by the failing performance of the steel wire fabric. The fabric's wire thickness and relatively close wire spacing prevented good bond between the top and bottom applications of shotcrete, and caused brittle cleavage along the plates' central planes long before the expected ultimate load. This problem should not exist provided an appropriate combination of fabric filament size and spacing is used, and adequate quality control of the shotcrete application is maintained. Further plate testing would be expensive but of value. Load tests on a series of full scale hypar elements would disclose local bending and buckling strength, ultimate shear strength, and possible delamination characteristics.

The failure of the Series B, C and D reinforced plates could be described as neither brittle nor plastic. Plastic failure was expected as the main reinforcement grid yielded, but pullout prevented that occurrence. Plastic failure in structures is preferred. Therefore tests should be re-done on a series of reinforced plates with the reinforcement firmly anchored, in order to clearly determine if the composite reinforcement system will provide a plastic failure.

The results of the test program will now be applied to a building construction cost program in Chapter 4.

FIGURE 3.4.1

Modified Edge Detail for Shear Test Apparatus

CHAPTER 4

BUILDING CONSTRUCTION COST PROGRAM

4.1 Program Formulation

The main objective of the cost program, as stated in the introduction, is to "determine the cost reducing potential of the proposed reinforced concrete shell construction system". This can be accomplished by first designing and costing a series of buildings with similar dimensions but different structural systems, and then answering these specific questions:

- (1) Does the proposed system reduce construction costs compared to a traditional shell construction system?
- (2) Is the proposed system competitive in cost with today's commonly used long-span structural systems?
- (3) Can the proposed system be improved and/or its costs reduced?

A group F3 light industrial building (42), such as a warehouse or prefabricated home assembly plant, was arbitrarily judged to be an appropriate building type for a comparative design and cost study. Dimensions were specified as 100 feet x 200 feet column-free in plan, with a minimum 18 feet clearance throughout. It is a basic type of building, one for which there are many well-documented structural systems, including shells. Design and costing of such a series of buildings using various structural systems was therefore expected to be a straightforward task giving relevant results.

Only the detailed costs of those building components heavily dependent on a change of structural system were considered necessary for an informative comparative cost analysis. Sitework and foundations were estimated, since the costs of those components are determined essentially by a 12 inch thick grade beam, a strip footing, and a slab-on-grade. Main structural components, primary cladding, and the primary roof were designed and costed in detail. Roofing, insulation, architectural finishes and

mechanical and electrical services were estimated.

In all cases, budget prices instead of tendered prices were used. It was strongly felt that unless a significant cost saving was apparent at the budget level, the proposed construction system would probably be not worth pursuing for actual construction application.

Before any structural planning or design was allowed to proceed, the original system proposal was critically assessed in the light of the plate shear test program results and other input from structural consultants. Without question the fibreglass fabric was to be used in the proposed system. There were, however, problems with the main reinforcement grid. The elements of the grid correspond in plan to the straight line generators of the hyper surface, but sag due to self-weight and the weight of the first application of shotcrete would be unavoidable. The stress equations for hypers would be useless and further research into advanced shell analysis would be required. Furthermore, the tension arising from the cable action of the grid elements would apply considerable bending moment to the edge beams. There was some thought of rotating the grid 45° to correspond to the lines of principal stress, which would solve the sag problem, but the edge beam bending problem would remain, coupled with higher uncertain labour costs for cutting and tensioning individual lengths of cable. It was therefore decided to retain the original grid orientation, support the grid at 5 feet intervals, and assume that the individual elements of the reinforcement grid would act as continuous beams rather than cables when subjected to the first application of shotcrete. Ordinary steel reinforcement bar could then be specified for the grid, because of its stiffness, rather than prestressing strand or wire rope. In summary, the problems of sag, edge beam bending moment and high-cost cable in the original construction system proposal were eliminated at the expense of temporary scaffolding and timber "stringers"

(figure 4.1.1). The modified concept was certainly not as elegant as the original concept, but budget costs for the newly specified methods and materials were expected to be accurate.

With the formulation of the cost program and material specifications of the proposed reinforced concrete shell construction system proposal complete, design and costing could begin. A description of the various structural systems selected for the same industrial building plan will now be given.

shotcrete (top layer applied
first, bottom layer applied
after top has cured sufficiently)

FIGURE 4.1.1

Revised Shell Construction System
for Building Cost Program

4.2 Selection of Buildings

4.2.1 Structural Concepts

Three distinct structural systems were selected. Two systems, Series B and C, employed a standard hypar shell roof system (figure 4.2.1).

Series A was of simple steel construction, using 10 feet deep flat roof trusses spanning 100 feet, and open web steel joists spanning the 8 interior 25 foot bays. The lateral K-brace was selected to allow virtually unrestricted structural thermal movement, and to permit a less severe seismic design load compared to simple diagonal tension bracing (43). Series A was chosen as a benchmark to represent an inexpensive and commonly used structural system for a light industrial building. Other inexpensive systems, such as arched girders might have been considered, but one representative system was judged to be enough.

Series B was of composite steel and concrete construction, using 8 concrete hypar elements 50 feet x 50 feet in plan x 10 feet in rise, connected to steel edge beams, thrust ties, K-brace and columns. The shells were two and one half inches thick throughout, thickened to three inches near the edge beams to accommodate an extra layer of local flexural reinforcement. This series was chosen to represent the least expensive system for the proposed shell construction method, by taking advantage of the expected low costs of single storey steel construction. The ten feet rise of the shells was specified to match the height of the roof trusses of Series A in order to maintain dimensional uniformity as much as possible.

Series C was of entirely poured-in-place concrete construction using an ungrouted prestressing operation to support the shell thrust. Its dimensions were specified to be the same as those of Series B. The series was selected to demonstrate one of the most expensive ways of executing an industrial building with a shell roof.

Both of Series B and C were further specified to have their shells constructed in 3 different ways, for a total of 7 different buildings to be designed and costed, including Series A.

The 3 different shell construction methods were specified as follows:

- (1) Construct the shells in a traditional manner, with temporary steel scaffolding, temporary stringers and plywood deck, permanent steel re-bar reinforcement, and poured-in-place concrete.
- (2) Construct the shells in the same manner as Method 1, but instead of poured-in-place concrete use shotcrete applied from an articulated truck-mounted crane.
- (3) Construct the shells using the proposed system, with temporary steel scaffolding, temporary stringers, permanent fibreglass fabric/steel re-bar composite reinforcement, and shotcrete applied from a truck-mounted crane. Use the FRC 1037 fibreglass fabric because of its higher strength than FRC 1027.

It is very important to note that the specified dimensions of the buildings did not guarantee the minimum cost for the individual structural systems. For example, optimization studies by Gensert et al (44) indicate the optimum rise for the shell roof would be about 25 feet, not the specified 10 feet. (A higher rise means lower stress, reduced reinforcement and smaller edge beams). A similar argument can be made for the height of the main roof trusses in the steel building, except that the optimum building would use smaller trusses instead of larger. It must be emphasized that the budget cost figures derived from this cost program were intended to be used in a management decision process. If there was any evidence at all that the proposed system offered substantial cost reductions compared to a traditional manner, then a building optimization would definitely be in order as a separately defined study.

Series A - Simple Steel Construction

Series B - Composite Steel and Concrete Construction

Series C - Poured in Place Concrete Construction

FIGURE 4.2.1

Perspectives of Series A-C Alternative
Structural Systems

4.2.2 Structural Components

Figures 4.2.2 to 4.2.4 illustrate plans, elevations and sections of the three building series, including detailed specifications of the structural components to be considered in the comparative cost analysis. Final component configurations were determined with the assistance of a number of consulting engineers from the firm of Adjeleian and Associates Inc., including J. Adjeleian, M. Allen, C. Bellinger and D. Allan.

Design details, takeoffs and costs are presented in the next section. The designs are preliminary and are for the purpose of budget costing only. Imperial units are used throughout, since the construction industry has only partially converted to metric units.

Although an ungrouted prestressing system has been specified in the drawings for the Series C concrete building, there are two arguments for the compulsory use of a grouted system. At the yield point of the ungrouted system, the exterior cracking pattern on the concrete wall would be a single large opening, rendering the building unserviceable. In contrast, a grouted system would display a uniform series of fine cracks and the building would remain serviceable. Also, failure of the ungrouted system would result in the catastrophic collapse of the entire building, whereas failure in the grouted system would provide ample warning for evacuation.

LEGEND

FIGURE 4.2.2

Series A Steel Building - Plans and Sections

FIGURE 4.2.3

Series B Composite Building - Plans and Sections

4.3 Construction Cost Program

The Series A, B and C buildings in figures 4.2.2 to 4.2.4 were designed according to the specifications of the National Building Code of Canada 1977, with the assistance of the firm of Adjeleian and Associates consulting engineers and a number of local contractors. The buildings can withstand climatic loads of at least those of Ottawa, Canada, which are 60 psf ground snow load, 7.8 psf 30 year wind velocity, and seismic zone 2. Appendix H shows sample design calculations for the hyper shells and edge beams for the Series C concrete buildings.

Tables 4.3.1 through 4.3.7 summarize the seven buildings' materials takeoffs and costs. Sources of the unit costs used are listed in Appendix G. Figure 4.3.1 is a bar chart comparing the structural cost of each building. An explanation of the "architectural, mechanical, electrical" cost entry into each of the tables is required. The total cost of the Series A steel building was arbitrarily specified to be \$30 per square foot or \$600,000 total. From this was subtracted the sub-total of the estimate for the foundation and the sub-total of the detailed structural cost analysis. The remainder of \$416,000 was then assumed to be constant for the buildings of Series B and

ITEM	DESCRIPTION	QTY	UNIT COST	ITEM COST	% OF TOTAL
1	foundation c/w slab on grade	20,000 ft ²	\$ 2	\$40,000	7%
2	structure:				
	(a) 7x10'x100' roof trusses	77K	\$500	\$39,000	
	(b) 72x24"x25' roof joists	25K	\$500	\$12,000	
	(c) 48 beams	21K	\$450	\$ 9,000	
	(d) 24 columns	26K	\$450	\$12,000	
	(e) 16 K-brace members	15K	\$450	\$ 8,000	
	(f) 10% misc. steel	16K	\$500	\$ 8,000	
	(g) steel diaphragm roof deck	20,000 ft ²	\$ 1	\$20,000	
	(h) reinf. masonry wall c/w 8x16 blocks	8,000 bl	\$ 3	\$24,000	
	(i) steel vertical cladding	6,000 ft ²	\$ 2	\$12,000	
	sub-total			\$144,000	24%
3	architectural, mechanical, electrical			\$416,000	69%
	total	20,000 ft ²	\$ 30	\$600,000	100%

TABLE 4.3.1

Series A Steel Building
Takeoffs and Costs

ITEM	DESCRIPTION	QTY	UNIT COST	ITEM COST	% OF TOTAL
1	foundation c/w slab on grade	20,000 ft ²	\$ 2	\$40,000	5%
2	structure:				
	(a) shell edge beams c/w flexible strips	168 K	\$500	\$84,000	
	(b) thrust ties	78 K	\$500	\$40,000	
	(c) columns	19 K	\$450	\$ 9,000	
	(d) K-brace members	8 K	\$450	\$ 4,000	
	(e) 10% misc. steel	27 K	\$500	\$14,000	
	(f) reinf. masonry wall c/w 8 x 16 blocks	8,000 bl	\$ 3	\$24,000	
	(g) steel vertical cladding	3,000 ft ²	\$ 2	\$ 6,000	
	(h) shell formwork contact area	20,000 ft ²	\$ 4.50	\$90,000	
	(i) poured concrete	165 yds. ³	\$ 52	\$ 9,000	
	(j) manual finishing	20,000 ft ²	\$ 1	\$20,000	
	(k) #4 grade 60 re-bar @ 6"	32 T	\$700	\$22,000	
	sub-total			\$322,000	41%
3	architectural, mechanical, electrical			\$416,000	54%
	total	20,000 ft ²	\$ 39	\$778,000	100%

TABLE 4.3.2

Series B Composite Building
Shells Formed c/w Concrete
Takeoffs and Costs

22

OF/DE

ITEM	DESCRIPTION	QTY	UNIT COST	ITEM COST	% OF TOTAL
1	foundation c/w slab on grade	20,000 ft ²	\$ 2	\$40,000	5%
2	structure:				
	(a) shell edge beams c/w flexible strips	168 K	\$500	\$84,000	
	(b) thrust ties	79 K	\$500	\$40,000	
	(c) columns	19 K	\$450	\$ 9,000	
	(d) K-brace members	8 K	\$450	\$ 4,000	
	(e) 10% misc. steel	27 K	\$500	\$14,000	
	(f) reinf. masonry wall c/w 8x16 blocks	8,000 bl.	\$ 3	\$24,000	
	(g) steel vertical cladding	3,000 ft ²	\$ 2	\$ 6,000	
	(h) shell formwork contact area	20,000 ft ²	\$ 4.50	\$90,000	
	(i) shotcrete inc. finishing, 10% loss to rebound	182 yds. ³ (9 days)	\$117	\$21,000	
	(j) crane	72 hrs.	\$ 60	\$ 4,000	
	(k) #4 grade 60 re-bar @ 6"	32 T	\$700	\$22,000	
	sub-total			\$318,000	41%
3	architectural, mechanical, electrical			\$416,000	54%
	total.	20,000 ft ²	\$38 ¹ / ₂	\$774,000	100%

TABLE 4.3.3

Series B Composite Building
Shells Formed c/w Shotcrete
Takeoffs and Costs

ITEM	DESCRIPTION	QTY	UNIT COST	ITEM COST	% OF TOTAL
1	foundation c/w slab on grade	20,000 ft ²	\$ 2	\$40,000	5%
2	structure:				
	(a) shell edge beams c/w strips	168 K.	\$500	\$84,000	
	(b) thrust ties	79 K	\$500	\$40,000	
	(c) columns	19 K	\$450	\$ 9,000	
	(d) K-brace members	8 K	\$450	\$ 4,000	
	(e) 10% misc. steel	27 K	\$500	\$14,000	
	(f) reinf. masonry wall	8,000 b1	\$ 3	\$24,000	
	(g) steel vertical cladding	3,000 ft ²	\$ 2	\$ 6,000	
	(h) shell scaffolding & stringers	20,000 ft ²	\$ 2	\$40,000	
	(i) shotcrete (incl. finishing and 20% loss)	198 yds. ³ (10 days)	\$117	\$23,000	
	(j) crane	80 hrs	\$ 60	\$ 5,000	
	(k) #4 grade 60 re-bar @ 7"	28 T	\$700	\$19,000	
	(l) FRC 1037 fibreglass fabric (incl. overlap)	24,000 ft ²	\$ 0.5	\$12,000	
	sub-total			\$280,000	38%
3	architectural, mechanical, electrical			\$416,000	57%
	total	20,000 ft ²	\$ 37	\$736,000	100%

TABLE 4.3.4

Series B Composite Building
Proposed Shell Construction System
Takeoffs and Costs

ITEM	DESCRIPTION	QTY	UNIT COST	ITEM COST	% OF TOTAL
1	foundation c/w slab on grade	20,000 ft ²	\$ 2	\$40,000	6%
2	structure:				
	(a) wall and perimeter edge beam contact area	21,000 ft ²	\$ 3	\$63,000	
	(b) wall and edge beam volume	460 yds.	\$ 52	\$24,000	
	(c) wall reinforcement	12 T	\$700	\$ 8,000	
	(d) edge beam reinforcement	29 T	\$600	\$17,000	
	(e) prestressing strand	13 K	\$ 1	\$13,000	
	(f) shells and central edge beams contact area	23,000 ft ²	\$ 4	\$92,000	
	(g) shell volume	165 yds ³	\$ 52	\$ 9,000	
	(h) finishing	21,000 ft ²	\$ 1	\$21,000	
	(i) #4 grade 60 re-bar @ 6"	32 T	\$700	\$22,000	
	sub-total			\$269,000	37%
3	architectural, mechanical, electrical			\$416,000	57%
	total	20,000 ft ²	36 ¹ / ₂	\$726,000	100%

TABLE 4.3.5

Series C Concrete Building
Shells Formed c/w Concrete
Takeoffs and Costs

ITEM	DESCRIPTION	QTY	UNIT COST	ITEM COST	% OF TOTAL
1	foundation c/w slab on grade	20,000 ft ²	\$ 2	\$40,000	6%
2	structure:				
	(a) wall and perimeter edge beam contact area	21,000 ft ²	\$ 3	\$63,000	
	(b) wall and edge beam volume	460 yds	\$ 52	\$24,000	
	(c) wall reinforcement	12 T	\$700	\$ 8,000	
	(d) edge beam reinforcement	29 T	\$600	\$17,000	
	(e) prestressing strand	13 K	\$ 1	\$13,000	
	(f) shells and central edge beams contact area	23,000 ft ²	\$ 4	\$92,000	
	(g) shell shotcrete, incl. finishing, 10% loss to rebound	182 yds ³ (9 days)	\$117	\$21,000	
	(h) crane	72 hrs	\$ 60	\$ 4,000	
	(i) #4 grade 60 re-bar @ 6"	32 T	\$700	\$22,000	
	sub-total			\$264,000	37%
3	architectural, mechanical, electrical			\$416,000	57%
	total	20,000 ft ²	\$ 36	\$720,000	100%

TABLE 4.3.6

Series C Concrete Building
Shells Formed c/w Shotcrete
Takeoffs and Costs

ITEM	DESCRIPTION	QTY	UNIT COST	ITEM COST	% OF TOTAL
1	foundation c/w slab on grade	20,000 ft ²	\$ 2	\$40,000	6%
2	structure:				
	(a) wall and perimeter edge beam contact area	21,000 ft ²	\$ 3	\$63,000	
	(b) wall and edge beam volume	460 yds.	\$ 52	\$24,000	
	(c) wall reinforcement	12 T	\$700	\$ 8,000	
	(d) edge beam reinforcement	29 T	\$600	\$17,000	
	(e) prestressing strand	13 K	\$ 1	\$13,000	
	(f) shell scaffolding and stringers	20,000 ft ²	\$ 2	\$40,000	
	(g) shotcrete (incl. finishing and 20% loss)	198 yds ³ (10 days)	\$117	\$23,000	
	(h) crane	80 hrs	\$ 60	\$ 5,000	
	(i) #4 grade 60 re-bar @ 7"	28 T	\$700	\$19,000	
	(j) FRC 1037 fibreglass fabric (incl. overlap)	24,000 ft ²	\$ 0.5	\$12,000	
	sub-total			\$234,000	34%
3	architectural, mechanical, electrical			\$416,000	60%
	total	20,000 ft ²	34 1/2	\$690,000	100%

TABLE 4.3.7

Series C Concrete Building
Proposed Shell Construction System
Takeoffs and Costs

LEGEND

- A Series A steel building (table 4.3.1)
- B1 Series B composite building, shells formed c/w concrete (table 4.3.2)
- B2 Series B composite building, shells formed c/w shotcrete (table 4.3.3)
- B3 Series B composite building, proposed shell construction system (table 4.3.4)
- C1 Series C concrete building, shells formed c/w concrete (table 4.3.5)
- C2 Series C concrete building, shells formed c/w shotcrete (table 4.3.6)
- C3 Series C concrete building, proposed shell construction system (table 4.3.7)

FIGURE 4.3.1

Building Structural Cost Results Bar Chart

4.4 Discussion of Results

4.4.1 Building Costs

Building Series A, of simple steel construction, was initially selected to represent one of the least expensive ways of satisfying the requirements of a light industrial building. The results of the design and cost program clearly demonstrate that long span simple steel construction is less expensive than long span composite or concrete shell construction. The total cost of \$30 per square foot does not represent the minimum cost for this type of building. \$25 per square foot could be obtained, depending mainly on the architectural, mechanical and electrical finishes.

Building Series B, of composite construction, was initially selected to represent the least expensive method of building a light industrial building with a concrete shell roof. It was hoped that there would be a great advantage from the low costs of single storey steel construction combined with the proposed cost-reduced shell construction system. It was a surprise to learn that the cost of the composite structure and cladding would be double that of simple steel and 20% more expensive than poured concrete. The main disadvantage arises from the large edge beams required. The costs of the steel alone are greater than the completed structure and cladding of the steel building. The quantity of edge beam steel required could be reduced by about 20% by increasing the rise of the shells to its optimum, but the same cost reducing advantage would also apply to the edge beams of the concrete buildings in Series C, and architectural and life cycle cost considerations would probably prevent such an occurrence. Another problem with the composite system is the attachment of the thin concrete shells to the steel edge beams. The proposed detail uses a long flexible steel strip welded to the edge beam at its centroid, from which would protrude shear studs top and bottom. The ends of the re-bars would be welded to the strip, and the

shotcrete would be applied in a thick envelope around the shear studs. In short, the detail is too complicated, and too sensitive to poor workmanship. The composite system would be best left alone.

Building Series C, of prestressed concrete construction, was initially selected to represent one of the most expensive methods of building a light industrial building with a concrete shell roof. It was anticipated that the costs of single storey formwork would be quite high. Recall that the advantage of concrete for multiple storey buildings is the repetitive use of the formwork. The cost analysis demonstrated that composite construction was in fact more expensive, for reasons already stated.

For the building of shell roofs using traditional formwork, the results indicate a marginal cost advantage for the use of shotcrete instead of poured concrete. The difference of \$6,000 in a \$720,000 project is insignificant and entirely dependent on the assumption that finishing costs for the poured in place shells were \$21,000 due to manual labour rather than machines. For a building of this nature, with large shallow shells, either shotcrete or poured concrete could be used, depending on bids from local contractors. It must be emphasized that the real advantage of shotcrete becomes apparent for non-shallow shells, where double forming would make poured concrete costs prohibitive.

The most important fact from the tables of results is that the use of the proposed fibreglass fabric/steel grid composite reinforced shotcrete shell construction system substantially reduces the cost of reinforced concrete shell construction compared to a traditional system. A cost reduction of over \$30,000, or 12% of structural costs, was realized. The costs of applying the fibreglass fabric and accepting a 20% loss of shotcrete during application were more than offset by a \$3,000 reduction in steel reinforcement and a \$40,000 reduction in temporary formwork. The real savings

could easily be higher, since two important estimates involved were intentionally made to be conservative. The installed cost of the fiberglass fabric was estimated to be \$0.50 per square foot, or 200% of its material cost. By comparison, the installed costs of concrete, re-bar and prestressing strand are 125%, 125 to 150%, and 150% respectively of their material costs. The estimate of \$2 per square foot for the scaffolding and stringers only, used by the proposed shell construction system, was said to be "very safe" by the consulting formwork contractor, in comparison to the complete forming costs of \$4 per square foot used by the traditional shell construction systems.

If the costs for the entire project are examined, the costs for a reinforced concrete light industrial building have been reduced by about 4% from \$36 per square foot for a traditionally formed and reinforced shell roof to \$34¹/₂ per square foot for a composite reinforced shotcrete shell roof. The proposed system could therefore be immediately used to advantage for any project specifying a reinforced concrete shell, but the system cannot yet compete for industrial projects currently being built for \$30 per square foot.

4.4.2 Fiberglass Fabric Function

The strength contribution of the fiberglass fabric was included in the proposed shell design, which resulted in the reinforcement bar spacing being increased from 6 inches to 7 inches. However, the corresponding saving of \$3,000 was less than 10% of the \$40,000 saving realized from the reduced requirements for temporary formwork. The strength contribution of the fabric would therefore be ignored in the design without much sacrifice. It would be enough to know that the fabric was strengthening rather than weakening the shell, and likely contributing to crack control.

On paper the optimum structural location for the fabric is between the layers of the reinforcement grid. In reality the best location is probably on top of the grid, since the two materials would be installed by two different subcontractors. The placement and securing of the reinforcement would be easier for both groups, with the only penalty being a longer and more uncomfortable overhead shotcrete application for the shotcrete contractor.

4.4.3 Conceptual Revision of the Proposed System

The proposed shell construction system succeeded in reducing the total construction costs for a light industrial building with a hypar shell roof by 4% from \$36 for a traditional construction system to \$34¹/₂ per square foot. That cannot yet be considered competitive with other structural systems which can be built for \$30 per square foot or less.

The initial proposal in Chapter 2 did not include any temporary scaffolding for the shells, but some scaffolding was finally introduced in Chapter 4 to prevent unwanted reinforcement sag and high edge beam bending moment. If the system is to become cost-competitive, then temporary scaffolding must be entirely eliminated. This can be accomplished by adopting a roof configuration which has been specifically designed for a cable net. Reference 45 gives many examples proposed by Otto, and figure I.1 illustrates one in particular that could be applied to the light industrial building in question. The revised system would function as follows:

- (1) Position the cables and apply a small pre-tensioning force to each cable to stabilize the cable net.
- (2) Position the fiberglass fabric.
- (3) Apply the shotcrete envelope from a truck-mounted crane. Allow the shotcrete to cure.

The completed assembly might be considered as a shell under working load conditions, but in fact the cables would be designed strictly as a cable net. At advanced loading conditions, with the shotcrete surface cracked, the total dead and live loads would be carried solely by the draped longitudinal cables.

Appendix I shows a very speculative design and cost summary of this "rigid cable net" concept as it might be applied to the light industrial building plan. Costs are assumed to be the same as the Series C buildings except for the roof, and three eighths inch diameter prestressing strand is assumed to be sufficient for the cable net. The further \$2¹/₂ per square foot reduction to \$32 can be attributed mainly to the removal of all temporary scaffolding. Despite the fact that a proper design has not been done and that there are major engineering problems with the concept as illustrated (a very low central rise of the arch, for example), a rigid cable net appears to have the potential to be competitive with the low cost roof systems. There is justification for further research.

4.4.4 Summary

The questions posed in paragraph 4.1, concerning the cost performance of the proposed fiberglass fabric/steel grid composite reinforced shell construction system, have been answered by the building cost program.

The proposed system reduces construction costs compared to a traditional shell construction system. Costs are reduced due to the strength of the woven fiberglass fabric reducing the amount of reinforcing steel required, and the system as a whole decreasing the need for temporary formwork. The strength contribution of the fabric can be ignored without a great cost sacrifice.

The proposed system is not competitive in cost with today's commonly used long-span structural systems. A sample light industrial building can be built for a total maximum cost of \$30 per square foot, whereas the proposed system cannot be executed for less than \$34¹/₂ per square foot. However, the proposed system represents a 12% structural cost saving or 4% total cost saving compared to traditional methods, and can therefore be used to good advantage in any project where a shell is specified.

The proposed composite reinforced shell construction system can be improved by completely eliminating temporary formwork. A "rigid cable net" roof appears to have the potential to reduce total costs by a further \$2¹/₂ per square foot to \$32 per square foot. Further investigation, including a detailed design and cost analysis, is justified.

It is important to note that the results of the building cost program might not be applicable to countries other than Canada. Such factors as the availability of raw materials, existence of shotcreting equipment, labour costs and design loads might all combine to increase or decrease the value of the proposed shell construction system.

CHAPTER 5

CONCLUSIONS AND RECOMMENDATIONS

5.1 Shotcrete Quality Control

Shotcrete quality control was not originally intended to be a subject of study in this thesis, but results of the prism testing program disclosed some important information.

There are two main conclusions:

- (1) Cutting and testing prisms from a sample plate of equal thickness as the shell is a convenient and satisfactory process for providing an accurate assessment of the overall quality and in-plane compressive strength of a thin shotcrete shell. The method could be used independently or in conjunction with standard cylinder tests.
- (2) Shotcrete quality in thin applications such as reinforced shells can be highly variable despite the efforts of an experienced nozzleman. The water/cement ratio and air pockets are two major factors. Because of the high standard deviation of the prism test results, the specified shotcrete strength should be at least 1200 psi. over the design strength, and the nozzleman should be equipped with a calibration device at the water nozzle to indicate the expected shotcrete strength corresponding to the water flow.

There is one recommendation for future research:

- (1) Conduct lab experiments to discover the compressive strength and standard deviation correlation between the test results of shotcrete prisms cut from plates and standard shotcrete cylinders. This must be accomplished before the cut prism method could be used independently.

5.2 Planar Shear Strength Testing for Square Reinforced Shotcrete Plates

In general, the shear strength test program was a qualified success.

There are three main conclusions:

- (1) The uniform series of vertical parallel cracks evident at the plates' failure indicated that the test concept of a hinged apparatus enveloping a square plate is satisfactory for the application of pure shear. Apparatus design flaws prevented a completely successful test program by applying an unwanted flexural prestress and by allowing the pullout of the main reinforcement grid.
- (2) From the predictable failures of the fibreglass fabric composite reinforced plates, there was some verification of the failure model which assumed that reinforcement would fail in tension.
- (3) The test program verified that woven fibreglass fabric is suitable as reinforcement for the proposed composite reinforced shotcrete shell construction system. The only other fabric considered, woven wire fabric, was judged to be unsuitable with respect to cost, handling and reinforcement characteristics.

There are four recommendations for future research:

- (1) Conduct a series of flexural tests on composite reinforced plates containing woven fibreglass fabric and a steel reinforcement grid. The tests will determine local bending strength and any possible tendency for delamination of the bottom surface away from the fabric.
- (2) Conduct another series of plate shear tests, using the proposed revised apparatus, to check the effects of varying the fabric's weave and location, to compare the reinforcement characteristics of re-bar versus prestressing strand versus wire rope, to fully verify the tensile failure model, and to check the prediction that

failure of the reinforced plates will exhibit plastic behaviour.

- (3) Test the crack control capabilities of the fibreglass fabric with respect to temperature and shrinkage.
- (4) If financially possible, build a series of full scale hypars using the proposed system to verify technical feasibility and construction costs. The hypars would be load tested to verify cracking and ultimate strengths.

5.3 Composite Reinforced Shotcrete Shell Construction

There are two main conclusions concerning the shell construction system proposed in this thesis:

- (1) The results of the building cost program show that the use of fibre-glass fabric/steel grid composite reinforced shotcrete reduces the costs of shell roof construction, compared to traditional methods, by two means; the strength contribution of the woven fibreglass fabric decreases the amount of reinforcing steel required, and the system as a whole decreases the need for temporary formwork. For the light industrial building examined in this thesis, total project costs were reduced from \$36 per square foot using poured concrete walls and traditional shell roof construction methods to \$34¹/₂ using the proposed system. The savings are not of a great magnitude, but the system could nevertheless be used to advantage in any project requiring reinforced concrete shells.
- (2) The costs of the proposed system are still higher than those of today's commonly used long span structural systems. For example, a simple steel system applied to the same light industrial building can be built for \$30 per square foot or less. However, with modifications that would completely eliminate temporary formwork, the proposed system has the potential to be competitive.

Finally, there is one recommendation for future research:

- (1) Because of the demonstrated cost-competitive potential of the proposed shell construction system, an optimization study is justified.

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APPENDIX A

GUNITE; GCA BROCHURE G-76

A large portion of the most recent Gunite Contractors Association brochure is reproduced in Appendix A. Of prime concern are the pages discussing specifications and architectural and structural applications.

The brochure was obtained from A. Bruce Benson Limited, 1810 Bank Street, Ottawa, a regional swimming pool contractor specializing in shotcrete (gunite) swimming pools. Further information can be obtained from Mr. A. Bruce Benson.

of GUNITE

Because of the rapid growth of the Gunitite industry immediately following World War II, it is often thought of as a post-war development. However about 1895, Dr. Carlton Akely, curator of the Field Museum of Natural Science in Chicago, developed the original cement gun. Dr. Akely was searching for a method to apply mortar over skeletal matrices to form the shapes of prehistoric animals, since conventional troweling could not form the convoluted shapes of the musculature systems of these giant reptiles and mammals. His search for a solution to the problem led him to try to spray a cement, lime and sand mortar. He failed with this method but his next attempt resulted in the original cement gun. In a single-chambered pressure vessel, he placed a mixture of sand and cement. Compressed air was then pumped into the chamber containing the mixture, forcing the material through an opening and into a material-conveying hose. As the material was ejected from the end of the hose, it passed through a spray of water which hydrated the mixture.

This crude apparatus was adequate to prove the theory of the process. Later refinements, such as another compression chamber and the addition of an agitating geared feed wheel made it possible to achieve a continuous flow of material. The process was patented in 1911, and except for the improvements in ancillary equipment, such as portable, large size air compressors, mixing and conveying devices which have increased production capacities, there has been literally no change in the Gunitite method of construction to date.

of the ASSOCIATION

Immediately following World War II the use of Gunitite in every facet of construction increased by leaps and bounds. It also became apparent that architects, engineers, designers and specification writers needed a source of expert information to enable them to design for Gunitite construction. In order to meet this demand, individuals started meeting informally to pool their knowledge and establish basic design criteria. Most of these individuals had been directly involved with the industry for a minimum of twenty years.

* This tremendous pool of first-hand knowledge was incorporated into a set of recommended practices and general specifications. In 1951, the Gunitite Contractors Association was formed by active, experienced firms. In February 1952, the Association was incorporated as a non-profit corporation for the express purpose of disseminating standards of the industry to anyone in need of this information. The Association has remained in continuous operation to date, and is internationally recognized as the fount of information and expertise. All requests for information are handled with the greatest dispatch.

The Association has, over the years, spent a great amount of money on research and development and has accumulated a vast reservoir of technical data to aid engineers in design. The Association also has expert speakers available to schools, colleges or other interested groups.

*All of the above listed services are available at no cost to members of the architectural, engineering and general construction industries.

In order to reproduce lifelike prehistoric animals and giant reptiles such as the one pictured above, the original cement gun was conceived and developed. This is why it all started.

HYDRATION

Only that amount of water necessary for the proper hydration of the mix is used. This water is added to the dry mix at a special premixing nozzle immediately prior to placing on the desired surface.

COMPRESSIVE STRENGTH.

A mix of 1:4½ usually produces a 7-day strength of 2400 psi and a 28-day strength of 4000 psi. As indicated under "Gunitite Characteristics" the more conservative design strength of 3000 psi at 28 days is advocated for this mix. In many cases "rich" mixes such as 1:3 are specified when the required design strength could be adequately met by a 1:4½ mix. Besides being uneconomical, such specifications may produce Gunitite which tends to be brittle and more susceptible to fine cracking. The specifications of the Association are based on a 1:4½ mix.

REBOUND - RELATION TO STRENGTH.

Due to the method of placement at fairly high velocities a certain proportion of material is rejected. This rejected material is called "rebound" and consists mainly of sand particles. It can be seen that if a 1:4½ mix is passed through the cement gun, the resultant mix in place is much "richer", hence of higher strength, than the original mix.

BULKING - RELATION TO STRENGTH.

In many cases the effect of sand bulking is not taken into consideration in the proportioning of Gunitite. A bulking factor of twenty percent is not uncommon. If this fact is not considered where materials are proportioned by volume, the actual mix in place is much "richer" than that specified. This condition is often undesirable as explained under "Compressive Strength" above.

BOND STRENGTH.

Gunitite bonds perfectly to properly prepared surfaces of other materials such as concrete, brick, rock, tile, stone or steel. Numerous tests have been made which indicate that the bond strength of Gunitite exceeds the shearing strength of good quality brick or concrete against which it is applied. In tests, loads in excess of 600 pounds per square inch in shear were sustained by the bond between brick and Gunitite, final failure being in the brick.

WEIGHT.

Gunitite will weigh 140 to 152 pounds per cubic foot depending upon the type of sand used. Lightweight aggregates can be used to further reduce weight as in concrete.

DENSITY.

Because it is applied under pressure with a rejection of excess material, Gunitite is an extremely dense product.

ELASTIC PROPERTIES.

Tests indicate that the modulus of elasticity of Gunitite approaches 4,670,000 pounds per square inch (see "Engineering News Record," August 31, 1933).

EXPANSION.

Numerous tests have indicated that the coefficient of expansion of Gunitite is almost identical to that of low carbon steel.

DIMENSION.

Gunitite can be successfully applied to any desired thickness with uniform quality throughout.

FIRE RESISTANT QUALITIES.

Gunitite is a highly economical medium of fireproofing for steel structures, both because of its natural fire resistant qualities and because of the fact that it is adaptable to placing on irregular shapes and can be finished to true and accurate lines. Gunitite is well adapted to firewall construction in buildings, bridges and piers.

RESISTANCE TO CORROSION.

Gunitite adheres better than poured concrete to structural steel and is, therefore, a better form of protection from corrosion.

STRENGTHENING OF STEEL BEAMS.

The strength of steel beams and girders can be increased within reasonable limits by the use of reinforced Gunitite.

ECONOMY.

Gunitite application is based upon the transportation of material through a hose and is consequently more economical in many instances than other types of concrete construction.

The fact cannot be too thoroughly stressed that Gunitite is a specialty product and that its design and application must be executed by men thoroughly experienced in its characteristics and uses.

PHYSICAL PROPERTIES.

When properly mixed and applied, Gunitite is extremely strong, dense and highly resistant to weathering and many forms of chemical attack. It is heat resistant to a high degree and can be made more so by substituting refractory aggregates for part or all of the sand. Resistance to abrasion is extremely high. The bond to other Gunitite, well cleaned masonry or other materials is equal to or greater than the shearing strength of the material to which it is applied.

OPERATING REQUIREMENTS.

Not less than 365 cubic feet of actual free air per minute, at a minimum pressure of 45 pounds per square inch in the gun chamber, is necessary for proper Gunitite placement and adequate "blow-out" jet requirements. Water under a pressure of at least 15 pounds in excess of air pressure is also required. Pressure requirements increase with the height of operation above the gun and length of hose required.

The cement gun should be operated at a minimum air pressure of 45 pounds per square inch on the gun tank when 100 feet or less of material hose is used and the pressure should be increased 5 pounds for each additional 50 feet of hose required.

MIXTURES.

Structural Gunitite proportioned on the basis of one part of cement by volume to various parts of sand by volume will easily attain the following compressive strengths in 28 days:

1:4	4000 lbs. per square inch
1:4½	3000 lbs. per square inch
1:5	2500 lbs. per square inch
1:6	2000 lbs. per square inch

Under proper curing conditions Gunitite will attain at least 60% of the above compressive strengths in 7 days.

CONTROL OF CONSISTENCY.

Consistency of the Gunitite is controlled at the nozzle by the nozzleman.

PREPARATION OF SURFACES.

If bond is required, surfaces to be Gunitied must be thoroughly cleaned of all dirt, oil or foreign matter and all loose, scaly, or unsound material removed.

Gunitite cannot be applied to a surface containing frost or ice.

Gunitite must be protected against freezing.

In some instances, such as in relining of existing concrete reservoirs, bond to existing material may be undesirable. In such cases, the usual procedures should be followed to prevent bond.

CURING.

Proper curing is essential. A light spray of water should be applied as soon as is possible without damage to the surface. After the surface has hardened, it must be kept moist for a period of from five to seven days, depending upon atmospheric conditions.

Satisfactory curing can be obtained by proper application of an approved sealing compound.

DESIGN.

Load tests on slabs show Gunitite conforms to the same design assumptions as ordinary concrete of equivalent compressive strength.

The fundamental principles used in the design of reinforced Gunitite are the same as the fundamental principles for the design of reinforced concrete in accordance with the rules of the American Concrete Institute.

Wire mesh, when used as reinforcement, should have a minimum area in each direction of .0025 times the cross sectional area of the Gunitite section.

NOTE: Inasmuch as Gunitite is readily adaptable to rounded corners without the use of forms, it is suggested that this added feature be utilized for the inside corners of reservoirs or similar locations.

GUNITE General Specifications

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A-1 DEFINITION:

Gunitite as herein specified is a trade name used to designate a mixture of Portland cement and sand thoroughly mixed dry, passed through a cement gun and conveyed by air through a flexible tube, hydrated at a nozzle at the end of such flexible tube and deposited by air pressure in its place of final repose.

A-2 PROPORTIONS:

Unless otherwise specified, all Gunitite shall be mixed in the proportions of 1 part of cement to 4½ parts of sand based on dry loose volume.

A-3 MATERIALS

CEMENT:

Only Portland cements of American manufacture complying with the current issue of "Standard Specifications for Portland Cement," A.S.T.M. C-150 shall be used. Type I or II Portland cement shall be used unless otherwise specified in the detailed specifications.

SAND:

Fine aggregate shall consist of washed sand and shall be hard, dense, durable, clean, sharp and graded evenly from fine to coarse in accordance with the "Standard Specifications for Concrete Aggregates," A.S.T.M. Designation: C 33-44. It shall be free from organic matter and shall not contain more than 5% by weight of deleterious substances.

LIMITS OF GRADING OF FINE AGGREGATES

Sieve Size	Percent by Weight
Passing a 3/8 inch	100
Passing a No. 4	95-100
Passing a No. 8	65-90
Passing a No. 16	45-75
Passing a No. 30	30-50
Passing a No. 50	10-22
Passing a No. 100	2-8

For proper placement of Gunitite, sand should contain between 3% and 6% moisture by weight. Sand and cement proportion may be corrected to provide for bulking due to sand moisture content. Percentage of bulking can be easily determined in the field, using a process based on the theory that 100% surface saturation by water will develop a material density equal to that of loose dry sand. To run this test simply fill any vertical sided watertight container level full of sand and fill container with water.

Physically measure the settlement of the sand and calculate the percent of shrinkage to vertical depth of the container.

Lightweight aggregates and refractory aggregates may be used in accordance with recommendations of the manufacturer.

WATER:

Water used for hydration at the nozzle shall be fit for drinking and shall be maintained at a uniform pressure which shall be at least 15 pounds per square inch above air pressure at the nozzle.

A-4 REINFORCEMENT:

All reinforcement shall be clean and free from loose mill scale, loose rust, oil or other coatings interfering with bond.

Reinforcement shall be rigidly secured in the position indicated on the drawings. Reinforcing bars shall be at least two and one-half (2½) inches apart in the clear. Minimum clear distance between forms and reinforcement and for cover shall be as shown on the drawing. Space shall be provided for splicing bars in an approved manner. Only round bars or wire mesh shall be used.

A-5 OPERATING REQUIREMENTS:

For lengths of hose up to 100 feet, air pressure at the gun shall be 45 pounds per square inch or more. Where length exceeds 100 feet, pressure shall be increased 5 pounds per square inch for each additional 50 feet of hose required. Constant pressure must be maintained. Nozzles used for structural Guniting shall have a maximum size of 1-5/8".

A-6 REBOUND:

Rebound, recovered clean and free of foreign matter, may be reused as sand in a quantity not to exceed 20% of total sand requirements.

A-7 CONSTRUCTION JOINTS:

Particular care shall be given to formation of construction joints. They shall be sloped to a thin edge and the entire joint shall be thoroughly wetted before adjacent Gunitite is placed. No square joints will be allowed, unless specifically required.

A-8 CURING:

Gunitite shall be damp cured for at least 7 days after placing or by proper application of an approved sealing compound. It shall be mandatory for the Gunitite Contractor to perform the curing operation. No Gunitite shall be placed during freezing weather except when protective measures are taken as with ordinary concrete work. Gunitite shall not be placed against frosted surfaces.

GUNITE General Specifications

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A-9 GUNITE QUALITY:

The minimum ultimate 28 day compressive strength shall be 3000 pounds per square inch (Based on a nominal 1:4½ mix).

The representative of the Engineer may require two test cylinders of Gunitite each day as a material control. Test cylinders shall represent the quality of Gunitite being placed in the structure, and if there is more than one crew or nozzleman on the work, test cylinders shall be made by each nozzleman in rotation so that the tests shall represent the quality of Gunitite being placed by each nozzleman, all as determined by the representative of the Engineer. Each cylinder shall be dated, given a number, the name of the nozzleman making the cylinder and the point in the structure represented by the cylinder.

The Gunitite Contractor shall furnish at his own expense, especially constructed cylinders 6 inches in diameter and 12 inches high, made of ¾ inch square mesh hardware cloth.

The test cylinders of Gunitite shall be made with the same air pressure, nozzle tip and hydration as the Gunitite in the structure at the point where the cylinder was made.

At the end of 24 hours after cylinders are made, the hardware cloth form shall be removed and the cylinders stored in the testing laboratories in accordance with the current issue of "Standard Method of Making Compression Tests of Concrete," A.S.T.M. Designation C 39.

Separate tests of Gunitite cylinders taken at the same place and time shall be made at the ages of 14 days and 28 days and shall be used for correlative purposes only. Compressive strength of structural Gunitite in place shall be determined by cores as specified below.

A-10 CORE TESTS:

For structural Gunitite a minimum of three cores shall be taken for each 250 cubic yards or fraction thereof, of structural Gunitite deposited. Cores shall be obtained and tested in accordance with A.S.T.M. C 42-55. One core shall be removed and tested at a Gunitite age of 14 days, the other two cores at a Gunitite age of 28 days.

Fourteen day cores shall develop a minimum strength of 2200 psi. Twenty-eight day cores shall develop a minimum strength of 3000 psi.

If Gunitite cores show deficient strength, additional cores shall be taken at the Contractor's expense from adjacent areas. Two cores shall be required for each deficient core. Should either additional core prove deficient, the defective Gunitite shall be removed and replaced. Should such deficiency be evident in 14 day cores, on approval of the Engineer, the Contractor may proceed with the work on his own responsibility until the 28-day cores are tested.

Where conditions preclude the possibility of obtaining cores from the Gunitite in place, the Engineer may approve cores taken from a representative test panel made at the same time and under the same conditions as the structural Gunitite.

A-11 WORKMEN:

Only foremen, nozzlemen, gunmen and rodmen with at least three years of structural experience shall be employed and satisfactory written evidence of such experience shall be furnished the Engineer or his representative upon demand.

A-12 ELIGIBLE GUNITE CONTRACTORS:

The Contractor, to be eligible as a bidder, must have had at least five (5) years' experience in Gunitite construction and must list at least twenty significant structural Gunitite installations which he has constructed and which, on investigation, have been found to be completed in a satisfactory manner. Bidders with limited experience are advised that very close scrutiny will be given all phases of this work. Unsatisfactory work will be immediately rejected. The Contractor is cautioned against attempting to substitute for specific equipment, items which have not been previously approved and items which may not meet all requirements of design and quality. Inferior equipment will not be accepted.

NEW GUNITE STRUCTURES

Specification B

B-1 SCOPE:

This specification with the General Specifications shall govern in all cases where Gunitite is placed in a new structure.

B-2 FORMS:

Forms shall be adequately braced to insure against excessive vibration. Forms shall be built so as to permit the escape of air and rebound and to facilitate the placing of Gunitite. Wall intersections shall be formed in such a manner as to afford a minimum loss of time in Guniting the intersection. This may be accomplished by the installation of short removable bulkheads at these points. Free standing columns may be formed on three sides or two adjacent sides whichever is practicable. Pilasters may be formed on two opposite sides. Forms for beams may be constructed of a soffit and one side or a segmented soffit and one side to permit Gunitite placement in supported layers. Bond beams shall be formed with a soffit only. Shores shall be provided below the soffit in such a manner that no deflection will occur under the load to be superimposed. Bucks shall be installed around all openings.

Sufficient time shall be allowed other crafts for installation of equipment or materials which must be fastened to the forms. Form surfaces shall be cleaned prior to application of Gunitite.

GUNITE: General Specifications

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B-3 REINFORCEMENT:

See General Specifications. (Special requirements for any one job may be listed here.)

B-4 GROUND WIRES:

Adequate ground wires, to be used as screeds, shall be installed to establish the thickness and surface planes of the Guniting work. Ground wires shall be placed so that they are tight and true to line and in such a manner that they may be easily tightened.

B-5 PLACING OF GUNITE:

Whenever possible, except when enclosing reinforcing steel, the nozzle shall be held at right angles to the Guniting surface at a distance of $2\frac{1}{2}$ to $3\frac{1}{2}$ feet. When enclosing steel, the nozzle shall be held so as to direct the material around the bars. A nozzleman's helper equipped with an air jet shall attend the nozzleman and blow out all rebound, sand, etc., which may have lodged on the forms, steel or Guniting. Guniting material shall emerge from the nozzle in a steady, uninterrupted flow. When flow becomes intermittent for any cause, the nozzle shall be diverted from the work until the flow again becomes constant. Hydration shall be thorough and uniform without the use of excessive water.

In shooting walls, columns and beams, application shall begin at the bottom and shall completely embed the reinforcement. The limit of the thickness and height has been exceeded when the material begins to sag.

In shooting beams, the nozzle shall be held at right angles to the surface of application.

In shooting formed slabs, the nozzle shall be held at a slight angle to the work so that rebound is blown onto the completed portion from where it shall be removed. The air jet shall be constantly employed to keep the area of placement free of rebound and all loose material. Wherever possible, slabs shall be completed in one operation.

Reinforcement shall be cleaned of any previously deposited Guniting which might prevent proper bond to reinforcement. Sufficient time shall be allowed between layers for the material to set. Before set has taken place, and before placing any succeeding layer, laitance shall be removed by brooming. Any laitance which has set shall be removed by sandblasting. Surfaces shall be damp at all times.

Rebound pockets, sags or other defects shall be carefully cut out and replaced with new Guniting or hand patched in a manner satisfactory to the Engineer.

B-6 FINISHING:

Upon reaching the thickness and planes outlined by forms and ground wires, the surface shall be rodged to true lines. Upon completion of rodging, ground wires may be removed. If possible, the finish coat shall be applied so that Guniting is not shot over the finished work. All exposed surfaces shall be finished to straight and true lines, as indicated on the drawings. Finish shall be as indicated below:

- a. Steel Trowel
- b. Wood Float - Granular texture
- c. Rubber Float - Coarse texture and finish
- d. Sack - Coarse sand texture with wavy outline
- e. Broom - Natural finish broomed
- f. Rodged - Natural finish removed by use of a rod
- g. Gun - Natural finish as left by nozzle

It should be pointed out that all Guniting finishes tend toward a coarser texture than plastered surface due to coarser aggregate.

REHABILITATION OF CONCRETE AND MASONRY STRUCTURES Specification C

C-1 SCOPE:

This specification with the General Specifications shall govern in all cases where Guniting is used to rehabilitate concrete or masonry structures.

C-2 CUTTING AND STRIPPING:

Where plans indicate that concrete or masonry is to be cut or stripped, this work shall be done by experienced workmen equipped with suitable power tools. Where, in the opinion of the Engineer, the use of power tools may damage the structure, the cutting shall be done by hand. Extreme care shall be taken that portions to remain are trimmed in such a manner so as to facilitate placing of Guniting and to present a neat finished appearance, if exposed.

C-3 PREPARATION OF SURFACES:

All concrete or masonry surfaces to receive Guniting shall be thoroughly cleaned by sandblasting. Sandblasting shall be done by experienced workmen using approved equipment and suitable sandblasting materials. Prior to receiving Guniting, all surfaces shall be cleaned of dust and debris, using compressed air and water. Concrete and masonry shall be thoroughly wetted before application of Guniting, but shall not be so wet as to overcome suction. Free water shall not remain on the surface to be Gunited, nor shall surfaces be so dry that there is excessive absorption of moisture from the Guniting.

GUNITE General Specifications

D-1 PROTECTION OF SURFACES:

Surfaces which do not receive Gunitite, such as wood framing, etc. shall be protected with waterproof paper or other adequate means.

D-2 BONDING TO STRUCTURAL STEEL:

Steel to be embedded or fireproofed with Gunitite shall be cleaned of all substances which may prevent bond. Steel members shall be sandblasted where necessary to remove paint or scale.

D-3 FORMS: Same as B-2

D-4 REINFORCEMENT: Same as B-3

D-5 GROUND WIRES: Same as B-4

D-6 PLACING GUNITITE: Same as B-5

D-7 FINISHING: Same as B-6

EARTH LININGS Specification D

D-1 SCOPE:

This specification with the General Specifications shall govern in all cases where Gunitite is placed against earth for channel, ditch, reservoirs, lake and pool linings.

D-2 PREPARATION OF SUBGRADE:

The surfaces against which Gunitite is to be applied shall be presented in a thoroughly compacted condition and shall be accurately trimmed to line and grade as shown on drawings. All dry surfaces shall be wetted before application of Gunitite, but Gunitite shall not be placed on any surface which is saturated, spongy or where free water exists.

D-3 REINFORCEMENT:

Reinforcement shall be as indicated on the plans, however, there shall be a minimum of .0025 times the cross sectional area of the lining in each direction. Wire mesh shall lap a minimum of one mesh spacing and laps shall be securely tied.

D-4 GROUND WIRES:

Ground wires, if required, shall be installed in such a manner that they accurately outline the section of the linings as indicated on the plans. They shall be located at intervals sufficient to insure proper thickness throughout and shall be maintained tight.

D-5 HEADERS:

Headers shall be installed where required or indicated on the plans and shall be securely set to line and grade.

D-6 PLACING OF GUNITITE:

Gunitite shall be placed in accordance with the Specifications. There shall be a nozzleman's helper continuously in attendance to accomplish at the

direction of the nozzleman the proper positioning of the reinforcement and cleaning of joints. Rebound shall not be incorporated in the work. Whenever possible, Gunitite shall be installed to the full thickness of the lining in one application. Where required, Gunitite shall be carried over the side in a berm not less than the lining thickness and/or a cut-off wall constructed to prevent water seeping under the lining.

D-7 FINISHING:

Gunitite shall be placed to the thickness indicated on the plans. The surface shall then be finished as specified.

D-8 WEEP HOLES:

Where weep holes are required they shall be installed in accordance with the drawings.

D-9 CURING:

Gunitite shall be promptly cured in accordance with Paragraph A-8 of the General Specifications.

D-10 OVERBREAK:

Overbreak is defined as excess material that is used to establish line or grade on any application where the original thickness has been increased due to over-excavation, erosion or lack of header boards. When present, overbreak shall be determined by a joint survey of the owner and/or general contractor with Gunitite contractor representatives. Costs for overbreak shall be determined and set forth in writing acceptable to both parties prior to placement of any materials.

FIREPROOFING Specification E

E-1 SCOPE:

These specifications shall govern work in which Gunitite is applied to steel columns, girders and beams for purposes of fireproofing.

E-2 PREPARATION OF SURFACE:

All steel surfaces to be encased with Gunitite shall be thoroughly clean and free of rust, paint scale, oil, grease, dirt or other materials which would prevent bond.

E-3 REINFORCEMENT:

Reinforcement shall consist of a system of electrically welded wire mesh and/or bars. The mesh shall be furred out from the member to be encased so that the mesh occupies a position in the center of the encasement. Mesh laps shall be a minimum of 4 inches and shall be securely tied with wire at intervals of 12 inches. In general, the mesh shall follow the outline of the member.

E-4 GROUNDS:

Grounds or forms shall be placed at corners and along plane surfaces in such a manner that the full thickness is insured at all points.

A. INTRODUCTION

Although the placement of concrete and mortar by pumping is not new, the method of air-placing the material has been used only in the past few years. Through growing familiarity with air-placed concrete or mortar architects, engineers and contractors are now able to produce designs and structures previously considered uneconomical. This recommended practice is designed to avoid misapplication of the material and illustrate the varied and many uses to which air-placed concrete or mortar is adaptable.

B. DEFINITION

Air-placed concrete or mortar is a proportioned combination of Portland cement, aggregates and water which is mixed by mechanical methods and pumped in a plastic state to the nozzle, where air is added to expel the material. The force of the air jet compacts the material.

C. GENERAL DESCRIPTION OF AIR-PLACEMENT OF CONCRETE OR MORTAR

1. The concrete or mortar is delivered to jobsite premixed or mixed in a mixer at the site.
2. The designed mix is discharged into a pump which passes the material through the delivery hose to the nozzle.
3. Compressed air is delivered at the nozzle by a separate air line, which propels the material onto the surface or form.

D. ADVANTAGES OF AIR-PLACED CONCRETE OR MORTAR

The advantages of air-placed concrete or mortar are:

1. Allows the use of two mixes:
Air-placed mortar, which consists of 100 percent washed, graded sand and the required cement.
Air-placed concrete, which consists of a mix of 20 percent to 30 percent pea gravel, washed, graded concrete sand and the required cement.
2. Does not require a mixing plant, except in remote areas not serviced by ready-mix trucks.
3. As the material is ready-mixed, the mixes can be designed to fit specifications since the material weights, mixing times and water content are recorded on delivery ticket.
4. Various tests have shown that air-placed concrete or mortar conforms to the same design assumptions as ordinary concrete of equivalent strengths.

5. Air-placed concrete or mortar bonds well to properly prepared surfaces of other materials, such as concrete, brick, rock, tile, stone or steel. Tests have been made which indicate that the bond strength of air-placed concrete or mortar compares favorably with concrete of the same design.
6. Air-placed concrete or mortar adheres well to structural steel and is a good protection against corrosion.
7. The rigidity of steel beams is increased, within reasonable limits, by the use of air-placed concrete or mortar.
8. This method is readily adaptable to difficult construction problems, especially where it is inaccessible to buggies, wheelbarrows or cranes, etc.
9. Since less labor is required and production rates are high, the economy is easily evident.

E. SPECIAL REQUIREMENTS FOR AIR-PLACED CONCRETE OR MORTAR

The special requirements for air-placed concrete or mortar are:

1. A contractor performing on air-placed concrete or mortar structures shall have had at least three (3) years experience. Upon request of the owner or engineer, the contractor may be required to list at least ten structural air-placed concrete or mortar installations that have been completed in a satisfactory manner.
2. Air-placed concrete or mortar should be placed by skilled operators only, as the structural value and texture of the finished surface are greatly affected by the skill of the workman. Three (3) years of experience will produce the required skills.
3. All surfaces may be finished to produce a uniform texture.

F. MATERIALS

Limits of grading of fine aggregates
(See Sect. A-3)

Pea Gravel Grading:

Sieve Size	Percent Passing
1/2 inch	100
3/8 inch	90

G. EQUIPMENT

1. Mixer - The mixing shall normally be done by the ready-mix truck and delivered to the job site.

- 2 Pumps - The pumps used for this method of application are either rotating roller squeeze pumps or a positive displacement piston type. A collecting hopper at the pump receives the pre-mix concrete from the ready-mix concrete truck. From there the concrete is pumped into a three (3) inch diameter tube which again reduces to a two (2) inch single or double line.
- 3 Hose and Nozzle - The practical limit of discharge hose length for very stiff mixes is approximately 100 feet, although it is possible to pump a distance of 200 feet by using metal pipe in place of hose. An air ring inside the nozzle injects high pressure compressed air into the flow of material for propulsion toward the surface.
- 4 Air Compressor - Any standard type of compressor, capable of providing a minimum of 100 cu. ft. of air per minute per nozzle, will provide sufficient air to produce an excellent end product.

H. RECOMMENDED STANDARDS

1. Transporting ready-mix Concrete or Mortar - The mixing time for materials delivered by ready-mix trucks to the job site, shall not exceed 1.5 hours or 250 revolutions of the drum, whichever comes first. Additional water may be added at job site only if requested by the Engineer. When additional water is added, the drum shall be rotated a minimum of 30 additional revolutions. The ready-mix plant shall certify the material for weight, water content and mixing time.
2. Preparation of the Surface or Subgrade - See Sections C-3, D-2, D-10.
3. Reinforcement - See Section A-4
4. Ground Wires - See Section B-4
5. Forms - See Section B-2
6. Proportioning - Structural air-placed concrete or mortar with these cement and normal water contents may be assumed capable of attaining the following compressive strengths at 28 days:

6.5 sack/cu. yd.	3000 lbs. per sq. in.
7.0 sack/cu. yd.	3500 lbs. per sq. in.
7.5 sack/cu. yd.	4000 lbs. per sq. in.

Properly cured and protected air-placed concrete or mortar can be assumed to attain approximately 60 percent of the above values at seven (7) days.

Unless a strength of 3000 lbs. per square inch is specified, air-placed concrete or mortar should be proportioned as follows:

Six and one-half (6½) sacks of cement per cubic yard, 80 percent sand, 20 percent pea gravel for air-placed concrete and 100 percent sand for air-placed mortar. Coarse aggregate may vary according to the type of finish required, with a maximum of 30 percent pea gravel per cubic yard.

Air-placed concrete or mortar will weigh approximately 145 pounds per cubic foot, depending upon the type of aggregate used.

No admixture shall be used without the permission of the Engineer. If admixtures are used to entrain air, to reduce water-cement ratio, to retard or accelerate setting time or to accelerate the development of strength, they shall be used at the rate specified by the manufacturer and approved by the Engineer.

In no event should a slump greater than four (4) inches be used in air-placed concrete or mortar; however, due to the type of work involved, this slump will decrease with different applications. As the work approaches the vertical, the maximum slump shall not exceed one (1) inch. Only that amount of water necessary for the proper slump of the mix is used. If necessary due to weather conditions, additional water may be added at the job site under the directions of the Engineer.

7. Placing Air-placed Concrete or Mortar - See Section B-5

Particular care shall be given to formation of construction joints. They shall be sloped to a thin or square edge as required by the Engineer, and the entire joint shall be clean and thoroughly wetted before air-placed concrete or mortar is placed.

8. Finishing - See Section B-6

9. Curing - See Section A-8

10. Tests for Compressive Strength - See Section A-10

SUMMARY

This recommended practice for air-placed concrete or mortar provides the architect and engineer with a guide to produce an economical variety of concrete shapes for their clients in the fields of structures, water control, water storage, slope protection and special architectural features.

The use of air-placed concrete or mortar is certain to expand as architects and engineers find additional areas of use. Adherence to the above recommended practices will aid in achieving excellent end products.

The use of Gunitite as a structural building system, plus its almost unlimited range of architectural applications give the designing architect a latitude for imaginative design that no other contemporary construction method can provide.

Pictured on these pages is a small sampling of projects where architects have used Gunitite to give them a free rein in producing graceful and beautifully shaped structures.

Vaulted arches roof this graceful building, giving it an ethereal beauty, not unlike the convoluted shapes of the more attractive mollusk shells picked up on the beaches of the world.

A memorial tower constructed in an unfolding tapered spiral, similar to a chambered nautilus, was a practical Gunitite application to develop this imposing structure.

These eye pleasing contoured shapes were made possible through the use of single forms and high strength, light weight Gunitite. To give you an idea of size, the highest shape soars to approximately one hundred twenty five feet.

A central dome section, gracefully flowing into diverging arches, forms the structure and roof for the nave and quadruple apse's in this modern house of worship.

A free span thin shell partial dome roof is supported on four buttressed supports with exterior raised beams distributing the load. Lightweight Gunite was used throughout the entire roof.

This unusual shape adds a new dimension to the famous attractions of San Francisco. The new St. Mary's Cathedral is a functional example of the tremendous latitude available to Architects who make use of the many possible values of Gunite construction.

One process combining aesthetic appeal and structural validity, makes Gunitite a unique method of construction.

Multiple use of single back-up forms, on-site portable mixing and placing equipment, insures owners of speed of construction offered by no other process.

White cement and snow, white aggregate give this apartment building a permanent maintenance-free exterior. A cast-in-place reinforced concrete frame, with Gunitite filler walls, was coated with one inch of white Gunitite to give this building its lasting beauty.

The architectural capabilities as well as the structural values of Gunitite are graphically pictured in this modern apartment tower. Shadow box frames surrounding the windows were easily and economically constructed with a minimum of forming to give this rare exterior appearance.

This contoured structural roof shell lends a unique relief to the conventional appearance of most civic buildings.

Even though it is not visible, Gunite affords the structural validity to this modern office building. All of the exterior walls of Gunite had incorporated in them dove-tailed anchor slots to form secure anchorage for the exterior cast stone facing.

The clean sharp lines of this desert auditorium were made possible by combining the structural and architectural properties of Gunite.

Mission style architecture, with its arches, splayed columns, molds and buttresses, forms an ideal marriage with the Gunite process. All of the architectural detail work, plus the basic structure, are offspring of this marriage.

SUNSHINE

Using the shaped earth as a form, the Guniting method is being used extensively as a structural element for construction of exterior basement retaining and building line walls under new structures. It provides plumb true surfaced walls, and when combined with temporary shoring systems, is unlimited as to depth of excavation or wall thickness. Guniting is also frequently used as a method of underpinning structures adjacent to property lines. By incorporating integral waterproofing agents in the mix, it is possible to construct walls below existing ground water tables and still guarantee a waterproof wall.

This picture gives the viewer some idea of the immensity of what will become the world's largest parking structure. This bit of trivia is authenticated by no less than Guinness Book of Records, 1975 Edition.

A further progress picture of the Century City parking structure, where Guniting walls up to twenty four inches in thickness are combined with soldier beams and stressed tieback rods, to construct the perimeter walls of this massive structure.

This view of the wall support system, where this area of the structure has now been excavated to its maximum depth, shows a wall approximately one hundred feet in height. Note the dowelled, recessed keyways which will tie in floor slabs and perpendicular walls. The small rectangular areas on the completed Guniting wall are cutouts for the stressed tieback rods. After the structure is complete, these rods may be removed and these cutouts flush filled to match the troweled surface of adjacent walls.

This in-progress picture shows the simplicity of single forming, to construct Gunitite walls for spiral access ramps for underground parking.

Air-placed concrete was used to construct this vertical reinforced concrete retaining wall.

Building walls from the top down may not sound practical, but here is a graphic example of how it is done. The top lift is supported on the first level of excavated material. Subsequent lifts are placed in a sequential operation similar to a typical underpinning operation. There is no limit to depths that can be achieved by this method. However, to maintain the verticality of the new wall, it may be necessary to install some type of temporary lateral support until the floor diaphragms are completed.

Since the advent of precasting and tilting up of building walls, a major problem in this construction method has been in the panel connection system.

The use of Guniting has been instrumental in helping solve this problem. The simplicity of forming and speed of placing have induced many contractors to use the Guniting method.

Pilaster sections are formed with simple side forms that are attached to the panels prior to erection. If panels are to have a space between them, a single back up form will also be required. This form may be eliminated if the panels abut.

Contingent upon certain job conditions, connections can be placed from either side of the wall. This latitude makes it possible to develop architectural features that might not be possible by other methods.

For the best job results it is recommended that an expert in this type of application be consulted. Such experts are available through the Guniting Contractors Association at no charge.

Fork lifts with safety platforms are used almost exclusively in lieu of other scaffolding methods for placing and finishing Guniting connections. Tight control of ascent and descent rates keep the nozzleman and finisher at the ideal working level at all times, thus facilitating placing and finishing.

This picture of finished pilasters indicates the plumb trowelled surface possible to match surfaces on adjacent panels. Note the modification of the pilaster immediately below the bond beam, to provide bearing for the beam. Also, note how the pilaster has been truncated below the parapet line to form a seat for the roof beam member. In this instance the roof beam will be reinforced concrete as the vertical pilaster steel has been allowed to project the proper distance, to provide adequate penetration into the roof beam. Anchor bolts, weld plates or saddles are easily installed for other types of roof structure beams.

Note the placing technique shown in this picture. The downward angle of the nozzle and the sloping face of the in-place material. This proven technique enables the nozzleman to apply against a surface that is free of rebound and does not require supplementary cleansing by another operator using an air lance. Note simplicity of forming for Guniting.

Pre-set architectural veneers which require a structural wall backing make the use of Gunitite a natural selection. The elimination of forming, plus the ability of Gunitite to bond to practically any material surface, guarantees a fast, economical job.

Any number of permanent finish architectural materials, such as single course masonry, marble, tile, thin section precast mosaic, tesserae or exposed aggregate panels, porcelainized metals and anodized aluminum have been backed up with a structural Gunitite wall.

This attractive permanent finish of decorative precast concrete panels was preset and a structural Gunitite wall was applied against the rear face. The reverse face of the precast panels is deliberately roughened and wire broomed to remove laitance and produce a superior bonding surface for the Gunitite. When completed the section is comparable to a monolithic wall.

In this department store building, selected random section masonry units were course laid and backed with a structural Gunitite wall section. Typical of this method of construction, no forming is required. The straight true line interior Gunitite wall can be finished ready to receive paint, or rodged to true lines to receive plaster.

An exterior view of this combination Gunitite and brick wall building.

Repair of concrete surfaces that have been damaged by fire, salt water intrusion, acid attack, frost action, erosion or any one of many other causes, is a natural function of Guniting. Because of its adaptability, plus its ability to develop bond strength to properly prepared concrete surfaces, that is higher in value than

that of the original material, make it a must where such repairs are required. The properties of Guniting that enable it to reintegrate fractured concrete are analogous to arc welding in structural steel construction.

A dock structure, severely damaged by salt water penetration into the reinforcing steel, is shown here being prepared to receive Guniting. All fractured material has been removed, new reinforcing added as required, and all surfaces have been sand blasted, to create a perfect bonding surface.

Shown here in action is the Guniting coating being applied.

A picture of the third in a series on this page, shows the completed Guniting. In this type of installation where appearance is not important, the Guniting is not finished in any way after it is deposited. However, it should be pointed out that it could have been restored exactly to original lines, duplicating all chamfers or any other architectural details.

Reinforcement

Disintegrated concrete, from any cause, can be easily and quickly repaired by the Guniting method. This picture shows Gem Lake Dam, located at an altitude of 9000 feet, which had been severely eroded by intermittent freezing and thawing. Because of the extremely low temperatures sustained for nine months out of the year, preparatory chipping of the loosened concrete was instituted in one summer, with the Guniting coating applied the following year.

Typical Section of
Gem Lake Dam

Reinforcing mesh has been applied over previously prepared surfaces and Guniting is being applied. At this altitude it required double capacity air compressors to adequately place the Guniting.

After damage to the concrete had occurred below winter reservoir level, the lower parts of the thin arch barrels were thickened in 1925 to gravity section by additional concrete.

Rehabilitation

One of many ideal applications for Gunitite is in the strengthening of existing masonry buildings that were not properly designed to withstand loads imposed by nature, such as earthquakes, floods or gale-force winds; also loads imposed by owners, such as added stories, heavy equipment installations or excessive floor loading of whatever nature.

One of the most common and widely known applications is in repairing damage incurred by earthquakes, or in adding structural values to buildings to resist seismic shocks.

The usual method of construction is to remove one wythe of masonry from the walls, and at designated intervals remove an additional wythe, to form a vertical recessed rib. Dependent on original wall thickness, tie lugs where the Gunitite can penetrate far enough to bond to the opposite face wythe should be installed in each twenty-five square feet of wall. These rectangular tie lugs serve two vital functions: They act in shear, but more importantly - they tie the wall together and reduce the chance of loose masonry units falling during a quake, which could cause physical injuries and greater property damage.

Gunitite can be installed from either side of a wall, which is beneficial on a building where exterior architectural features are to be retained.

Photo shows Gunitite being applied to a column and lintel. Note additional column of steel at both sides of section where second wythe of brick has been removed. Also, note simplicity of forming for Gunitite jamb and lintel.

Sketches (reinforcing not shown) show a typical section, plan and detail of this type of work.

➤ Shown above lines of exposed finished Gunitite on completely rehabilitated school building. All architectural details and exact exterior dimensions have been restored.

As America celebrates its two hundredth birthday, Americans are becoming more and more conscious of the need to preserve their heritage of landmark structures.

From such diverse areas as the Vieux Carre French Quarter of New Orleans to the Missions of California, Gunite is being used to preserve facades of buildings in their original form. Gunite becomes a concealed structural skeleton when applied from the inside of these structures.

Future generations of Americans will be able to point with pride to totally preserved historical buildings that have withstood the ravages of time because a previous generation had the desire, knowledge and the unique values of the Gunite construction method to achieve their aims. Wherever brick, concrete or adobe was used in the original structure, Gunite can be the answer to the conservation of our legacy of historic structures.

Here, a declared historical and architectural California landmark has been made structurally sound by use of Gunite. All of the strengthening work was done from the inside in order to preserve the original exterior architectural appearance.

While it cannot be claimed to be as American as apple pie, the transplanted old London Bridge cannot be denied as an historical landmark. When the stones from the bridge spanning the Thames were removed, number-coded and crated for shipment to the U.S.A., it was discovered that the lowest stones in the arches were badly disintegrated due to frost action and erosion. It was decided that these stones could be reconstructed in place by using Gunite.

Mission San Gabriel Archangel, built in 1771, recently reinforced with interior Gunite ribs and beams to maintain its exterior architectural beauty and to provide structural validity to this queen of the missions.

Huntington Memorial Library recently structurally strengthened with a Gunite overlay.

To protect valuable property from the destructive erosion of wave action, winds or surface waters has long been successfully accomplished by the installation of a simple, lightly reinforced, Gunit blanket. These blankets can be installed over existing contours or over true line grades as desired. Earthtone color can be added to the mix to make the blanket blend harmoniously with adjacent surfaces.

Gunit is used extensively to anchor cobble or massive derrick stone together, to resist wave action. It is also used to protect banks and cliffs on beach front properties from wave erosion. For this type of installation it is advisable to extend the blanket as far below the lowest tide level as is practicable. This can be accomplished by means of a graving dock or by selecting

an installation time from a tide calendar, to determine the lowest minus tide level.

Where an erosion blanket is to be applied to an overhanging, vertical or near vertical surface, it is common practice to install some type of anchorage system. These may be rock bolts, grouted tie rods, driven steel pins or simple sub-surface excavated holes to be filled monolithically with the blanket installation. Where blankets are installed to stop surface water erosion, it is advisable to install sub-surface perimeter cut-off walls, to prevent water from getting under the blanket where it could possibly develop enough pressure to float the blanket. As additional insurance against this hydrostatic pressure, relief or weep holes can be cut at intervals.

➤ Hillside erosion blanket protects expensive home and pedestrians from falling rock.

➤ Slope protection being placed under and adjacent to overhead bridge spans to stop erosion and facilitate cleaning.

➤ Gunit being applied over a nearly vertical embankment. Usually cost of this type of protection is shared by two or more homeowners as it is protection for both upper and lower residences.

➤ Air-placed concrete has been installed on this embankment. Notice how tree wells were installed to plant new trees as well as conserve existing trees. Combinations of Gunit or air-placed concrete and planter wells for shrubs and trees provide pleasing vistas as well as utility.

Water Systems

Differing goals in water conveyance systems can all be achieved speedily and economically by the use of Gunitite or air-placed concrete.

In the coastal plains of the Gulf of Mexico, new canals and existing bayous have been lined to accelerate flow of water into the Gulf, thereby draining rich agricultural crop lands and aiding in mosquito abatement.

Totally different goals have been attained in the High Sierra Nevadas of California, where high velocity channels have been lined to increase flows of water to hydro-electric generating plants.

Deserts of the southwestern United States have been turned into oases by Gunitite lined canals, carrying water to date palm and citrus groves and also to irrigate thousands of acres of truck, forage and fibre crops. Without the impervious Gunitite or concrete lining in these canals, this precious water would soon disappear into the desert sands.

Combination water control flow systems are prevalent in semi-arid areas, where natural water courses have been converted by constructing lined check dams and retention basins, with controlled out-flow structures. With this system, two desirable results are achieved, conservation of water and reduction of storm damage.

Whether your requirements are to conserve water or to get rid of it, Gunitite or air-placed concrete will fill your need.

➤ Air-placed concrete channel lining being installed. Note lines and finish on completed section. This is particularly important where high flow coefficients are required.

Pigmented curing compound being applied to seal surface of newly placed lining.

➤ Twin placing hoses attached to one concrete pump are shown here, rapidly and economically installing a reinforced canal liner of air-placed concrete. Note transit mix truck discharging directly into the pump.

Fresh water is one of the basic necessities of life, and leading world scientists are predicting growing shortages of this precious fluid on a global scale. For this reason, conservation of fresh water has been given top priority by environmentalists all over the world.

Gunite and air-placed concrete construction methods have contributed a great deal and can contribute much more to this program. Lined subterranean reservoirs can produce storage at a lower per gallon cost than practically any other method. Either Gunite or air-placed concrete in lightly reinforced, comparatively thin sections, are ideal methods for this type of storage. Either of these two systems, when properly installed is comparatively waterproof; however, by placing a polyvinyl chloride blanket under the lining, total waterproofing is assured. In this type of reservoir, the soils in which it is located provide

the structural support values and should be adequately tested to insure that they do have the values required. In low humidity areas, water loss by evaporation should be evaluated against the cost of roofing. Structural footings can be easily installed monolithically with the lining operation, to support conventional roof systems; however, inflated balloon-type roofs lend themselves well to this type of installation.

Neither of the two lining methods are limited to this method of construction, and because of their structural values, can be used to construct aboveground vessels of almost unlimited design. Both Gunite and air-placed concrete have been used in the construction of waste water treatment plants, which are being designed at this time - again on a high priority level - to treat industrial and domestic wastes.

Pictured here is an example of a composite lining, consisting of an impervious poly vinyl-chloride sheet, a continuous layer of electrically welded wire fabric and three inches of air-placed concrete. This composite section will provide an absolutely waterproof barrier for an unlimited time.

Under construction is this contoured weir at a retention basin. Gunite was selected for use on this project, where minimal deviations in finished sections were required. Reduction of turbulence in liquid flows conveying abrasive solids is necessary to prevent eroding surfaces of the weir.

stop

Even where rainfall is plentiful, as it is on the northern end of the island of Hawaii, it still must be conserved. In this picture air-placed concrete lining is being installed in this fifty million gallon reservoir.

Water Conservation

Looming over a desert in Iran like a giant wine glass, is this two hundred thousand gallon reservoir, shown here under construction. Above the supporting stem, the storage section is constructed of reinforced Gunite without the use of forms.

Pictured here is the completed reservoir, towering to ninety feet in the air, to provide gravity pressure flow to the distribution system.

Gunite has been used extensively in new construction of sewage treatment plants. In this aerial photo are six structures wherein all vertical walls were constructed completely of reinforced Gunite. Gunite is particularly adaptable in a structure such as the large multi-walled vessel in the background. This vessel called a Clareator, or combination Clarifier and Aerator, has an actual circular tunnel with a top section left open encircling the center area. Excellent flow properties are essential in this tunnel to obtain maximum aeration of the fluids. For this reason the wall and floor sections are all radiused together. To do this particular operation with any other method than Gunite would be prohibitive in cost. In the other structures thinner sections were possible due to the higher compressive values of Gunite, thereby reducing costs considerably, both in the structural walls as well as in the foundations.

The properties of Gunitite make it possible to apply it against any shaped surface and insure its adaptability to the typical fireproofing application. Refineries and petro-chemical plants around the world are using this method to fireproof structural steel members that support piping – and also to fireproof vessel skirts. This method has also been used extensively on high-rise structural steel framing for office and apartment buildings.

While this type of application is essentially fireproofing, two additional values can be obtained: Number one, Gunitite bonds well to clean steel surfaces, and has such high natural density, that it serves as a barrier to rust and corrosion. Number two, because of its structural properties it can be used in a composite design section, which can result in a savings in structural steel.

Any designs for Gunitite fireproofing, should include welded wire reinforcing, which is attached to the steel by welded studs or crimped furring rods. In normal designs the reinforcing should be placed in the center of the section.

To insure a proper bond, the surface to be fireproofed should be cleansed of all millscale, oil, corrosion, paint or other foreign material. Sand blasting is the preferred method for this cleansing.

Another example of Gunitite fireproofing on a tower support structure. Note that the smaller members of the "X" bracing system have also been fireproofed.

In this structure, the circular legs, rectangular shaped "X" bracing and the annular cap beam have all been fireproofed, again demonstrating the adaptability of Gunitite.

A typical structural steel platformer rack with Gunitite fireproofing installed.

Low thermal conductivity material was applied to these liquid propane gas storage tanks to help maintain low internal temperatures.

The importance of insulation has long been a major factor in industry. Today insulation problems have been simplified and costs materially reduced by the use of air-placed castable materials. Materials are now available to meet every need from the super cold of outer space to the inferno of the modern blast furnace. The modern way to contain these extremes is to surround them with speedy, economically placed refractories.

Increased efficiency of operations in refrigerant plants and in catalytic cracking stills in petro-chemical refineries, by the use of proper insulating materials, is a proven fact. The seemingly endless war against air pollution can be brought much nearer to an end by the use of refractories in the lining of stack and breechings. Properly insulated breechings, dampers and stacks will enable plant operators to operate at much higher exhaust temperatures. This eliminates secondary combustion, excessive discharge of unburned fuel particles and corrosion due to condensates coming into contact with the unprotected steel stack. Higher initial firing temperatures mean greater fuel efficiency and will help in the conservation of our fossil fuels.

Most of the castable refractory materials available today have been developed specifically for air-placement, and are composed of manufactured aggregates and cementative binders which are chemically inert. This is an extremely important factor as the dilute acids that can generate from cooling flue gases can be highly corrosive against regular Portland cement and particularly against exposed steel.

Stack and breeching lining today is an almost exclusive province for Gunitite. For this application, the Gunitite method gives the designer great latitude as to lining thicknesses, weight, density, refractory values, corrosion and abrasion resistances. Composite sections combining any of the above values can be easily and economically installed by the use of Gunitite equipment.

Here refractory material is being placed against a steel stack shell. Note the furred electrically welded wire reinforcing curtain which is being incorporated into the center of the lining.

Rising some five hundred feet into the air, this steam plant stack was lined in its entirety with an air-placed castable refractory. All Gunitite equipment except for material-conveying hose was located at ground level.

concussion compression of surrounding materials. When the compressed material attempts its natural release into the excavated space it induces added support values in the arch lining by placing the entire section in compression.

The placement of either Gunitite or air-placed concrete accomplishes the same results when used with tunnel augers or boring machines. Here it can follow even more closely than in a drilled and shot operation - with the material being placed only inches behind the cutting head bits. Design for this type of support system can be varied to meet job conditions as encountered, by adjusting setting times and thickness of lining. Tremendous savings in time and cost of construction have been realized by the use of this modern construction method.

Highway tunnel lining being placed over rock bolt and mesh anchorage system. Lower profile arches may require this type of anchorage while shorter radius arches would not.

One of the many expanding areas of Gunitite usage is in tunnel support systems. While Gunitite has been used for over a half century for tunnel lining to prevent rock falls caused by air slacking, to seal mud seams, to stabilize fractured rock and to consolidate areas surrounding portals, its use as a structural support system in lieu of conventional methods is comparatively new. The major innovative element in this field has been the development of accelerating and hardening agent additives. These patented materials react with Portland cement to produce flash-set Gunitite or air-placed concrete. By varying ratios of additive to cement, controlled set times can be attained. This faculty enables tunnel excavators to install a support system immediately after a blast round is fired, thus taking advantage of

A typical drilled and shot horseshoe shaped water tunnel prior to Guniting.

The same tunnel after Gunitite is complete. Note straight true lines obtained to facilitate flow of domestic water.

Pipe Coatings and Linings

➤ A large diameter pipe is being lined with two inches of Gunitite. The finish will be true line hard steel trowel to furnish flow coefficients as specified.

Coating of pipe normally has one basic aim, however the economical use of Gunitite can achieve several other ends. One of these is in weight coating to attain negative buoyancies for submerged lines. One valuable asset of the Gunitite method is the ability to place high specific gravity as well as normal weight materials. Many submerged lines are installed in saline and brackish waters. Dependent upon the higher weights of this water and the weight and displacement of the coated pipe, it might be that only a heavyweight coating will give the required results.

Size of pipe to be coated has an almost unlimited range when the coating is applied by the Gunitite method.

Pipe to be lined with Gunitite does have a minimum physical range of about thirty six inches in diameter, with no maximum range limitation. Here again the ease of placement of the Gunitite method makes it possible to line pipe sections such as bifurcations, radiused segments, converging diverging sections and other differing shapes that no other method can achieve.

➤ Partially lined steel pipe for water transmission system. Invert section has been lined first and allowed to attain initial set to facilitate rebound removal after balance of lining is installed.

➤ Four inch Gunitite weight coating being applied here to provide negative buoyancy and corrosion resistance for this Big Inch pipe line at a water crossing.

Simulated rock formations — whether for bad guys to hide behind on a western movie set or to form containment areas for wild animals — have long been constructed of Gunitite. Many major zoos and wild animal parks of the world have used light steel framing, paper-backed mesh or expanded metal lath and Gunitite to provide natural appearing background and demising walls in their exhibit areas.

Dry moats where either Gunitite or air-placed concrete — are assigned both structural and architectural values, are designed to produce unimpaired views of exhibit animals, as well as serving as a protective barrier. Since many animals require water as a part of their natural habitat, these water-containing ponds can be easily and economically incorporated into the overall design, and lined with the same Gunitite or air-placed concrete equipment used in other construction of the exhibit area.

Gunitite or air-placed concrete can be sculptured to simulate any type of natural rock formation. Color added to the mix or surface treatment by staining or painting, can be used to further the illusion of natural formations.

Dense, durable surfaces in an animal containment area are particularly desirable to facilitate cleaning. These are natural properties of Gunitite and air-placed concrete, another reason these processes are used so extensively.

Zoo containment area being lined with Gunitite. Front center area is a trowel finished water containment pond. Large natural rocks in the background were preset and locked in position by surrounding Gunitite.

A dry containment moat, with exhibit area surface paving and demising walls all of Gunitite to simulate rock formations.

In this simulated rock formation are concealed chambers for animal feeding and sleeping quarters.

Swimming Pools

Portable Guniting equipment has made the private backyard swimming pool a reality instead of a dream to average homeowners all over the world. A trained crew with a modern portable Guniting rig can complete the lining in as many as three average size pools each day. This has made it economically possible for thousands of home owners to enjoy their own pool.

The flexibility of the Guniting operation allows total freedom in building pools that are not stereotyped copies of each other. Whether a new pool owner wants a pool shaped like a grand piano or a four leaf clover or any other imaginable shape, it can be constructed with little if any additional cost. Location sites for pools can be almost as varied as shapes. Many pools are being constructed on top of existing hotels, in existing basements, half in and half outside of homes, hotels and motels.

Old vertical walled pools of concrete, steel or masonry can be modernized easily and economically by placing a new Guniting lining inside the old shell. In this new lining can be incorporated new plumbing runs, subterranean lights, rope anchors, new steps and a new shape with easy to maintain radiused corners and the clean new look of a pool of today.

These pool pictures will give the reader a very small cross-section of the range of sizes and shapes available to private and commercial owners. In the picture containing two pools, the small one is therapeutic — with higher temperature and induced water turbulence — for that relaxing sensation of a water massage.

It has never been the policy of the Gunitite Contractors Association to refer to products or equipment by name, however, this new product is so revolutionary and timely that the Association has altered its policy in order to report on this patented product.

Produced by CS&M Incorporated, of Chino, California, the "W" Panel system consists of three-dimensional space frames composed of two parallel sheets of electric-welded wire fabric, using welded diagonal wire to create the spaced frames. The diagonals welded to each of the longitudinal wires of the sheets act as truss members to give the panels rigidity. Centered in the two-inch space between the parallel sheets is a cast-in-place plastic foam which bonds to and is locked in place by the "W" conformation of the diagonal wires. This composite shape reveals approximately one half inch space between the foam core and the parallel electrically-welded wire sheets on each face. The space thus created offers an ideal application surface for either Gunitite or air-placed concrete.

These "W" Panels are light in weight (approximately twenty-five pounds per four foot by eight foot section), and can be easily installed manually by vertical abutting and using "X" ties of galvanized tie wire at joint seams.

The initial economy, ease of installation plus values such as being waterproof, fireproof, decayproof, verminproof, and having excellent insulating properties makes this new building component as near to the ideal system as the science of construction design has been able to achieve.

Design data on this completely researched and developed system is available from the manufacturer.

A partially encased demonstration segment of a "W" Panel, showing the parallel curtains of welded wire fabric, the foamed plastic center core and the diagonally welded wire truss cord members which is really the secret ingredient of this successful building component.

Gunitite is being applied to one side of a "W" Panel on a building under construction. It should be noted that the Gunitite section applied from either side of the panel can be extended to any desired thickness to attain any desired structural value.

A completed two story condominium dual unit, where all vertical walls, second story floor system and roof structure were constructed by structural encasement of "W" Panels. Except for furnishings, this insulated structure is totally fireproof, thereby greatly reducing fire insurance premiums.

For centuries masons and plasterers have been adding wood shavings, horse hair or cereal grain straw to their sun dried mud (adobe) bricks and to their lime and sand mortar or plaster to stabilize and - to a degree - reinforce these ancient building materials. Modern technology has updated this basic idea by the introduction of various modern fibres to be incorporated in Portland cement mortar and concrete. Fibres of steel, glass, asbestos, and various synthetics have been added to mortar and concrete mixes in volumetric amounts varying from one and one half percent to six percent with some astounding physical property results.

With fibres completely supplanting ordinary reinforcing, compressive values have been doubled and flexural strengths have been increased up to two and one half times the values of regular reinforced concrete. The most impressive finding is that even after failure of fibre concrete, a structural member continues to function to a degree until total fracture and severance of every bridging fibre unit is accomplished.

It has been proven that fibre concrete sections of one-half the standard thickness for floor slabs, driveways and side walks, will provide the same service. Due to its greatly increased flexural values, fibre concrete and mortar are being recommended for applications in blast furnaces, catalytic cracking stills, docking facilities and wherever intense shock loading is prevalent.

Fibre concrete is also showing amazing resistance to corrosive attack by salt water or other corrosive agents that attack conventionally reinforced concrete. Spalling of surfaces, caused by heavy wheel loading is practically eliminated by thin fibre concrete overlays.

While the fibres for Gunitite and air-placed concrete add considerably to material costs, they may be more than offset by the elimination of reinforcing and by reduction of section, plus the added values inherent in this recently upgraded construction product.

The severely fractured test beam, shown above, is still in one piece due to the steel fibre reinforcing. Two and one-half percent by weight of this particular coated steel fibre, will yield sixty-five thousand fibres per cubic foot of concrete.

Shown above in a size comparison photo with a Jefferson Nickel, is one type of coated steel fibre.

Fibre reinforced Gunitite being applied to this fractured rock face. An average of one inch of this high value material proved to be adequate to prevent further rock falls.

Special Marine Projects

Certain physical properties of Gunitite give it a high resistance to salt water penetration. The heavy concentration of mineral salts contained in sea water combine with surface layers of partially hydrated cement particles to create a special gel that tends to close off the surface pores of the Gunitite and create an almost impermeable barrier. The reason for this is that Gunitite in situ has more than double the cement to aggregate ratio of normal concrete.

As an example of this surface sealant capability, a Gunitite barge constructed and launched in 1914, was ballasted with fresh water. Over forty years later the original fresh water ballast was still found to be potable.

Because of this proven value, Gunitite is much in demand for construction of marina floats, spacer camels, boat hulls, wooden

pile encasement and restoration of unseaworthy steel and wooden barges. Gunitite applied directly over marine borer-infested wooden piling seals out oxygen so effectively that these destroyers are suffocated and further deterioration of the piling is precluded. The Gunitite process lends itself so well to streamlined hull configurations, that it has been used extensively in ferro cement, mesh and fibre-reinforced hulls for pleasure crafts, house boats and catamarans.

➤ Pictured above Gunitite and mesh are being applied over a steel barge that had rusted so badly that it was no longer seaworthy. The entire surface of this barge will be encased in Gunitite and its usefulness restored and prolonged indefinitely.

Shown here is two inch Gunitite being applied over mesh wrapped wooden piling. This type of protection is applied as low as lowest possible tide levels permit and extends above the maximum tide and splash levels.

➤ This beautifully shaped vessel is constructed entirely of mesh-reinforced Gunitite and is ready for internal fitting, equipment and superstructure.

This system of wood piling encasement, though more expensive than that pictured on page 34, has one major value that the other system does not have. The protection afforded can be extended up to five or more feet into the harbor bedding material. In order to accomplish this, a mesh-reinforced annular shell is placed over a furred building felt form. This shell is constructed in approximate five foot vertical lifts, which mesh and Gunitite form into one continuous tube. As each lift is added, previous lifts are lowered, and by jetting will settle into the harbor bottom. After desired penetration into the harbor bottom is achieved, cement grout is tremied into the space between the shell and piling. Water which has infiltrated into the space will be displaced by the grout.

Shown above is Gunitite being applied to a five foot lift. Note the cable and sheave device to the left of the pile, for lowering completed sections into final position.

A composite cross section plan view showing details of furred form and completed section.

Completely protected piling, after Gunitite and grouting operations have been completed.

Completely installed and equipped, these new floating Gunitite docks are ready to accept their first pleasure craft for safe mooring.

To construct this unique residence, was a simple matter of inflating a balloon, adding a layer of welded wire mesh and applying the Gunite. This is an obvious over-simplification, however that is the basic procedure used.

The preceding pages of this brochure have indicated specific types of applications. Lest you be led to believe that Gunite applications and air-placed concrete are limited to these specific areas, a small sampling of the many other places where these methods can be used is shown below.

It is doubtful if any other construction process has a wider range of application than the methods described herein.

For expert advice with your next project, no matter how conventional or how far out it might appear to be, feel free to contact the Gunite Contractors Association.

Conversion of open irrigation canals to closed conduits for conveying domestic water to new houses that have replaced orchards and fields, is simple and easy with Gunite. Arch supports of reinforcing rod carry the paper-backed mesh which serves as combined form and reinforcing for the Gunite cover. Once the Gunite has been placed and attained its strength, the steel supports and the paper backing the mesh could be removed with no effect on the arch.

This bank of cement storage silos were Gunited without the use of forms. The reinforcing steel cage was erected in its entirety and tack-welded at strategic points to insure alignment and rigidity. Steel wire fly screen was then wrapped tightly around the exterior of the reinforcing and the inner section of the wall was placed. The fly screen was stripped leaving a rough textured surface ideal to bond the exterior Gunite against.

The sixty degree slopes of this world's largest coal storage bunker have been lined with six inch thick Gunite. The extreme slope angle is necessary to allow the coal to slide downward to the loading tipples. To have attempted this lining in conventional cast in place would have been prohibitively expensive.

The unique three dimensional sign letters were constructed of Gunite applied over a shaped-lathe matrix to give special identity to this college campus. The concept for this novel idea originated with one of the engineering students enrolled at this university.

Completed Gunite steps which may be used as is for seating, or conventional stadium-type seats can be installed later.

Doubling the life expectancy of petroleum storage tanks has been accomplished by the addition of two inches of wire mesh reinforced Gunite. Shown above is Gunite being applied over a sand-blasted steel tank wall to which welded furring and welded wire mesh had been attached.

APPENDIX B

CONCRETE REINFORCEMENT MATERIALS; STRENGTH AND COSTS

ITEM	DESCRIPTION	UNIT COST	STRENGTH	STRENGTH COST
WOVEN FABRICS				
1	FRC 1027 fibreglass fabric	24¢/ft ²	3.88K	6.19¢/K
2	FRC 1037 fibreglass fabric	25.5	7.76	3.29
3	TYLER 3-035* grade 100 steel wire cloth	59	6.93	8.52
4	TYLER 4-035 grade 100 steel wire cloth	77	9.24	8.33
5	TYLER 5-035 grade 100 steel wire cloth	94	11.55	8.14
6	TYLER 7-054 grade 100 steel wire cloth	299	38.48	7.77
STEEL REINFORCEMENT				
1	#3 grade 60 bar	9.5¢/ft	6.63K	1.43¢/K
2	#4 grade 60 bar	16.9	11.78	1.43
3	0.375 grade 270-7 wire strand	20.3	23.0	0.9
4	0.500 grade 270-7 wire strand	36.4	41.5	0.9
5	0.250 6 x 19 wire rope	47.5	5.8	8.2
6	0.375 6 x 19 wire rope	54.9	12.8	4.3
7	0.500 6 x 19 wire rope	65.8	23.6	2.8

* 3 wires per inch, 0.035 inch diameter wire

TABLE B.1

Material Strength and Costs

left to right:

- (1) Tyler 7-054 woven steel wire fabric
- (2) Non-woven polyethylene onion bag
- (3) FRC 1027 woven fibreglass fabric
- (4) FRC 1037 woven fibreglass fabric

PLATE B.1

Sample Fabrics

APPENDIX C

SHEAR TEST APPARATUS; DRAWINGS, TAKEOFFS AND COSTS

ITEM	QTY	DESCRIPTION	TOTAL COST
1	8	arm plates	\$304.00
2	8	corner plates	108.00
3	8	high strength bolts $1\frac{1}{2}$ " x 8" c/w nuts, washers	178.00
4	26	high strength bolts $\frac{1}{2}$ " x $5\frac{1}{4}$ " c/w nuts, washers	62.00
		total apparatus cost	\$652.00

TABLE C.1
Apparatus Costs

FIGURE C.2

Test Apparatus Component Drawings

APPENDIX D

PLATE FORMING; DRAWINGS, TAKEOFFS AND COSTS

ITEM	QTY	DESCRIPTION	UNIT COST	TOTAL COST
1	20	sheets 4' x 8' x $\frac{3}{4}$ " plywood RBS	\$17.95	\$359.00
2	9	sheets 4' x 8' x $\frac{1}{4}$ " plywood	8.95	79.65
3	140	feet 2 x 4 spruce	0.25	35.00
4	700	feet $\frac{1}{4}$ " 6 x 19 steel cable	0.43	301.00
5	75	cable clips	0.37	27.75
6	20	rods $\frac{3}{8}$ " x 4' wood dowel	0.52	10.40
7	67	ft ² FRC 1027 fibreglass fabric	0.24	16.00
8	67	ft ² FRC 1037 fibreglass fabric	0.255	17.00
9	56	ft ² 7-054 Tyler wire cloth	2.99	191.36
10	15	sheets 4' x 8' x 1" styrofoam	3.25	48.75
11	4	castors	16.00	64.00
12		screws, nails, solvent, etc.		60.00
13		tax, shipping, etc.		30.00
		sub total		\$1239.91
14		shotcrete operation incl. crew		600.00
15		shotcrete materials		161.29
		sub total		\$761.29
		Total		\$2001.20

TABLE D.1

Formwork Costs

FIGURE D.1

Plate Formwork Drawings

PLATE D.1

Close-up Showing Fibreglass Fabric/
Steel Grid Composite Reinforcement

PLATE D.2

Plate Curing Arrangement, Showing
Intermediate Styrofoam Padding

APPENDIX E
PRISM TEST DATA

SPECIMEN No	BASE (in.)	WIDTH (in.)	HEIGHT (in.)	Pu (k)	f _c (ksi)
1	1.98	2.07	4.0	8.25	2.01
2	1.97	2.06	4.0	8.40	2.07
3	1.97	2.07	4.0	8.20	2.01
4	1.97	2.08	4.0	8.40	2.05
5	1.95	2.00	4.0	6.50	1.67
6	1.97	2.09	4.0	7.70	1.87
7	1.96	2.08	4.0	8.70	2.13
8	1.98	2.08	4.0	9.65	2.34
9	1.98	2.04	4.0	7.55	1.87
10	1.95	2.10	4.0	9.25	2.26
11	1.97	2.06	4.0	7.90	1.95
12					
13					
14					
15					

AVERAGE f_c = 2.02 ksi

STD DEVIATION OF f_c = 0.59 ksi

TABLE E.1

14-DAY SHOTCRETE PRISM TEST
(AS PER ASTM C39-72)

SPECIMEN No	BASE (in.)	WIDTH (in.)	HEIGHT (in.)	P _u (k)	f _c (ksi)
1	1.87	2.10	4.0	10.75	2.74
2	1.98	1.99	4.0	8.20	2.08
3	1.98	2.02	4.0	8.70	2.18
5	1.97	1.98	4.0	8.45	2.17
6	1.96	2.05	4.0	8.45	2.10
7	1.97	2.08	4.0	9.45	2.31
8	1.97	2.06	4.0	10.45	2.58
9	1.97	2.04	4.0	10.10	2.51
10					
11					
12					
13					
14					
15					

AVERAGE f_c = 2.32 ksi

STD DEVIATION OF f_c = 0.67 ksi

TABLE E.2
21-DAY SHOTCRETE PRISM TEST
(AS PER ASTM C39-72)

SPECIMEN No	BASE (in.)	WIDTH (in.)	HEIGHT (in.)	Pu (k)	f _c (ksi)
1	1.96	2.07	4.0	9.98	2.46
2	1.97	2.06	4.0	9.50	2.34
3	1.99	2.02	4.0	10.21	2.54
4	2.00	2.05	4.0	9.14	2.23
5	1.98	2.03	4.0	9.05	2.76
6	1.98	1.99	4.0	11.09	2.89
7	1.99	2.08	4.0	10.47	2.53
8	1.97	2.06	4.0	8.85	2.18
9	1.98	2.05	4.0	9.82	2.42
10	1.96	2.02	4.0	9.94	2.51
11	1.97	2.00	4.0	10.52	2.67
12	1.97	2.01	4.0	10.73	2.71
13					
14					
15					

AVERAGE f_c = 2.52 ksi
 STD DEVIATION OF f_c = 0.71 ksi

TABLE E.3
 28-DAY SHOTCRETE PRISM TEST
 (AS PER ASTM C39-72)

SPECIMEN No	BASE (in.)	WIDTH (in.)	HEIGHT (in.)	Pu (k)	f _c (ksi)
1	1.94	2.00	4.0	9.05	2.33
2	1.96	2.04	4.0	11.55	2.89
3	1.97	1.94	4.0	10.85	2.84
4	1.98	2.00	4.0	11.45	2.89
5	1.99	1.99	4.0	12.30	3.11
6	2.00	1.96	4.0	10.70	2.73
7	1.96	1.98	4.0	10.55	2.72
8	1.96	2.08	4.0	11.00	2.70
9	1.97	1.99	4.0	12.35	3.15
10	1.96	1.97	4.0	7.85	2.03
11	1.94	2.06	4.0	10.70	2.68
12	1.97	1.96	4.0	10.00	2.59
13					
14					
15					

AVERAGE f_c = 2.72 ksi

STD DEVIATION OF f_c = 0.79 ksi

TABLE E.4

42-DAY SHOTCRETE PRISM TEST

(AS PER ASTM C39-72)

SPECIMEN No	BASE (in.)	WIDTH (in.)	HEIGHT (in.)	Pu (k)	f _c (ksi)
1	1.95	1.96	4.0	11.30	2.96
2	1.97	1.99	4.0	12.00	3.06
3	1.95	1.96	4.0	11.15	2.92
4	1.99	1.98	4.0	12.60	3.20
5	1.98	1.98	4.0	10.00	2.55
6	1.99	1.99	4.0	11.60	2.93
7	1.96	1.96	4.0	12.05	3.14
8	1.97	1.94	4.0	13.50	3.53
9	1.96	1.95	4.0	12.70	3.32
10	1.97	1.96	4.0	11.10	2.87
11	1.97	1.93	4.0	10.35	2.72
12	1.98	1.97	4.0	10.75	2.76
13					
14					
15					

AVERAGE f_c = 3.00 ksi

STD DEVIATION OF f_c = 0.90 ksi

TABLE E.5
56-DAY SHOTCRETE PRISM TEST
(AS PER ASTM C39-72)

APPENDIX F
PLATE TEST DATA

Besides total vertical deflection, the plates were also instrumented for local deflections and deflection uniformity (figure F.1). The local deflection measurements in the uncracked portion of the tests were of the same magnitude as the reading error of the 4-inch Demec gauge used, and therefore the results were unreliable. In the cracked portion of the tests, the local deflection measurements depended directly on the random appearance of vertical cracks, and therefore any comparison between left, right and center measurements was not reasonable. For this experimental program, only the total vertical deflection measurements can be considered useful.

FIGURE F.1

Test Instrumentation

- (1) Series A; plates not reinforced

$$P_u(E) = 0.11 \times 3.0 \text{ ksi} \times 2 \text{ in.} \times 32 \text{ in.} \times \sqrt{2} = 29.9 \text{ k}$$

- (2) Series B; plates c/w cable grid reinforcement

$$P_u(E) = 7 \times 5.8 \text{ k} \times \sqrt{2} = 57.4 \text{ k}$$

- (3) Series C; plates c/w fibreglass fabric/cable grid composite reinforcement

$$P_u(E) = (7 \times 5.8 \text{ k} + 32 \text{ in.} \times 0.16 \text{ k/in.}) \times \sqrt{2} = 64.8 \text{ k}$$

- (4) Series D; plates c/w wire fabric/cable grid composite reinforcement

$$P_u(E) = (7 \times 5.8 \text{ k} + 32 \text{ in.} \times 1.6 \text{ k/in.}) \times \sqrt{2} = 129.8 \text{ k}$$

FIGURE F.2

Calculations for Expected Failure Loads $P_u(E)$
(Information and Data Sources - figure 3.2.2,
paragraph 3.3.3, appendices B and E)

P (k)	DV3 (in.)	CRACKS	DV4 (in.)	CRACKS	DV5 (in.)	CRACKS	DV6 (in.)	CRACKS	DVavg (in.)
0	0		0		0		0		0
3	0.013		0.067		0.026		0.037		0.037
6	0.026		0.134		0.052		0.064		0.064
6.7				fail					
9	0.072				0.082		0.105		
9.6									fail
10.0						fail		fail	
11.7		fail							

TABLE F.1

Series A Plate Tests
Unreinforced Shotcrete

P (k)	DV7 (in.)	CRACKS	DV8 (in.)	CRACKS	DV9 (in.)	CRACKS	DV10 (in.)	CRACKS	DVavg (in.)
0	0		0		0		0		0
3	0.037		0.045	1	0.021		0.033		0.034
6	0.074	1,2	0.090	2	0.042	1	0.066		0.068
9	0.110		0.126	3,4	0.063		0.132	1,2	0.108
12	0.150	3,4	0.163		0.094		0.184	3,4	0.148
15	0.199	5	0.249	5	0.120	2	0.225		0.198
18	0.241		0.287		0.183	3	0.265	5,6	0.244
21	0.286	6,7	0.323		0.226		0.339	7	0.294
24	0.326		0.361		0.316	4,5,6	0.383	8	0.347
27	0.362	8	0.427	6	0.369	7	0.433		0.400
30	0.421	9	0.470		0.401		0.486	9	0.446
33	0.491		0.549	7	0.458		0.518		0.504
36	0.554		0.614		0.502	8,9	0.556		0.557
37.1			0.677	fail					
38.4	0.634	fail							
39					0.546		0.609		
41.0									0.698
42					0.591	10			
43.8							0.769	fail	
44.8					0.712	fail			

TABLE F.2

Series B Plate Tests
Steel Grid Reinforced Shotcrete

P (k)	DV11 (in.)	CRACKS	DV12 (in.)	CRACKS	DV13 (in.)	CRACKS	DV14 (in.)	CRACKS	DVavg (in.)
0	0		0		0	1	0		0
3	0.032		0.032		0.031		0.034		0.032
6	0.064		0.064	1	0.062		0.068	1	0.064
9	0.088		0.090		0.092		0.102	2,3	0.093
12	0.120	1	0.138	2,3,4	0.129		0.121		0.127
15	0.176	2,3	0.194	5	0.182	2	0.155	4,5	0.177
18	0.233		0.230		0.209		0.216	6	0.222
21	0.290	5,6,7	0.276	6,7,8	0.295	3,4	0.257	7	0.280
24	0.347	8	0.347	9	0.356	5	0.317	8,9	0.342
27	0.414	9	0.391		0.421	6	0.356		0.396
30	0.449		0.480	10,11	0.468		0.394	10	0.448
33	0.491		0.513		0.516		0.440		0.490
36	0.523		0.564		0.562	7	0.496		0.536
39	0.609		0.626	12	0.612	8	0.567		0.604
42	0.685		0.649		0.660		0.620		0.654
44.3					0.766	fail			
45	0.722		0.712				0.674		0.716
45.9							0.740	fail	
46.8									0.809
47.7	0.832	fail							
49.4			0.899	fail					

TABLE F.3

Series C Plate Tests
Fibreglass Fabric/Steel Grid
Composite Reinforced Shotcrete

P (k)	DV15 (in.)	CRACKS	DV16 (in.)	CRACKS	DV17 (in.)	CRACKS	DV18 (in.)	CRACKS	DVavg (in.)
0	0	1,2	0	1	0		0		0
5	0.043		0.032		0.059		0.059	1,2	0.048
10	0.086		0.064	2,3	0.118	2	0.118	3,4	0.097
15	0.128	3,4	0.100	4	0.166		0.167	5,6	0.140
20	0.182	5,6	0.146	5,6	0.217	3,4,5,6	0.226	7,8	0.193
25	0.247	7,8,9	0.214	7,8	0.286		0.301	9,10	0.262
30	0.316		0.293	9	0.345	7,8	0.373		0.332
35	0.375	10,11	0.376	10	0.400	9	0.444		0.399
40	0.432	12	0.434		0.450	10,11	0.491		0.452
45	0.479		0.499	11	0.498	12	0.543		0.505
50	0.544		0.629		0.546		0.607		0.582
53.0			0.743	fail					
55	0.614				0.593		0.681		
56.5							0.759	fail	
58.1									0.751
60	0.742				0.659				
60.8					0.705	fail			
62.0	0.797	fail							

TABLE F.4

Series D Plate Tests
Steel Wire Fabric/Steel Grid
Composite Reinforced Shotcrete

PLATE F.1

Close-up Showing Pullout of
Main Reinforcement

PLATE F.2

Spalling of Fibreglass Reinforced
Plate at Advanced Deflection

PLATE F.3

Cleavage of Wire Fabric Reinforced
Plate at Advanced Deflection

APPENDIX G

BUILDING CONSTRUCTION UNIT COST QUOTATIONS, JUNE, 1979

ITEM	DESCRIPTION	BUDGET	QUOTATION	SOURCE
		MATERIAL	LABOUR	
1	light industrial bldg. c/w long span roof	\$25-30/ft ²	incl.	M. Allen, Adjeleian & Associates, Ottawa
2	light industrial bldg. fdn. incl. slab	\$ 2/ft ²	incl.	M. Allen
3	rolled structural steel	\$950-1000 /T	incl.	C. Bellinger, Adjeleian & Assoc. Ottawa
4	reinf. masonry wall, 8 x 16 block	\$ 3/block	incl.	C. Bellinger
5	vert. steel cladding colour coated 4 High	\$ 2/ft ²	incl.	J. Daler, Westeel & Rosco, Ottawa
6	steel roof deck T-30-6 as diaphragm	\$ 1/ft ²	incl.	J. Daler
7	concrete forming: walls (conc. bldg.) shells (conc. bldg.) shells (comp. bldg.) shells (scaffolding only)		\$3/ft ² \$4/ft ² \$4.50/ft ² \$2/ft ²	C. Gnani, Right Forming, Ottawa (note: quotations are for surface contact area)
8	poured concrete	\$42/YD ³	\$10/YD	C. Gnani
9	concrete finishing: machine manual		\$0.20/ft ² \$1/ft ²	C. Gnani
10	#2-#7 grade 60 bar #8-#12 grade 60 bar	\$500/T \$500/T	\$200/T \$150/T	C. Gnani
11	ungrouted prestressing strand	70¢/lb	30¢/lb	Mr. Bradley, Cdn. Lift-Slab Co.Ltd., Mississauga, Ont.
12	FRC 1037 fibreglass fabric	25.5¢/ft ²	25¢/ft ² (est.)	M. Denhoed, FRC Composites, Toronto

TABLE G.1

Unit Cost Quotations

ITEM	DESCRIPTION	BUDGET	QUOTATION	SOURCE
		MATERIAL	LABOUR	
13	Tyler 7-054 woven steel wire fabric	\$2.99/ft ²		C.E. Tyler, St. Catharines, Ontario
14	shotcrete	\$42/YD	\$75/YD ³	A.B. Benson, Benson Pools, Ottawa
15	2 man articulated truck mounted crane		\$60/hr	Eugene Tree Removal, Ottawa

TABLE G.1 (cont'd)
Unit Cost Quotations

APPENDIX H

HYPAR SHELL DESIGN CALCULATIONS
(SERIES C CONCRETE BUILDINGS)

(1) Shells

design loads: snow 60 psf (no reduction because of roof valleys)

shell 32 psf

roofing 5 psf

M & E 5 psf

edge beams 6 psf (half only)

$$\begin{aligned}\text{total design load: } w &= 1.7 \times 60 \text{ psf} + 1.4 \times 48 \text{ psf} \\ &= 169 \text{ psf}\end{aligned}$$

$$\begin{aligned}\text{shear stress: } v &= \frac{0.169 \text{ ksf} \times 50' \times 50'}{2 \times 10' \times 2.5' \times 12' / ' } \\ &= 0.704 \text{ ksi}\end{aligned}$$

from the failure model of figure 3.2.2, $v_{st} = \phi P_n$

for #4 grade 60 bar, $P = 11.8 \text{ k}$

for FRC 1037 fibreglass fabric, $P = 0.32 \text{ K/in.} \times s$

$$\begin{aligned}\text{(a) simple reinforcement grid: } s &= \frac{0.9 \times 11.8 \text{ k}}{0.7 \text{ ksi} \times 2.5 \text{ in.}} \\ &= 6.1 \text{ in.}\end{aligned}$$

Specify $s = 6 \text{ in.}$

(b) composite reinforcement grid:

$$\begin{aligned}0.7 \text{ ksi} \times 5 \times 2.5 \text{ in.} &= 0.9 (11.8 \text{ k} + 0.32 \text{ k/in} \times s) \\ s &= 7.3 \text{ in.}\end{aligned}$$

Specify $s = 7 \text{ in.}$

(2) Edge Beams

(a) perimeter edge beams: $P = 0.704 \text{ ksi} \times 2.5 \text{ in.} \times 51 \text{ ft} \times 12 \text{ in/ft}$
 $= 1077 \text{ k}$

$$1077 \text{ k} = 0.7 (0.04 A_g \times 60 \text{ ksi} + 0.85 \times 0.96 A_g \times 4 \text{ ksi})$$

$$A_g = 272 \text{ in}^2$$

$$= 16.5 \text{ in} \times 16.5 \text{ in}$$

Specify 18 in. x 18 in.

$$A_{st} = 0.04 \times 272 \text{ in}^2$$

$$= 10.9 \text{ in}^2$$

Specify 8 # 11's

reduce to $0.01 \times 18 \text{ in.} \times 18 \text{ in.} = 3.24 \text{ in}^2$ (8 # 6's) at halfway

(b) central edge beams: $P = 2 \times 0.704 \text{ ksi} \times 2.5 \text{ in.} \times 50 \text{ ft.} \times 12 \text{ in/ft}$
 $= 2112 \text{ k}$

$$2112 \text{ k} = 0.7 (0.04 A_g \times 60 \text{ ksi} + 0.85 \times 0.96 A_g \times 4 \text{ ksi})$$

$$A_g = 530 \text{ in}^2$$

$$= 23.0 \text{ in} \times 23.0 \text{ in}$$

Specify 24 in x 24 in

$$A_{st} = 0.04 \times 530$$

$$= 21.2 \text{ in}^2$$

Specify 12 # 14's

reduce to $0.01 \times 24 \text{ in} \times 24 \text{ in} = 5.8 \text{ in}^2$ (12 #7's) at halfway

(3) Thrust (from service loads only)

(a) perimeter thrust $T = \frac{50}{51} \times 689 = 675 \text{ k}$

(b) central thrust $T = 2 \times 675 \text{ k} + 521 \text{ k from main central beam}$
 $= 1871 \text{ k}$

(4) Load to columns

(a) at corner columns: $C = 2 \times \frac{10}{51} \times 1077 \text{ k} = 422 \text{ k}$

(b) at central columns: $C = 2 \times 422 \text{ k} = 844 \text{ k}$

(c) total: $4 \times 422 \text{ k} + 2 \times 844 \text{ k} = 3380 \text{ k}$

(check: $0.169 \text{ ksf design load} \times 20,000 \text{ ft}^2 = 3380 \text{ k}$)

APPENDIX I

RIGID CABLE NET CONCEPT

Paragraph 4.4.3 describes one idea for completely eliminating the temporary formwork from the proposed composite reinforced shotcrete shell construction system. Figure I.1 shows how the "rigid cable net" concept might appear if it was applied to the same industrial building considered in Chapter 4. Table I.1 shows that the further \$2 per square foot saving from the removal of all temporary scaffolding lowers the total building to \$32 per square foot, only 7% higher than the steel building.

A valid design and costing procedure was not used for either figure I.1 or table I.1. However, the potential is such that a proper design and optimization study is fully justified.

Stage 1 - Walls and Central Arch

Stage 2 - Cable Net and Fibreglass Fabric in Place

Stage 3 - Shotcrete Top and Bottom

FIGURE I.1

Modified Shell Construction System
"Rigid Cable Net" Concept

ITEM	DESCRIPTION	QTY	UNIT COST	ITEM COST	% OF TOTAL
1	foundation c/w slab on grade	20,000 ft ²	\$ 2	\$40,000	6%
2	structure:				
	(a) concrete walls, pre-stressed edge beams, central arch (est. only)			\$125,000	
	(b) 0.375 prestressing strand @ 12", 100 pcs @ 250', 200 pcs @ 125', 0.29 plf	15 K	\$ 1	\$15,000	
	(c) shotcreting, incl. crane and fiberglass fabric (est. only)			\$40,000	
	sub-total			\$180,000	28%
3	architectural, mechanical, electrical			\$416,000	66%
	total	20,000 ft ²	\$ 32	\$636,000	100%

TABLE I.1

Rigid Cable Net Building
 Modified Shell Construction System c/w Shotcrete
 Takeoffs and Costs
 (refer to figure I.1)

END

12 · 12 · 80

FIN