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New Developments and Management

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Contents

Foreword

This Record contains information on high early strength concrete, flowable fly ash, flowable mortars, controlled low-strength materials, performance of rigid concrete, quality assurance, concrete construction provisions, concrete construction equipment and procedures, and the application of robotics. It should be of interest to state and local materials, construction, and maintenance engineers as well as contractors and material producers.

Grove notes that the use of special Type III cement and insulating blankets to accelerate the cure in Fast Track concrete does not come without cost. He developed a test program to determine the benefit derived from the use of insulating blankets to accelerate strength gain in mixes using Type I cement. The results showed that insulating blankets significantly improve the early strength gain of the concrete.

Krell identifies many of the features that distinguish flowable fly ash from other backfill materials, both conventional and controlled low-strength materials. Rose, Hope, and Ip examined variables influencing the 28-day compressive strength and 50-cycle salt scaling loss of concretes made with different Type 10 cements. They concluded that cements with a high alkali content yielded concretes with lower compressive strengths and that fineness is related to low salt scaling loss. Whiting and Dziedzic studied the chloride permeability of three different types of rigid concrete bridge deck overlays: latex-modified concrete; superplasticized dense concrete; and condensed silica fume concrete. Results of the study indicate that condensed silica fume concrete is the most impermeable to chloride ions. Buss discusses the development and use of flowable mortar for different applications dealing with pipe culverts and bridges. Fox discusses the use of coarse aggregates in controlled low-strength materials. Lynn describes the development of an improved contract quality assurance program within the Virginia Department of Transportation. Charonnat, Gallenne, and Deligne describe their success in France with a floating table for placing concrete pavements. The floating table forms the placed concrete at the rear of the paving tractor, allowing paving widths to exceed the tracks of the paving tractor.

Hinze and Couey examined the provisions of the construction contract documents of all state highway agencies. Focusing on time and weather provisions, they found a wide variation in practices among the highway agencies. Najafi and Naik discuss the use of robotics in transportation engineering, particularly for harsh or hazardous tasks. Jorge and Herbsman developed a model method for determining construction equipment rental rates for the state of Florida. The method could replace the negotiation process as well as the use of nationwide guidelines for average rates currently being used for equipment force account work. Karshenas presents graphical solutions of the queuing method to determine a reliable forecast of multiloader-truck fleet production. Fwa discusses a procedure to assess the weathering effects of a wet tropical climate on the wear resistance of concrete pavements.

Blanket Curing to Promote Early Strength Concrete

JAMES D. GROVE

Fast Track concrete has proven to be successful in obtaining high early strengths. This benefit does not come without cost. Special Type III cement and insulating blankets to accelerate the cure add to its expense when compared to conventional paving. This research was intended to determine the benefit derived from the use of insulating blankets to accelerate strength gain in three concrete mixes using Type I cement. The goal was to determine mixes and curing procedures that would result in a range of opening times. This determination would allow the most economical design for a particular project by tailoring it to a specific time restraint. Three mixes of various cement content were tested in the field. Flexural beams were cast for each mix and tested at various ages. Two test sections were placed for each mix, one section being cured with the addition of insulating blankets and the other being cured with only conventional curing compound. Iowa Department of Transportation specifications require 500 psi flexural strength before a pavement can be opened to traffic. Concrete with Fast Track proportions (nominal 7 1/2 bag), Type I cement, and insulating blankets reached that strength in approximately 36 hr, a standard mix (nominal 6 1/2 bag) using the blankets in approximately 48 hr, and the Fast Track proportions with Type I cement without blankets in about 60 hr. The results showed a significant improvement in early strength gain with the use of insulating blankets.

In 1986, 1987, and 1988 several Portland cement concrete (PCC) paving projects were constructed using the Fast Track mix and procedures developed in Iowa. Figure 1 represents a compilation of test data obtained from six Fast Track projects constructed in Iowa during these years. Many dissimilarities existed among them: thickness of pavement, temperature during construction, brands of cement, type of mixing, and type of transport vehicles. However, these data produce a distinct locus of points in the first 24 hr. All achieved 400 psi flexural strength in 12 hours or less. These and other projects have established that Fast Track can produce the high early strengths for which it was designed.

Two aspects of Fast Track that normally are not seen in conventional paving include the use of a special Type III cement and the placement of insulating blankets over the finished pavement. (Note: Special Type III cement is a modification of AASHTO M85 to include a compression strength of hydraulic mortar, using 2-in.-cube specimens, of at least 1,300 psi at 12 hr, when tested in accordance with ASTM C109. Further reference to Type III hereafter means Special Type III.) The Type III cement is used to accelerate the hydration process, and the blankets are used to trap the heat from that process. The Type III cement is not a widely used product. Most ready-mix plants do not keep this cement in inventory and many do not have the storage facilities to accommodate more than one cement. The insulating blankets are a costly item in terms of initial cost and the labor-intensive procedure for installation. Both add to the expense of the fast track procedure. If either or both could be eliminated while still achieving an acceptable time of opening, significant savings could be realized.

Not all projects need to obtain opening strengths in less than 24 hr. Some roadways, however, may need to be open to traffic in less than the 5 to 14 days that are required for conventional paving. This research was intended to determine the strength gain over time for various mixes, each being cured with and without insulation blankets. The goal was to determine what effect the type of mix and insulation had on early strength gain. This information will be helpful in determining the most economical design for a project with a given timetable for opening the facility to traffic.

OBJECTIVES

The objective of this research was to establish a range of alternative designs, in terms of various mixes and curing methods, by using Type I cement and either conventional curing or enhanced conventional curing through the addition of insulating blankets, which provide opening strengths at various times earlier than conventional paving but not as early as Fast Track.

SCOPE

The research examined two standard Iowa Department of Transportation concrete mix classes and a modified fast track class. Each mix was placed and then divided into two sections. Conventional curing and insulating blankets were used to cover one section and conventional curing only was used on the other. This division resulted in six test sections. Test beams were cast from each section and tested at particular ages. Temperatures in the pavement and test beams were monitored. Conventional cure would consist of a single application of white pigmented curing compound at a rate of 0.067 gal/yd^2. The insulating blankets consisted of a layer of closed-cell polystyrene foam, protected by plastic film, with an R-value rating of at least 0.5. The three mixes used in this research are shown in Table 1. The Fast Track mix (the *F* mix) was modified by the use of Type I cement instead of Type III cement.

The sand and gravel source was Hallett's at the Jenkins-Sturtz pit, north of Ames, Iowa. Ash Grove cement was used. Class *C* fly ash from Midwest Fly Ash, Sioux City, Iowa, was

Office of Materials, Highway Division, Iowa Department of Transportation, Ames, Iowa 50010.

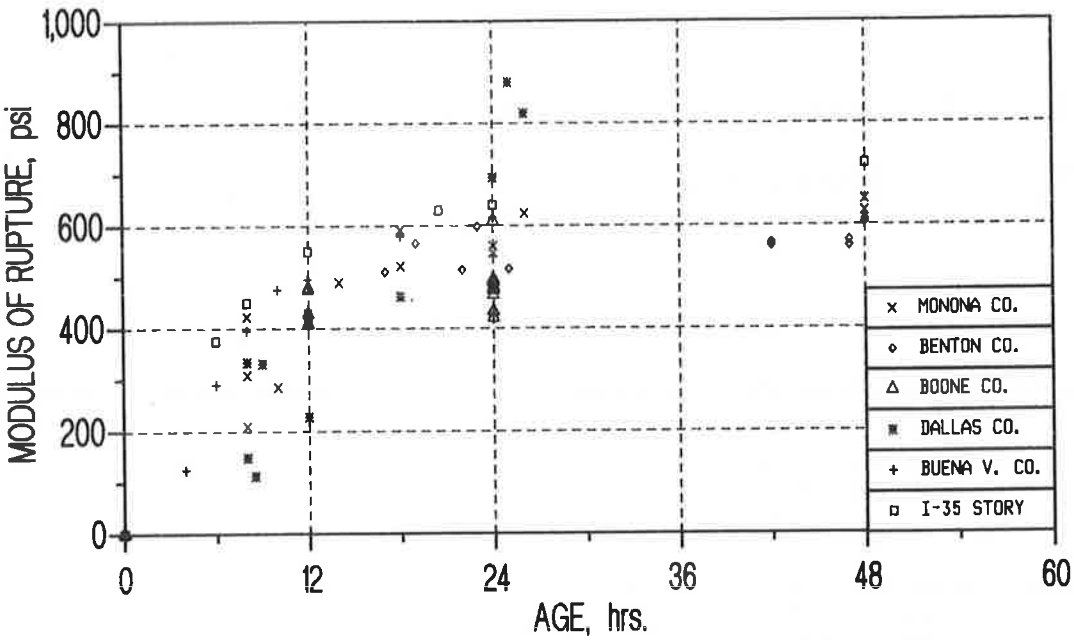

FIGURE 1 Iowa Fast Track concrete.

TABLE 1 MIX PROPORTIONS (LB/YD³)

Mix No.	Cement-Type I	Fly Ash	Fine Aggregate	Coarse Aggregate	Water
F-4	710	0	1403	1434	218
C-4	624	0	1483	1516	195
B-6-C	444	74	1820	1228	253

used. Air entraining agent was W.R. Grace, Darvar R. No water reducer was used.

CONSTRUCTION

The research project was located in Boone County, just east of Boone, Iowa. The test sections were on E41 (Old Highway 30), a short distance west of the east line of Section 26, Township 84N, Range 26W. This section of roadway was being reconstructed with a new 24-ft pavement, 9 in. thick, using a class *B* PCC pavement mix. The portion of the construction project where this research took place was 1.36 mi long.

The sections were constructed on September 25, 1987, on a clear day with the temperature in the mid-70s and a slight breeze. The first section was placed at approximately 2:20 p.m., with the last section being finished at 3:25 p.m. The locations and tests of the concrete are listed in Table 2.

Eight beams were cast for each of the six test sections. After the curing cart had passed the test sections, these beams were placed adjacent to the edge of the slab. All sections and beams were sprayed with curing compound at the normal specified rate. The insulating blankets were then spread over both the slab and beams on those sections being cured in that manner. The blankets were placed at approximately 5:00 p.m. and remained over the pavement for approximately 24 hr.

TESTING

Strength

Two beams were tested from each section at each of the following times: 18 hr, 24 hr, 3 days, and 7 days. The results of the tests of the 48 flexural beams are listed in Table 3. These data are shown in Figure 2 as modulus of rupture versus age.

TABLE 2 TEST SECTION LOCATIONS AND CONCRETE TEST RESULTS

	Location		Slump	Air
	Station	Station	in.	%
Class F				
with blankets	1076+60	1077+00	2	7.2
Class F				
without blankets	1077+00	1077+40	2 1/4	7.5
Class C				
with blankets	1075+95	1076+30	1 3/4	7.8
Class C				
without blankets	1076+30	1076+60	2 1/4	7.8
Class B				
with blankets	1078+90	1078+20	1 1/2	6.5
Class B				
without blankets	1078+20	1078+50	3/4	6.0

TABLE 3 FLEXURAL TEST RESULTS, MODULUS OF RUPTURE (LB/IN.^{2})[a]

Section	18 hr	24 hr	3 day	7 day
Class F w/Insulation	418	462	619	744
Class F Std. Cure	294	363	550	677
Class C w/Insulation	318	406	606	712
Class C Std. Cure	282	319	538	669
Class B w/Insulation	200	282	506	650
Class B Std. Cure	153	282	506	638

Note: Insulation removed after 24 hours of cure.

[a]Average of two tests; center point loading.

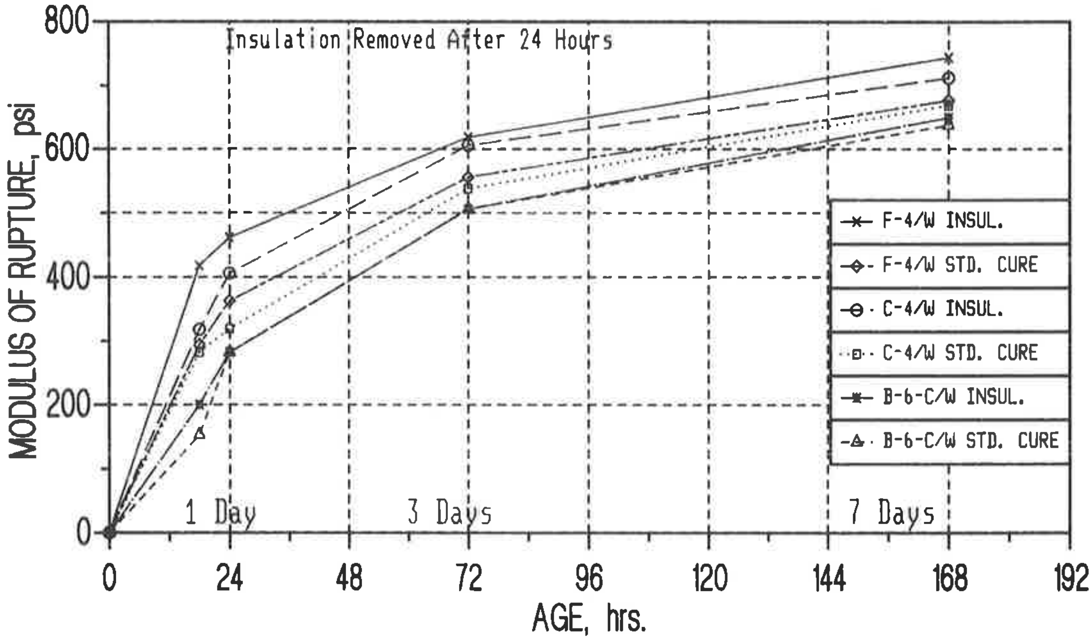

FIGURE 2 Beam strengths.

TABLE 4 COMPRESSIVE STRENGTHS TEST RESULTS (LB/IN.^{2})

	28-day Compressive Strength	
	Tests	Average
Class F w/Insulation	4945; 3820	4385
Class F Std. Cure	5065; 3945	4505
Class C w/Insulation	3500; 3630	3565
Class C Std. Cure	4610; 4295	4455
Class B w/Insulation	3450; 3545	3500
Class B Std. Cure	3130; 2960	3045

The *F* mix showed a 42 percent increase in strength with the blankets at 18 hr and a 27 percent increase at 24 hr when compared with the section cured without blankets. The *C* mix gave a 13 percent increase at 18 hr and a 27 percent increase at 24 hr. The *B* mix had a 31 percent increase at 18 hr but no difference at 24 hr.

Cores of 4 in. were taken from the pavement test sections and tested at 28 days. The test results are shown in Table 4.

Temperature

Table 5 shows the temperatures that were recorded during the research. Measurements were taken in a test beam and also in the pavement for each of the test sections. Figures 3 and 4 are plots of these data with the former showing the temperatures in the pavement and the latter representing the temperatures in the test beams.

The heat retained by the blankets resulted in an increase in temperature, compared with the conventionally cured sections (at the time of the coolest air temperature) of 21°F, 13°F, and 9°F for the Class *F*, *C*, and *B* mixes, respectively.

DISCUSSION OF RESULTS

Strength

With both the Class *F* and Class *C* mixes, a significant gain in additional strength during the first 24 hr resulted from the

TABLE 5 PAVEMENT AND BEAM TEMPERATURES (°F)

Section		Mix	1.5-2 hr[a]	7-hr	`17-hr	24-hr[b]	3-day[b]
Class F w/Insulation	Pavement	77	80	97	105	103	74
	Beam	83	92	83	87	77	
Class F Std. Cure	Pavement	77	79	83	84	97	75
	Beam	81	71	59	95	73	
Class C w/Insulation	Pavement	77	83	92	93	94	74
	Beam	85	75	74	81	68	
Class C Std. Cure	Pavement	75	79	78	80	95	73
	Beam	88	71	62	93	72	
Class B w/Insulation	Pavement	78	76	83	84	90	71
	Beam	82	82	75	80	73	
Class B Std. Cure	Pavement	76	76	79	75	91	71
	Beam	77	69	59	91	74	
Ambient Air		80	80	64	54	82	68

[a]Temperature taken when beams were moved next to slab and insulation was placed on the slab.

[b]Insulation removed after 1 day of cure

use of the insulating blankets. Overall the *F* mix had the largest gain in strength with the use of blankets and the additional gain was evident over the longest period of time. The tests show that some gain in extra strength occurred when the blankets were used with the Class *B* mix, but only in the initial curing period. By 24 hr, both Class *B* mix sections exhibited the same strength. The extra strength gained before that time may not be of great importance since the actual flexural strength at that time was still very low.

The Iowa Department of Transportation Standard Specifications, Section 2301.36, require a strength of 500 psi before a pavement can be opened to traffic, in addition to a minimum age. Based on this strength, three sections exhibited early strength gain sufficient to provide three distinct early opening times. The insulated *F* mix reached 500 psi in approximately 36 hr, the insulated *C* mix in about 48 hr, and the noninsulated *F* mix in about 60 hr. As a comparison, the Fast Track mix normally reached that strength in 18 hr.

The results of the 28-day compressive strength tests performed on core samples were inconclusive. The intent of including these tests was to provide information on pavement strength at a more mature age. A significant variation in strength occurred between each of the test beams for each *F* mix section. The *B* mix exhibited little strength gain or temperature change with the use of the blankets. Nevertheless, a significantly higher strength was exhibited by the *B* mix with the use of the blankets. A loss in ultimate strength may be expected with a gain in early strength, but the significant loss of strength at 28 days exhibited by the *C* mix seemed high.

Temperature

When the temperature plot is compared with the figure showing the strengths, it appears that the uniformity in temperature contributes to the higher gain in strength. Even if the maximum temperature is not as high, the consistent temperature has a significantly favorable effect.

The insulating blankets affected the pavement temperatures by reducing the effect of both the ambient air temperature

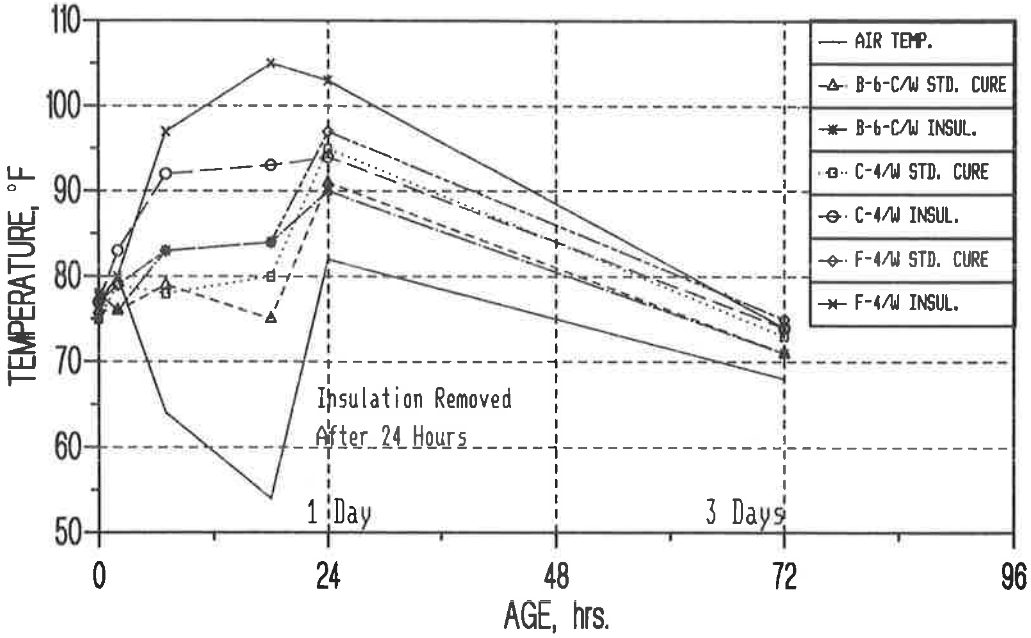

FIGURE 3 Pavement temperatures.

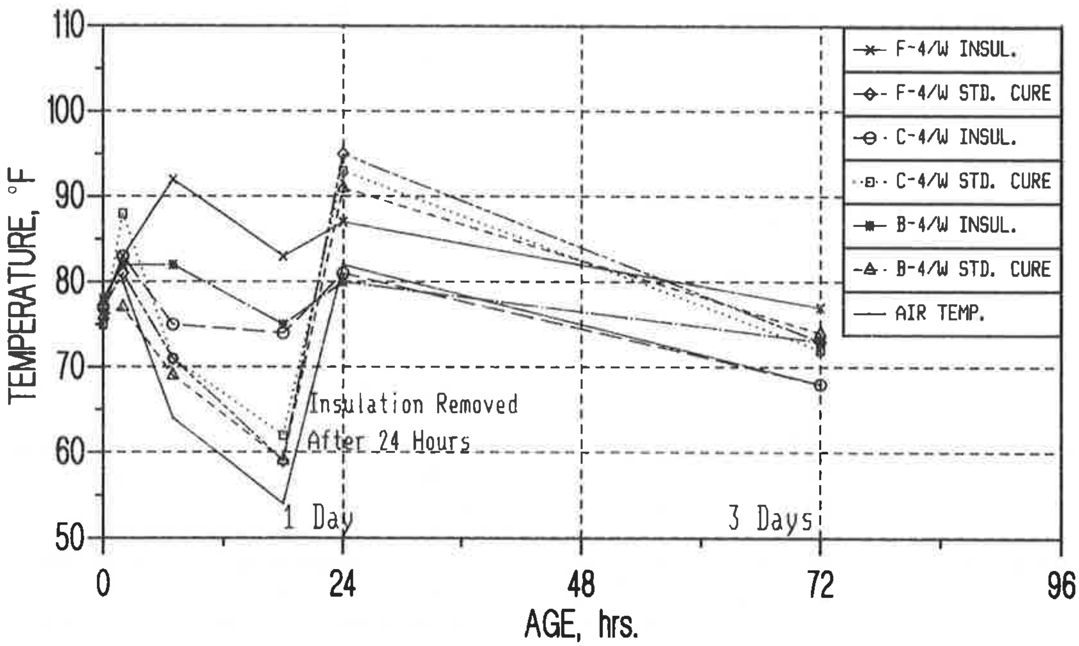

FIGURE 4 Beam temperatures.

and the solar heat. In all three classes of concrete, the pavement sections with the insulating blankets gained some temperature during the cool night. On the other hand, the uncovered *B* and *C* mix sections dipped in temperature as the air cooled. The *F* mix section, without blankets, exhibited a small temperature gain during the night. Even with the gain, it was much less than the *F* mix section that received the insulating blankets.

All the beams gained in temperature, initially, during the sunny afternoon. But, the noninsulated beams closely paralleled the air temperature during the night. The next day those noninsulated beams achieved higher temperatures than the insulated beams. The warm ambient temperature and the heat from the sun actually warmed the beams more than the heat derived from the blankets. It may also be true that the insulated beams achieved more hydration earlier and over a longer period so that a much smaller portion of the process was left to take place during the heat of the following afternoon.

A comparison of the pavement temperatures and the beam temperatures reveals an unsettling situation. The beam temperatures were roughly 20° cooler than the pavement temperatures at 14 hr. The beams generally followed the air temperature, whereas the pavement temperatures were somewhat constant. This raises the question of how well the strengths obtained from testing the beams actually represent the strengths existing in the pavement. Fortunately, the error will likely be such that the pavement is actually stronger than what the beam tests would indicate.

SUMMARY

The following were found to be true in this study:

1. Insulating blankets promote a greater increase in early strength for concrete mixes with a higher cement content.
2. Insulating blankets reduce temperature loss during the first night after placement and thereby prevented interruption of the hydration process.
3. There was no significant strength benefit with the use of insulating blankets for a 444-lb cement–79-lb fly ash mix.
4. Insulating blankets may inhibit temperature gain on a warm, sunny day by shielding the solar heat.
5. An *F* mix (7 1/2 bag mix with Type I cement) can be expected to reach opening strength (500 psi) when cured with insulating blankets in 36 hr.
6. A *C* mix (6 1/2 bag mix) can be expected to reach opening strength when cured with insulating blankets in 48 hr.
7. An *F* mix can be expected to reach opening strength with conventional curing in 60 hr.

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Flowable Fly Ash

William C. Krell

This paper identifies many of those features that distinguish flowable fly ash from other backfill materials, both conventional and controlled low-strength materials (CLSM). The physical properties of the flowable fly ash (using Type F ash) that are discussed are also provided with physical units that define these same properties. These characteristics include its resistance in both a fresh plastic and a hardened state to erosion from flowing water; this resistance to erosion from water normally allows the material to be placed directly into water without the need for tremies. Flowable fly ash, using either dry or conditioned fly ash, can be mixed in ready-mix trucks, pug mills, or turbine mixers. Flowable fly ash can be placed by chutes from ready-mix trucks, by end-dumping from conventional trucks or conveyors, or by pumping. The material can be competitive in cost and has been used in many applications in which its advantages in properties, scheduling, and elimination of hazards to manual labor have far outweighed those of other conventional backfill materials.

Detroit Edison, Detroit, Mich., was the first utility to burn pulverized coal. The company also used electronic precipitators to capture the fine fly ash particles from the stack gases. To capitalize on this by-product, Detroit Edison looked for ways to commercially use this new material. They introduced use of this by-product into the production of concrete, becoming pioneers in the specification that became ASTM C-618. Then, along with Kuhlman Ready-Mix as co-sponsors, they developed K-Krete, the forerunner of the "controlled low-strength materials" (CLSM), now considered under American Concrete Institute's Committee 229. Detroit Edison then developed "flowable fly ash," which has all the properties of a CLSM, but also has additional unique characteristics of its own, and which uses a Type *F* fly ash (*1*).

Flowable fly ash was first developed to use as a backfill material to be placed into deep flowing water. Local granular backfill materials have become harder to obtain, and the cost has been rising. The use of fly ash for a backfill material resulted in two big advantages: it used a by-product that had been wasted, thus helping the environment, and provided a low-cost construction material. The material not only serves as conventional backfill material but also has some additional desirable characteristics. A trial program began in 1979 and consisted of an embankment constructed in flowing water 20 ft deep. Flowable fly ash has been used both on land and in water with excellent success. In this discussion, only Type *F* fly ash is considered.

BACKGROUND

Fly ash is available in quantity from coal-burning utilities. It is a very fine material (similar to portland cement) that is captured from the stack gas stream with emissions control equipment. It ranks sixth by weight of minerals produced yearly in the United States, following stone, sand, gravel, coal, iron ore, and Portland Cement. Flowable fly ash is placed at a stiff plastic to a fluid consistency. It consists of Type *F* fly ash, water, and a small amount of Portland Cement. The design mix is as follows:

1,800 lb of dry fly ash,

90 lb of portland cement,

±80 gal of water,

±7-in. slump.

YIELD

Many ready-mix companies prefer to deal in quantities of yield and cubic yards of material delivered. The total weight of the flowable fly ash mixture given above is 2,556 lb. The net weight of the material varies from plant to plant but generally ranges from about 100 to 105 lb/ft^3. With a unit weight of 102 lb/ft^3, as an example, this mixture would yield about 25 ft^3.

Most conventional backfill materials are charged to a project based on the number of tons delivered. Once the wet unit weight has been established, flowable dry ash can be delivered and costed to a project by similar units. Flowable fly ash eliminates the constant quarrel about price that exists with other conventional backfill materials when they have been delivered to a project either too dry, too wet, or of questionable gradation. The following is also true about fly ash:

1. Flowable fly ash can be used as a fill material in place of compacted soils or granular materials. In addition, it can be placed directly into water or wet conditions and it can be placed by pumping or conventional methods.
2. Dry fly ash obtained directly from ready-mix hoppers has been used, but fly ash conditioned with water to control dusting can be obtained directly from power plants at a fraction of this cost.
3. Flowable fly ash can normally support the weight of a loaded truck within the first 24 hr. In hot weather, it has supported trucks and construction loads in a matter of a few hours.

PRELIMINARY INVESTIGATION

Fly ashes vary because of the type of coal burned and the type of furnace used to burn the coal. Detroit Edison has conducted numerous tests on their own fly ash and also managed a program for the Electric Power Research Institute, which was concerned with a wide base of fly ashes, both

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geographically and with respect to physical qualities. All of the tests indicate a general similarity in the performance of Type *F* fly ash when used as flowable fly ash.

Producers should test local materials before supplying a project. A common-sense program can be done at a minimum cost. Experience has shown that the engineer and contractor both need to develop a feel for the material to take full advantage of its properties. The following are some particular cautions:

- Moist, conditioned ash can hang up in hoppers, which is not a problem unless it is not anticipated.
- Pumping flowable fly ash requires that the concrete pump be in top shape. Leaky gates just do not work.
- The finish surface is not a wearing surface. It will tend to abrade and eventually dust under traffic conditions.
- Droppings on roadways should be avoided as this may also result in dusting.
- Contractors should be aware that when it is wet, the surface will be slippery, similar to a clay. Any coarse-grained material can be spread on top to control this condition.

MANUFACTURE OF FLOWABLE FLY ASH

Flowable fly ash has been successfully produced in pug mills, concrete ready-mix trucks, concrete central mix, and pan turbine mixers.

Quality Control

The burden of ensuring the proper cement content and mixing should be the producer's. Field controls should include compressive tests on 2-in. cubes and the performance requirement that it support the weight of construction equipment. Tests on various-sized samples in this strength range indicate that the 2-in. cube strength is comparable to a standard concrete cylinder of 6-in. diameter by 12-in. length and does not require any correction factor as in the case of concrete testing.

Testing

Flowable fly ash should be tested. First, 2-in. standard cubes should be taken to determine the 28-day strength. Various-sized specimens have been used and, in this strength range, the results have been found to be comparable. Although the standard 6-in. cylinder has been used on projects, it has been determined that the standard 2-in. cube is virtually trouble free. The 2-in. cube requires little storage space and often can be tested on simple soil compression machines.

It should be noted that a typical mix using 5 percent Portland Cement of the dry weight of the fly ash was established for the convenience of the contractor. Most soil procedures use the percent of cement as that of the total weight of all the material. This means that a flowable fly ash using 5 percent Portland Cement by dry weight of fly ash is approximately equal to 2.3 percent Portland Cement based on the total weight of all materials as would normally be reported by a soils laboratory. The point made here is that if the designer compares either costs or materials all elements must be compared on a basis of equal units.

Slump

Slump is measured with a standard slump cone. Its use is limited to the start of a project where it serves to help the contractor get a feel for the proper consistency of the flowable fly ash. After the proper consistency is determined, the slump becomes self-policing, and seldom is it necessary to use it as a field control.

Strength

Unconfined compression tests of flowable fly ash using Type *F* fly ash at 28 days containing 5 percent Portland Cement (by dry weight of fly ash) will show values of about 100 psi. This value is better than 7 tons/ft^2, which is considered superior to a good backfill material that would have a bearing strength from 3,000 to 4,000 lb/ft^2. Flowable fly ash also ensures uniform placement of the material, which is not always the case with conventional compacted materials.

Flexural tests of flowable fly ash indicate that the modulus of rupture (tensile value) is high and is often equal to its compressive strength. This characteristic is not unusual in low-strength cementitious materials. The strength varies with the moisture and the cement content, and for that reason, no value is offered. The tests indicate, however, the slab action provided by the material and the reason it can bridge over soft spongy soil areas.

The subgrade modulus for the design of paving is 50 times the unconfined compressive strength, which is superior to most other sub-base materials (*2*). Its high value indicates that it is better than any backfill that it would replace. The subgrade modulus of a good base course is often assumed to be 500 lb/in.^{3}, and flowable fly ash is 5,000 lb/in.^{3} or over.

Erosion Properties

Field placement has shown that flowable fly ash resists erosion in both its plastic and hardened state. No test (either physical or environmental) could be found that identified the loss of materials when placing backfill in water. A tank was constructed with water being pumped continuously pasts a sample of material being tested. This test was developed to compare materials and determine how they would affect water quality. The flowable fly ash proved to be better than the other backfill materials (Table 1). Clean rock fill is probably the only other material that could achieve these results. This property of flowable fly ash when used as a backfill placed under water should eliminate the need for silt curtains, containment by weirs, and other environmental concerns.

The fly ash and the cement particles are approximately the same size. For this reason, segregation does not occur even when the flowable fly ash is placed through water. In deep flowing water, it would not be difficult to use a tremie for placement of the material if it is required.

Corrosion

Most fly ashes are alkaline. In addition, the 28-day compressive strength is dependent on the action of the Portland Cement,

TABLE 1 TEST FOR EROSION OF VARIOUS FILL MATERIALS

	EROSION TEST	
Material*	% of Sample In Suspension	% Total Sample Loss
Silica Sand	1.6	10.0
Coarse Sand & Gravel	2.7	11.4
Fill Sand	8.3	44.9
Clay	8.9	22.6
Flowable Fly Ash (Fresh Plastic)	0.7	1.3

Obtained from local construction sites

which is also alkaline. The basic alkalinity of the fly ash, along with the enhancement effect of Portland Cement, provides the same beneficial properties as concrete. The flowable fly ash is an ideal environment to inhibit the rusting of iron and steel.

Compaction and Density

Compaction tests were run in accordance with the ASTM D-1557 modified proctor test on individual mixes using 5 percent Portland Cement. It was found that the flowable fly ash attained a density equivalent to 85 percent of that obtained by using the full compactive effort of the test. This property allows the flowable fly ash to be loaded by construction equipment soon after placement. The continuance of the pozzolanic cementing action, along with the hydration of the Portland Cement, provides additional strength. The user should be aware of the following rule of thumb when comparing costs on a given project: 1 ton of fly ash is equivalent to about 1 yd^3, and this volume requires about 1.5 tons of granular material.

TRANSPORTATION AND PLACEMENT

Flowable fly ash has been placed by end-dumping and chuting from ready-mix trucks. The material also has been placed from conveyor lines and by pumping. In its first application, flowable fly ash was end-dumped and then bulldozed into water depths up to 20 ft. Its placement method is limited only by one's imagination.

Freeze-Thaw Resistance

The material has been used with success in several waterfront installations on the Detroit River. When placed in a zone of total water saturation and subject to severe winter freezing at well below 0°F, flowable fly ash broke into pieces about the size of a hand. Sacrificial thickness was provided to allow for this loss, and the projects were successful. Without water saturation, flowable fly ash (Type *F*) at 5 percent cement content appears to perform well under freeze-and-thaw conditions in the field.

Laboratory freeze-thaw tests in water indicate that a cement content of about 10 percent should be used. However, vacuum saturation tests that were suggested by the Michigan Department of Transportation, indicated that the typical 5 percent Portland Cement mixture would perform well in a freeze-thaw environment. It is hoped that a field test that would provide practical proof of this will be done. All the evidence seen in the field to date indicates that the 5 percent Portland Cement mixture should perform as a road base under severe winter conditions.

Permeability

Although the permeability of Type *F* fly ash used in flowable fly ash varies depending on the source of the fly ash, tests on a number of ashes indicate that permeability ranges from 1.9×10^{-6} to 3.3×10^{-7}. This type of permeability places the material in the region of a poor clay soil. Although the use of the material in any environmental application should be checked at that site, laboratory tests indicate that at a 15 percent Portland Cement content (by dry weight of fly ash) can be considered suitable as a lining for landfill applications.

Vibration

Initially, it was felt that this Type *F* fly ash had great potential for use in atomic power plants. Today, economics indicate that this future market is dead. Consideration of cross-hole shear and column resonance tests to determine vibration characteristics of the flowable fly ash were abandoned because of the high cost of conducting such programs.

The initial application of the material was to support railroad tracks that would supply coal to a power plant. A simple test was recommended by the Construction Testing Laboratories. The test was to apply the load found at the bottom of the railroad ties under the full loading of a moving coal train.

No consideration was given to the beneficial effects of the ballast stone. This test concluded that a railroad embankment constructed with a fly ash stabilized with 5 percent cement could carry 620,000 wheel load applications. Loads were applied through a 6-in.-diameter steel plate. The block of hardened flowable fly ash was 12 in. deep and rested on a steel plate support with the lower 2 in. immersed in water. Loads were applied at a rate of 2.5 cycles per second. This test provided sufficient confidence that the material could be used under this vibration condition and would perform satisfactorily.

Ability to Be Dug

Flowable fly ash gains strength with age. Material having a 28-day strength in excess of 100 psi has been dug by a small backhoe at the end of a year's time. At this age, its strength was in excess of 300 psi. No consideration was given to hand excavation because the local construction people emphasized that this simply was not done in today's labor market.

Subsidence

Subsidence does occur with flowable fly ash, and in tests of a 12-in. depth, subsidence appears to be about 0.25 in. This drop or settlement occurs in a matter of minutes. Where the material has been placed in deep fills (12 to 15 ft), this effect is not noticed since time and pressure from new material being placed offset this decrease.

A check of the filling of tunnels and sewers and under slabs indicates that subsidence is not a problem and that tight contact for support can be easily accommodated during construction.

Segregation

Mixtures with slumps up to 10 in. have been placed without segregation. With the addition of more water, the mixture will release free water. Since most mixtures are workable below this slump range, segregation is not a problem.

When free water flows from a mixture, it does not appear to segregate. Most probably the reason segregation does not occur is because the fly ash and Portland Cement are close in size and generally do not contain a larger aggregate.

CASE HISTORIES

In selection of any construction material for use on a job, the effect of the material on costs should be considered. Although its costs vary with availability and the market, flowable fly ash offers many properties that cannot be found in other materials, such as erosion resistance, ability to be placed in water, minimum testing costs, elimination of compaction, and other advantages. The following case histories (*3*) are offered to suggest some of the cost items, beside those of direct material costs that should be considered.

The first case history is that of a six-story apartment building that was found to have pockets of soft material under the foundations. The contractor simply excavated these pockets; then at the end of his working day, he had flowable fly ash (Type *F*) chuted into these voids from a ready-mix truck. The next morning, his carpenters set forms and the footings were cast. Had he resorted to conventional material that required compaction, it would have taken days of manual labor and testing to get the correct material in place. In addition, this work would have seriously disrupted his construction activity. In this case, schedule alone had sufficient cost advantage to support the use of flowable fly ash as a backfill material.

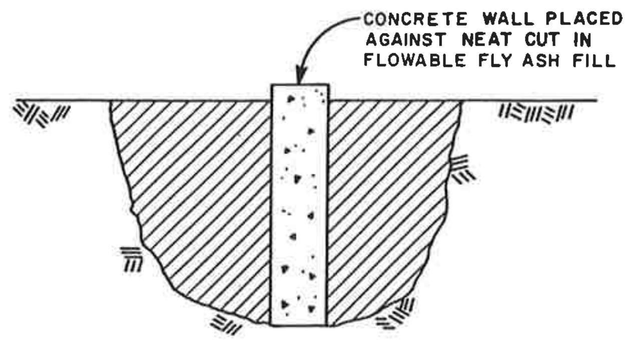

FIGURE 1 Use as formwork and support in caving soils.

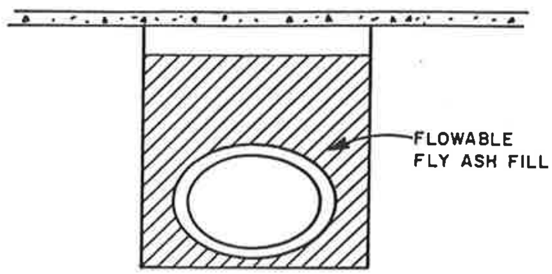

FIGURE 2 Settlement-free backfill around buried pipes.

The second case is that of a multistory office, hotel, and parking structure constructed on a site that consisted of old basements filled with loose materials including bricks, broken concrete, and timber. During excavation, this material was collapsing, creating a wide irregular trench that required form work to cast concrete foundation walls. Flowable fly ash was rapidly placed in these oversized trenches. On the following day, the neat excavation for concrete placement was accomplished (Figure 1). This procedure saved both schedule and money and allowed the walls to be constructed by the subcontractor without change in the original contract.

The third case shows a common use of flowable fly ash in the filling of various utility cuts (Figure 2). When conventional backfill materials are used, the trench must be wide and sloped back to allow room for equipment to compact the material, and to provide for the testing that should be done. In spite of these precautions, dips and cracks often show up in cuts made through roadways. Here, as long as the material can be seen to flow under the pipe, good bedding is provided. The use of flowable fly ash does away with future settlement, minimizing both the amount of excavation and the amount of backfill material required. Inspection costs are also reduced.

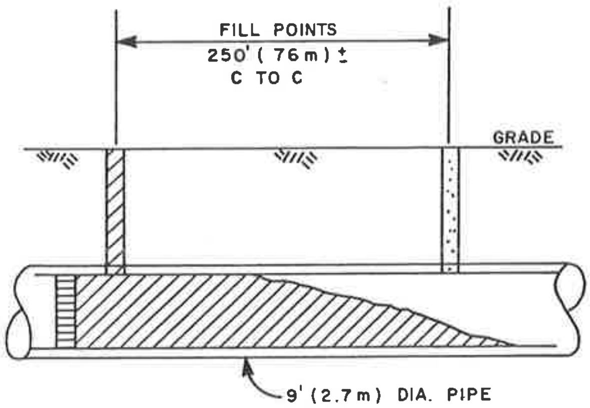

FIGURE 3 Structural fill of underground enclosures.

The last case history is illustrated by Figure 3. Flowable fly ash has been used to fill abandoned gasoline tanks, tunnels, pits, and sewers. In this instance, the material was used to fill an abandoned sewer under a new automotive plant. The fill points were drilled about 250 ft apart. After flowable fly ash was placed in one point, the air was vented at the next point 250 ft away. That point, in turn, became the next fill point. Drilling after placement of the flowable fly ash served to check that the method was successful.

CONCLUSION

This paper was prepared for the Transportation Research Board's annual meeting held in January 1989. This particular session was concerned with "Controlled Low-Strength Concrete," now represented by the American Concrete Institute's Committee 229.

The author has strived to present sufficient technical information that can be used in the design of a fly ash backfill or other installation. The information presented should serve to provide a sound background to design most proposed projects. Other testing has been done for specific projects. Only a few of the available case histories have been presented in this paper (*3*). Much has been learned on each new application of the material.

The use of flowable fly ash is not limited once it is used on a project. The imagination and creativity of the persons involved with it has propelled flowable fly ash into new and various uses. Its unique properties indicate that flowable fly ash has potential to perform well in many unusual applications. The state of the art is that the material can be used by the engineer for new projects, and in most instances only a limited amount of testing is necessary.

It would be appreciated if any new use, test, or other information on the application of flowable fly ash would be sent to the author, so this information can be made available to others.

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Factors Affecting Strength and Durability of Concrete Made with Various Cements

K. ROSE, B. B. HOPE, AND A. K. C. IP

This investigation examined variables influencing the 28-day compressive strength and 50-cycle salt scaling loss of concretes made with 18 Type 10 cements. Statistical analysis was performed on test data consisting of chemical and physical properties of cements; properties of fresh concretes; compressive strength, salt scaling loss, and air void parameters of mature concretes. For a similar water-cement ratio and cement content, results from the correlation analysis indicated that the 28-day concrete strength and 50-cycle salt scaling loss were influenced significantly by the chemical and physical properties of the cement used in the mix. Compressive strength has a strong negative correlation with alkali content, indicating that cements with a high alkali content produced concretes with lower compressive strength. Fineness (percent of particles in the 4 to 20-μm range) is related to low salt scaling loss. Equations predicting strength and salt scaling loss of concrete were developed by using multiple linear regression.

It has been observed for some time that the quality of the concrete used in Ontario has shown considerable variation, even though all the cements used in the manufacture of concrete met the Ontario Provincial Standard Specifications Form 1301 (CSA standard CAN 3-A5-M83) requirements for Type 10 cements. As a result of the observed variation in concrete quality in Ontario, a program of testing (*1*) was undertaken by the Concrete Section in the Engineering Materials Office of the Ontario Ministry of Transportation (MTO). First, physical and chemical properties of 18 samples of cement from 14 different cement plants were measured. These cements were then used to produce samples of concrete with nominal compressive strengths of 20 and 30 MPa, respectively. Within each of these strength categories two types of coarse aggregate were used in the mix, a good-quality crushed dolomitic limestone called A and a poorer-quality partially crushed natural gravel called B. Various properties of these four resulting concrete types (20A, 20B, 30A and 30B) were also measured.

The experimental investigation was designed to examine the influence of various cements on the strength and durability of concrete. Traditional factors such as cement content and water-cement ratio were not intended to be variables. These two factors have a profound influence on the strength and durability of concretes made from a single cement. Within each of the four concrete types examined in this experiment, all used the same aggregate, had the same cement content, and had virtually the same water-cement ratio. For example, the 30A concretes had the following batch quantities:

Fine aggregate, 37.3 kg; coarse aggregate, 53.7 kg; cement, 17.75 kg; and WRDA, 78.3 ml. DAREX AEA was adjusted to give an air content of 6.0 percent and total water was adjusted to give a slump of 80 mm.

The four concrete types had the same components so that the effect of the cements alone could be studied. This paper is based on a statistical analysis of the test data and was undertaken to address three objectives:

1. Verify that there was a significant difference between the quality of the concretes produced by the various cements.
2. Identify the cement properties responsible for the observed variations in the quality of the concrete specimens.
3. Develop a methodology to help predict the quality of a concrete produced by a cement.

It was decided that the quality of the concrete should be assessed in terms of the compressive strength of the concrete (CSA standard A23.2-3C, 9C) and also the cumulative mass loss (mass of material lost from surface) in a salt scaling test (ASTM C672-84, using 3 percent sodium chloride solution as the de-icer), since these properties most closely reflect the concerns expressed about the durability of concrete. Strength was measured at 3, 7, 28, and 91 days, and salt scaling loss was measured at 5, 10, 15, 25, and 50 cycles. It was decided that the 28-day compressive strength and the 50-cycle salt scaling loss would be the most appropriate variables to use. Therefore, for the purposes of this analysis, these two variables (for each of the four types of concrete) were considered to be the dependent variables. A more detailed description of the analyses reported in this paper is available (*2*).

DATA FILES

The MTO data have been reported (*1*) in the form of 12 tables labeled *A* through *L*. These tables included measurements of chemical and physical properties of the cements, measurements made on the fresh concrete, and measurements made on the mature concretes. These variables are defined in Table 1.

Table *C* of the MTO data contained the grading curves for the cements. The original 13 variables showed the percent passing specified sieve sizes. It was decided to augment these data by calculating the percentage of cement between two sieve sizes that was passing the larger sieve but retained by the smaller. This step resulted in 42 additional variables. Because the data are highly correlated, only one or two were

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TABLE 1 VARIABLES AND DEFINITIONS AND TEST METHODS

VARIABLE	DEFINITION/TEST METHOD
Air content	Measured at time of casting (%)
Air void content (total)	Measured on hardened concrete (%)
Alkali content	Na_2O + 0.658(K_2O) (%)
Aluminum oxide	Al_2O_3 content of cement (%)
Carbonate addition	[L.O.I.(at 1050 ^oC) - L.O.I. (at 550 ^oC)] limestone (L.O.I.)] (%)
Cement content	Cement content per m^3 concrete (kg/m^3)
Chord length (total)	Determined by ASTM C457-82a (mm)
Density, Concrete	CSA Standard CAN 3-A23.2-M77 (kg/m^3)
Effective water-cement ratio	Mass of water/mass of cement
Effective water-cement ratio*	Mass of water/mass of cement finer than 45 μm
False set	CSA Standard CAN 3-A5-M83 (%)
Ferric oxide	Fe_2O_3 content of cement (%)
Fineness (% passing xx μm)	Cement finer than xx μm
Fineness (% between x-yy μm)	Cement between x and yy sizes (%)
Free calcium oxide	ASTM C114-83a (%)
Insoluble residue	CSA Standard CAN 3-A5-M83 (%)
Loss on ignition	CSA Standard CAN 3-A5-M83 (at 1050 ^oC)(%)
Paste-air ratio (total)	Measured on hardened concrete
Potassium oxide	K_2O content of cement (%)
Relative density, cement	CSA Standard CAN 3-A5-M83
Spacing factor (total)	Measured on hardened concrete (mm)
Specific surface area	Calculated from cement grading (mm^{-2})
Set time	CSA Standard CAN 3-A5-M83 (min)
Silicon dioxide	SiO_2 content of cement (%)
Slump	CSA Standard CAN 3-A23.3-M77 (mm)
Soundness	Le Chatelier Test (% expansion)
Strength x-day, cement	CSA Standard CAN 3-A5-M83 at x days (MPa)
Strength x-day, concrete	CSA Standard CAN 3-A23.2-M77 at x days (MPa)
Sulphur trioxide	SO_3 content of cement
Tetracalcium aluminoferrite	C_4AF content of cement (%)
Tricalcium aluminate	Calculated C_3A content of cement
Tricalcium silicate	Calculated C_3S content of cement
Voids per 25 mm (total)	Measured on hardened concrete
Void specific surface (total)	Measured on hardened concrete (mm^{-1})

used in any models that were developed. Finally, it was decided to include the calculated total specific area of the cements as a variable. This variable was included because its calculation made use of all the data in the grading curve.

Table *D* contained the results of 19 chemical tests performed on the cements. In addition, the data were used to calculate three more variables (the percentage of C_3S, C_2S, and C_4AF) thought to be of possible relevance.

PRELIMINARY ANALYSIS

The first stage in the analysis was to verify that the observed variation in the measured strength and salt scaling loss for the various concretes was not due to chance alone. The 28-day compressive strength of each concrete sample was measured for four cylinders, and the 50-cycle salt scaling loss for each concrete sample was measured on two cylinders. From these data an analysis of variance was undertaken to test if the variation in results was caused by the various cements. The results of this analysis are summarized in Table 2.

These results show that in all cases there is better than a 99.5 percent confidence (i.e., the probability that the *F* values could occur by chance is less than 0.005) that the cements differ in their effects on the observed compressive strength and salt scaling loss in the concretes. We therefore concluded that the observed differences in quality are real.

CORRELATION ANALYSIS

The basic step in developing models that can be used to predict concrete quality is to identify the degree of association between a dependent variable and the independent variables. The SYSTAT (*3*) package of statistical programs was used to analyze the data. The Pearson correlation coefficient provides a measure of the interrelationships between pairs of variables. A correlation coefficient close to +1 (or −1) indicates a strong association with a positive (or negative) relationship. A coefficient close to zero indicates little or no association between the pair of variables. The calculated correlation coefficient between the dependent variables and each of the independent variables was classified as

- potentially important (correlation coefficient between −1.0 and −0.4 or +0.4 and +1.0);
- possibly important (correlation coefficient between −0.4 and −0.2 or +0.2 and +0.4); and
- not important (correlation coefficient between −0.2 and +0.2).

The first breakpoint (of ±0.4) for making a decision is based on the fact that the 95 percent confidence level for the correlation coefficient of 18 pairs of observations is ±0.44.

Initially the correlation between the 28-day strength and the 50-cycle salt scaling loss for each type of concrete was calculated. Using the above criteria, there was no relationship between these properties for 30A, 30B, and 20B concretes. In the case of the 20A concrete, the correlation coefficient was −0.51. In all cases the calculated correlation coefficient was negative, indicating that the samples with higher strength had lower durability. Tables 3 and 4 show which of the measured variables were found to be potentially important for the 28-day strength and the 50-cycle salt scaling loss, respectively, for each of the four concrete types. The number associated with each variable is the calculated correlation coefficient.

The independent variables that were found to be potentially important were further analyzed to detect any cross-correlation between them. This analysis was necessary because a good predictive model should not contain closely related variables.

One other test was made on the potentially important variables. Influence plots were made for each variable. In this way it was possible to verify that the high correlation coefficient was not caused by an unusual outlying point. In no case was it necessary to reject a variable because of outlying points, suggesting that the experimental data were representative.

Of equal interest are the variables that were found to be unimportant in their association with concrete strength and durability. These variables are listed in Tables 5 and 6.

TABLE 2 VARIANCE ANALYSIS FOR TEST DATA

CONCRETE TYPE		F RATIO FOR CEMENT SOURCE	PROBABILITY THAT F VALUE COULD OCCUR BY CHANCE
30A	STRENGTH	76.31	< 0.001
	SALT SCALING	4.57	0.003
30B	STRENGTH	116.12	< 0.001
	SALT SCALING	3.89	0.005
20A	STRENGTH	60.09	< 0.001
	SALT SCALING	24.75	< 0.001
20B	STRENGTH	52.07	< 0.001
	SALT SCALING	16.92	< 0.001

TABLE 3 VARIABLES POTENTIALLY IMPORTANT TO 28-DAY STRENGTH OF CONCRETE SPECIMENS IN ORDER OF DECREASING IMPORTANCE

INDEPENDENT VARIABLE	COEFFICIENT
20A CONCRETE	
Soundness	-0.66
Alkali content	-0.57
Silicon dioxide	0.53
Spacing factor (total)	-0.52
Free calcium oxide	-0.48
Chord length (total)	-0.47
Potassium oxide	-0.47
Set time	0.46
Relative density, cement	0.46
Voids per 25mm (total)	0.45
Void specific surface (total)	0.44
Sulphur trioxide	-0.43
Fineness (% passing 40 μm)	0.40
Carbonate addition	-0.40
20B CONCRETE	
Alkali content	-0.85
Cement content	0.77
Potassium oxide	-0.71
Free calcium oxide	-0.68
Density, concrete	0.64
Silicon dioxide	0.60
Relative density, cement	0.59
Soundness	-0.59
Ferric oxide	0.58
Slump	-0.55
Set time	0.55
Sulphur trioxide	-0.53
Strength 3-day, cement	-0.49
Effective water-cement ratio*	-0.46
Tricalcium aluminate	-0.45
Effective water-cement ratio	-0.43

INDEPENDENT VARIABLE	COEFFICIENT
30A CONCRETE	
Alkali content	-0.75
Ferric oxide	0.68
Tetracalcium aluminoferrite	0.68
Potassium oxide	-0.60
Strength 28-day, cement	0.60
Relative density, cement	0.57
Set time	0.55
Fineness (% passing 50μm)	0.55
Carbonate addition	-0.54
Soundness	-0.48
Density, concrete	0.46
Tricalcium aluminate	-0.46
Cement content	0.45
False set	0.45
Loss on ignition	-0.41
30B CONCRETE	
Alkali content	-0.79
Silicon dioxide	0.62
Potassium oxide	-0.62
Cement content	0.62
Sulphur trioxide	-0.62
Soundness	-0.58
Density, concrete	0.56
Air void content (total)	-0.53
Air content	-0.53
Free calcium oxide	-0.51
Tricalcium aluminate	-0.49
Paste-air ratio (total)	0.48
Aluminum oxide	-0.46
Relative density, cement	0.44
Ferric oxide	0.44
Tetracalcium aluminoferrite	0.44
Strength 3-day, cement	-0.43
Set time	0.43
Carbonate addition	-0.43
Effective water-cement ratio*	-0.41

* Mass of water/mass of cement finer than 45 μm

TABLE 4 VARIABLES POTENTIALLY IMPORTANT TO 50-CYCLE SALT SCALING LOSS OF CONCRETE SPECIMENS IN ORDER OF DECREASING IMPORTANCE

INDEPENDENT VARIABLE	COEFFICIENT	INDEPENDENT VARIABLE	COEFFICIENT
20A CONCRETE		30A CONCRETE	
Fineness (% between 2-20 μm)	-0.79	Fineness (% between 2-20 μm)	-0.75
Effective water-cement ratio*	0.68	Effective water-cement ratio*	0.67
Aluminum oxide	0.68	Aluminum oxide	0.66
Tricalcium aluminate	0.67	Tricalcium aluminate	0.65
Fineness (% passing 45 μm)	-0.66	Strength 91-day, concrete	-0.63
Paste-air ratio (total)	0.61	Fineness (% passing 45 μm)	-0.61
Air void content (total)	-0.59	Strength 7-day, concrete	-0.60
Strength 28-day, concrete	-0.52	Insoluble residue	0.58
Ferric oxide	-0.49	Potassium oxide	0.48
Tetracalcium aluminoferrite	-0.49	Tricalcium silicate	-0.46
Voids per 25 mm (total)	-0.49	Ferric oxide	-0.44
Insoluble residue	0.49	Tetracalcium aluminoferrite	-0.44
Strength 91-day, concrete	-0.48	Strength 28-day, concrete	-0.44
Set time	-0.42	Specific surface areas	0.43
Tricalcium silicate	-0.41	Chord length (total)	0.43
		Effective water-cement ratio	0.43
		Cement content	-0.40
		Spacing factor (total)	0.40
20B CONCRETE		30B CONCRETE	
Fineness (% between 4-20 μm)	-0.53	Effective water-cement ratio*	0.73
Air content	-0.46	Fineness (% between 2-20 μm)	-0.72
Set time	-0.45	Fineness (% passing 45 μm)	-0.62
Strength 3-day, concrete	-0.43	Tricalcium aluminate	0.61
		Aluminum oxide	0.57
		Ferric oxide	-0.57
		Tetracalcium aluminoferrite	-0.57
		Potassium oxide	0.50
		Alkali content	0.48
		Insoluble residue	0.46
		Set time	-0.46
		Air content	-0.46

* Mass of water/mass of cement finer than 45 μm

TABLE 5 VARIABLES NOT RELATED TO 28-DAY STRENGTH OF CONCRETE SPECIMENS

20A Concrete	20B Concrete
Fineness (air permeability), cement	Fineness (air permeability)
7 day strength, cement	Air content, cement
Insoluble residue, cement	7 day strength, cement
Magnesium oxide, cement	Magnesium oxide, cement
Sodium oxide, cement	Sodium oxide, cement
Tricalcium silicate, cement	Tricalcium silicate, cement
Water (total), concrete	Dicalcium silicate, cement
Slump, concrete	Air void content, concrete
Air content, concrete	Voids per 25mm (total), concrete
Relative density, concrete	Average chord length (total), concrete
Yield, concrete	Paste-air ratio (total), concrete
Cement content, concrete	Void specific surface (total), concrete
Effective water-cement ratio, concrete	Spacing factor (total), concrete
Effective water-cement ratio*, concrete	Specific surface area, cement
Air void content (total), concrete	
Paste-air ratio (total), concrete	

*Based on cement passing 45 micron sieve

30A Concrete	30B Concrete
Fineness (air permeability), cement	Fineness (air permeability), cement
Air content, cement	Air content, cement
Sodium oxide, cement	7 day strength, cement
Tricalcium silicate, cement	Sodium oxide, cement
Dicalcium, silicate, cement	Tricalcium silicate, cement
Water (total), concrete	Average chord length (total), concrete
DAREX AEA dosage	Void specific surface (total), concrete
Slump, concrete	Spacing factor (total), concrete
Effective water-cement ratio, concrete	
Average chord length (total), concrete	
Void specific surface (total), concrete	

TABLE 6 VARIABLES NOT IMPORTANT TO 50-CYCLE SALT SCALING LOSS IN CONCRETE SPECIMENS

20A Concrete	20B Concrete
False set, cement	Relative density, cement
Air content, cement	Soundness, cement
3 day strength, cement	False set, cement
7 day strength, cement	Air content, cement
Loss on ignition, cement	3 day strength, cement
Sulphur trioxide, cement	7 day strength, cement
Free calcium oxide, cement	Loss on ignition, cement
DAREX AEA dosage	Insoluble residue, cement
Air content, concrete	Tricalcium aluminate, cement
Relative density, concrete	Magnesium oxide, cement
3 day strength, concrete	Alkali content, cement
Average chord length (total), concrete	Aluminium oxide, cement
Void specific surface (total), concrete	Ferric oxide, cement
	Calcium oxide, cement
	Free calcium oxide, cement
	Tricalcium silicate, cement
	Tetracalcium aluminoferrite, cement
	Water (total), concrete
	Slump, concrete
	Yield, concrete
	Cement content, concrete
	Effective water-cement ratio, concrete
	Effective water-cement ratio*, concrete
	28 day strength, concrete

*Based on cement passing 45 μm sieve

TABLE 6 (*continued on next page*)

GRADING CURVE DATA

The 56 variables derived from the cement grading curve data were treated separately because of their strong interrelationships. For each dependent variable, plots were made showing the range of particle size under consideration, labeled with the corresponding correlation coefficient. From an inspection of these plots a representative variable was selected for inclusion in the modeling process.

A consistent pattern emerged from these plots. The 28-day strength had few or no strong correlations with grading. The salt scaling results on the other hand exhibited two common features. First, the percentage of very fine particles (<2 μm) had a strong positive correlation. Second, the percentage of particles in the variables around the range of 4–20 μm had very strong negative correlations (the strongest of any of all the variables measured), much stronger than the commonly used percentage passing 45 μm.

Table 7 lists the variables from the grading curve data, (and their correlation coefficients), that were retained for the model building phase of the analysis.

REGRESSION ANALYSIS

For this stage of the analysis, it was decided to use regression analysis to develop an equation to predict the 28-day strength and 50-cycle salt scaling loss of the concretes.

Three criteria were used to select the equations:

1. The equation should preferably contain variables that can easily be measured before the concrete has set.
2. The equation should contain only three or four variables. That is, a simple equation that requires the measurement of a few variables is preferable to an equation requiring many measurements, even if some precision is lost.

TABLE 6 (*continued*)

	91 day strength, concrete
	Voids per 25 mm (total), concrete
	Average chord length (total), concrete
	Void specific surface (total), concrete
	Spacing factor (total), concrete
30A Concrete	30B Concrete
Fineness (air permeability), cement	Fineness (air permeability), cement
Set time, cement	False set, cement
Air content, cement	3 day strength, cement
3 day strength, cement	7 day strength, cement
7 day strength, cement	Loss on ignition, cement
Loss on ignition, cement	Sulphur trioxide, cement
Sulphur trioxide, cement	Sodium oxide, cement
Free calcium oxide, cement	Silicon dioxide, cement
Carbonate addition, cement	Calcium oxide, cement
DAREX AEA dosage	Free calcium oxide, cement
Air content, concrete	Carbonate addition, cement
3 day strength, concrete	Dicalcium silicate, cement
Voids per 25mm (total), concrete	Relative density, concrete
Average chord length (total, concrete	Yield, concrete
Void specific surface (total), concrete	Cement content, concrete
Spacing factor (total), concrete	3 day strength, concrete
	Air void content (total), concrete
	Paste-air ratio (total), concrete

3. The equation should be logical, i.e., the signs of the coefficients should make sense.
4. The equations must be statistically significant.

The SYSTAT program contains a set of routines called Multiple General Linear Hypothesis that can calculate several types of regression equations and apply many tests to check their statistical significance. One particular option in the SYSTAT package is stepwise regression. With this method the program reviews the selected independent variables and introduces them into the equation one at a time in an attempt to maximize the coefficient of multiple regression. This technique is useful in reviewing candidate variables, but the resulting equations must be examined with care. In this analysis over 30 equations were evaluated before final selection. This approach was applied to the 28-day compressive strength and the 50-cycle salt scaling loss for each type of concrete. Tables 8 and 9 show the final equations derived for the strength and the salt scaling loss of each type of concrete (along with the most relevant statistics), respectively. An explanation of the output is as follows: "Variable" lists the variables in the estimated regression equation; and "Coefficient" is the calculated value of the coefficient for each variable in the equation.

TABLE 7 POTENTIALLY IMPORTANT VARIABLES FROM THE GRADING CURVE

Case	Variable	Coefficient
28-Day strength		
20A concrete	% Passing 40 μm	0.40
20B concrete	None	—
30A concrete	% Passing 50 μm	0.55
30B concrete	None	—
50-Cycle salt scaling		
20A concrete	% Between 2 and 20 μm	−0.79
20B concrete	% Between 4 and 20 μm	−0.53
30A concrete	% Between 2 and 20 μm	−0.75
30B concrete	% Between 2 and 20 μm	−0.72

TABLE 8 REGRESSION ANALYSIS FOR 28-DAY STRENGTH OF CONCRETE SPECIMENS

TYPE	VARIABLE	COEFFICIENT	t	R^2	E	F
20A	Constant	-128.49	-3.26	0.86	1.09	17.07
	Soundness	-18.23	-3.84			
	Chord length (total)	-43.91	-4.13			
	Relative density, cement	52.69	4.19			
	Fineness (% passing 40 μm)	0.18	2.29			
20B	Constant	45.28	20.45	0.89	1.13	34.97
	Alkali content	-15.21	-6.55			
	Free calcium oxide	-1.76	-2.45			
	Set time	0.03	3.20			
30A	Constant	-163.23	-1.91	0.78	2.09	13.87
	Alkali content	-14.86	-3.31			
	Fineness (% passing 50 μm)	0.55	2.85			
	Relative density, cement	55.05	2.13			
30B	Constant	71.23	15.99	0.88	1.22	30.81
	Alkali content	-16.12	-6.84			
	Air content	-3.04	-4.33			
	Carbonate addition	-0.55	-2.66			

TABLE 9 REGRESSION ANALYSIS FOR 50-CYCLE SALT SCALING LOSS OF CONCRETE SPECIMENS

TYPE	VARIABLE	COEFFICIENT	t	R^2	E	F
20A	Constant	1858.58	1.39	0.74	215	11.23
	Fineness (% between 2-20 μm)	-26.59	-1.21			
	Aluminum oxide	238.08	2.39			
	Air void content	-195.34	-1.74			
20B	Constant	10673.21	2.83	0.28	1160	5.78
	Fineness (% between 4-20 μm)	-203.88	-2.40			
30A	Constant	3705.41	3.99	0.68	245	13.71
	Fineness (% between 2-20 μm)	-66.95	-4.25			
	Potassium oxide	577.65	2.14			
30B	Constant	-748.56	-0.61	0.80	237	17.08
	Effective water-cement ratio	8674.63	4.65			
	Air content	-492.64	-3.64			
	Insoluble residue	622.65	2.00			

For example, for the 28-day strength of type 30A concrete:

$$\text{28-day strength} = -163.23 - 14.86\,(\text{alkali content}) + 0.55\,(\text{fineness, \% passing 50 }\mu\text{m}) + 55.05\,(\text{relative density, cement})$$

t is the Student's t statistic for each coefficient. It is a test to see whether the value of the coefficient is different from zero. A value of t larger than ± 2 for a sample size of 18 indicates a 95 percent confidence that the calculated coefficient is significant.

R^2 is a measure of the success of the regression equation in the variation in the data. The value is between 0 and +1—the larger the value the better the equation. Care must be taken not to overload the equation with variables just to improve the value of this statistic.

E is the standard error of the estimate. This an estimate of the variation about the regression. The smaller the value, the more precise will be the predictions. Analysis of Variance is a test of the overall significance of the regression. The higher the value of the F-ratio, the better.

Reviewing these equations reveals the following observations:

1. Strength: Good equations were obtained for 20A, 20B, 30A, and 30B concretes. The R^2 values were large and all of the coefficients were significant.
2. Salt scaling.
   - 20A concrete: The overall quality of the equation was getting worse. In particular, the t value of the coefficients was approaching ± 2, indicating lower confidence in the calculated values of the coefficients.
   - 20B concrete: The equation had a very low value for R^2 and would be of little practical use.
   - 30A concrete: The R^2 value in the equation was reasonably large. This, combined with the fact that all the coefficients were significant, means that the equation has some potential.
   - 30B concrete: The R^2 value in the equation was large, but the t value of the constant term was very small. This difference is probably because of the inclusion of a variable in the equation that has a large mean value but a small standard deviation. To the regression equation this variable looks like a constant term even though the variable is highly correlated to the dependent variable.

To give an overall impression of the effectiveness of the equations, Figure 1 shows a typical plot of the predicted values along with the corresponding measured values of 28-day strength for 30B concrete. Since it contains several variables, the regression equation cannot be plotted. The graph shows a line at 45 degrees to the axes, indicating perfect agreement between the predicted and measured 28-day strengths. The points show the results from the equation.

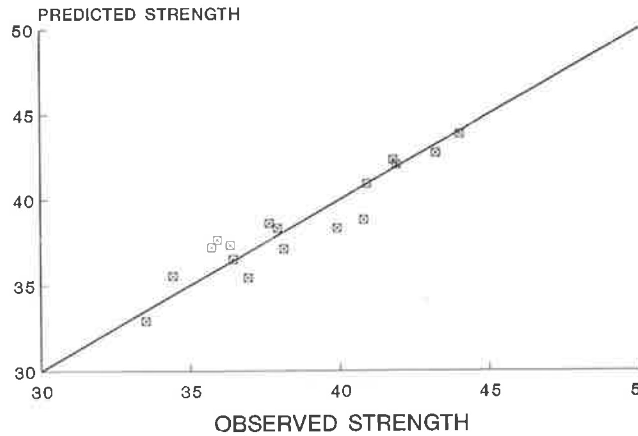

FIGURE 1 Predicted versus observed strength of 30B concrete.

CONCLUSION

The results of this analysis indicate that, for the cements used in this experiment (with similar water-cement ratio and cement content), the concrete strength and salt scaling loss are highly dependent on the chemical and physical properties of the cement used in the mix. The cement variables found to be correlated to the concrete strength and salt scaling loss are shown in Tables 6 and 7, respectively.

A review of these tables indicates that consistent patterns emerge for both compressive strength and salt scaling and that several variables were found to be significant for all of the concretes tested. The 28-day compressive strength had a strong negative correlation with alkali content (in particular K_2O content), indicating that cements with a higher alkali content were associated with lower strength concretes. Soundness (as measured by a Le Chatellier expansion) also had a significant negative correlation in all cases. On the other hand, relative density and set time had positive correlations.

In the case of the 50-cycle salt scaling loss, fineness (percentage of particles in the range of 4–20 μm) has a strong negative correlation. Since we are interested in low scaling losses, this is a desirable attribute in a cement. To a lesser extent the iron content and aluminum content are also associated with salt scaling loss. The iron content has a negative (desirable) correlation, and the aluminum content has a positive (undesirable) correlation.

Because this study investigated a limited number of cements it is premature to draw general conclusions. However, cement manufacture has changed significantly in the past two decades, mainly as a result of environmental and energy concerns (*4*). Environmental concerns have resulted in the use of fuels with reduced SO_3; in addition, kiln dust, which is rich in alkalis, is routinely collected and returned to the kiln, resulting in cements with a higher alkali content. Energy concerns have probably led to a gradual reduction in kiln firing temperatures and also, perhaps, the length of time the clinker is retained in the kiln.

Certainly it is now being reported worldwide that modern cements do not produce concretes as durable as those made with older cements. Concrete placed since about 1970 is much more vulnerable to carbonation (*5*), and older parking structures appear to suffer less deterioration than newer ones (*6*).

It has been suggested that excessive expansion of concrete subjected to wetting and drying cycles is related to incomplete kiln reactions and that deleterious reactions occur in the larger cement particles (*7*).

The experimental results clearly show that different cements produce concretes with widely differing strengths and durabilities at the same cement content, air content, slump, aggregate type, and content (the basic concrete properties held constant within each class of concrete). Equally clearly, the different cement properties are a function of the raw material properties and cement manufacturing processes.

Unfortunately, we have no knowledge of the manufacturing process or the raw materials used for the cements studied. Some of these data were requested, but no information was made available to us by the various cement companies. Although we do not know the sources of the various cements, we are aware that some were manufactured in Canada and some were manufactured in the United States.

The concrete samples and measurements were made under carefully controlled laboratory conditions, and the statistical analysis has shown that the differences in strengths and durabilities are a function of the cement and are not a random effect of experimental error. We therefore concluded that the finding of this analysis should prove useful in identifying methods to improve the strength and durability of concrete.

To refine these results, we would suggest that researchers consider undertaking similar measurements using other cements so that the data base used to estimate these relationships can be broadened.

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Chloride Permeabilities of Rigid Concrete Bridge Deck Overlays

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A study of the chloride permeability of rigid concrete bridge deck overlays was conducted during the 1987 construction season. A total of thirteen latex-modified concrete (LMC) overlays, ten superplasticized dense concrete (SDC) overlays, and two condensed silica fume concrete (CSFC) overlays, were investigated. Concretes were tested both by the rapid chloride permeability test and by 90-day chloride ponding. Results of this study indicate that, of the three materials tested, CSFC is the most impermeable to chloride ions. Following CSFC, in order of increasing permeability, are LMC and SDC. Although all materials show a decrease in permeability with age, chloride ions slowly penetrate these materials and can build up to substantial levels over long periods of time. For newly constructed overlays, there was a good correlation between results of the rapid chloride permeability and 90-day ponding tests.

One of the most serious problems facing the highway engineering community today is the state of highway bridges across the country. It is estimated that at least 160,000 bridges on the interstate system alone are exhibiting such deterioration (*1*).

Of all solutions proposed, rigid overlays have certain advantages, including familiarity of technology, restoration of riding quality, and relatively low cost. The two most popular forms of rigid overlay are latex-modified concrete (LMC) and low-slump dense concrete (LSDC), the "Iowa" method. Superplasticizers may be used with the latter to increase workability, resulting in superplasticized dense concrete (SDC). The use of condensed silica fume concrete (CSFC) in bridge deck overlay applications is relatively new, although use is expected to increase. A comparison of these various overlay systems is needed with respect to their ability to reduce further ingress of chloride ions, so that the most effective methods of protecting this large number of bridge decks may be chosen.

OBJECTIVES AND SCOPE

The primary objective of this research program was to obtain information on the chloride permeabilities of LMC, SDC, and CSFC overlay concretes, both for in-place and new construction, for the purpose of ascertaining which system affords a more impermeable barrier to deicing salts. A secondary objective was to assess the utility of the rapid chloride permeability test (RCPT) (*2, 3*) as a means of measuring permeability of in-place structures and of monitoring permeability of concrete placed on new construction.

The objectives were carried out within the following scope:

1. Cores were obtained from existing LMC, SDC, and CSFC overlays and tested for chloride ion content and rapid chloride permeability. Overlays ranged in age from 1 month to 14 years.
2. Test specimens were cast during construction of LMC and SDC overlays and a CSFC bridge deck during the 1987 construction season. Specimens were subjected to 90-day chloride ponding and RCPT.
3. Results were analyzed to determine relative permeabilities of LMC, SDC, and CSFC overlay materials. Relationships between RCPT and 90-day ponding results were developed.

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BACKGROUND

The performance of rigid overlays, in general, has been favorable. The precursor to SDC, LSDC, incorporates a cement factor of approximately 820 lb/yd^3. A low water-cement ratio (w/c) of 0.30 to 0.32 by weight is most commonly used. Consolidation to a level of 98 percent of rodded unit weight must be achieved, and air contents are specified at 6.5 ± 1.0 percent. LSDC overlays generally are placed at thicknesses ranging from 1.75 to 2 in. Vigorous vibration is needed to achieve the required degree of consolidation of the concrete. A study by the Iowa Department of Transportation (DOT) on 15 overlays, ranging from 5 to 13 years of age, indicated that none of the installations showed evidence of riding surface distress (*4*). Although the top 1-in. of most of these overlays exhibited significant chloride penetration after 11 years of service, chloride contents at depths below 1.5 in. were less than 1 lb/yd^3 and did not appear to increase progressively with time, at least within the 11-year period of study. Likewise, LSDC overlays installed by the Minnesota DOT from 1975 through 1977 have shown good performance (*5*), although in these overlays the measured concrete chloride contents were somewhat higher at equivalent ages and depths than in the Iowa study. Most of these data, however, relate to the low-slump version of the system. Less work has been done and hence fewer performance data are available for SDC. A short-term laboratory study (*6*) indicates that SDC and LSDC should be equally resistant to chloride ion penetration. Results reported by Whiting and Kuhlmann (*7*), on the other hand, indicate significantly higher permeabilities (as measured by AASHTO T277) for SDC than for LMC.

LMC concretes have exhibited generally good performance in terms of their ability to reduce chloride penetration. Mix designs used for LMC overlays usually call for 658 lb/yd^3 of cement, relatively low water content (w/c = 0.39 or less) and 15 percent (by cement weight) of a 46 percent emulsion of styrene-butadiene latex. Air entrainment normally is not used with LMC. Bishara (*8*), in a study of 132 bridge decks, reported much lower chloride contents in decks overlaid with LMC than in nonprotected decks. In the Minnesota study previously cited (*5*), chloride contents below 1.5 in. in decks overlaid with LMC were at the same low levels (< 1 lb/yd³) as similar decks overlaid with LSDC. Studies in Indiana, Kentucky, and Michigan summarized in a recent publication (*9*), show equally good performance for LMC.

A new type of overlay system currently undergoing experimental trials in a number of states incorporates condensed silica fume and may be termed CSFC. These concretes generally use a relatively high cement content (600 to 700 lb/yd³), low w/c ratio (in many cases <0.30), and sufficient high-range water reducer to obtain a workable concrete mixture. Additions of silica fume have ranged from 5 to 15 percent, based on cement weight. Laboratory data (*10*, *11*) indicate that reduction in permeability to chloride ions is a function of silica fume addition, with the greatest reductions occurring at additions above 10 percent by weight of cement. Bunke (*12*) notes that permeability of silica fume concretes taken from CSFC placements in Ohio compares favorably with that of other systems including LMC, SDC, high-molecular-weight methacrylates, silane sealers, and epoxy overlays.

In spite of this generally favorable background, questions relating to the performance of both LMC and SDC remain unanswered. The relative effectiveness of each of the bridge overlay systems in forming impermeable barriers to chloride ions needs to be quantified. Although laboratory data may indicate favorable improvements in impermeability through the use of these protective overlay systems, the possibly deleterious effects that field construction variables may have on permeability are not known. Therefore, study of field-produced concretes is vital to answering these questions. Recent reports of high permeabilities exhibited by LSDC overlays (FHWA, unpublished data) and lack of field data on chloride permeability of SDC and CSFC creates a need for more information on these systems. Further work is needed before rigid overlays can be accepted as a long-term solution to the bridge deck deterioration problem.

SAMPLING PLAN AND PROCEDURES

The sampling plan is depicted in Figure 1. A total of 25 field sites were selected; these consisted of 13 in-place sites, 10 placements, and 2 additional sites. Sites selected for study of overlays are located in Figure 2. The majority of the sites lie in a band running roughly from the southwest to northeast corners of the state of Ohio. Two other sites were located in the southeast area of the state near the city of Marietta. In-place overlays ranged in age (at time of sampling) from 1 to 14 years for LMC, from 3 to 5 years for SDC, and less than 1 year for CSFC. All sampling from new construction was carried out during August and September 1987.

The 13 in-place sites were chosen to include 8 in-place LMC overlays and 5 in-place SDC overlays. At each of these sites five cores were taken. Four cores from each deck were tested for rapid chloride permeability by using AASHTO T277-83 (Standard Method of Test for Rapid Determination of the Chloride Permeability of Concrete).

For test preparation, each core was cut at the overlay bond line (or at 2.0 in. in those cases in which overlay depth exceeded 2 in.) so as to include only overlay material in the test.

The remaining core from each site was sectioned at 0.5-in. intervals up to the overlay bond line, and each slice was ana-

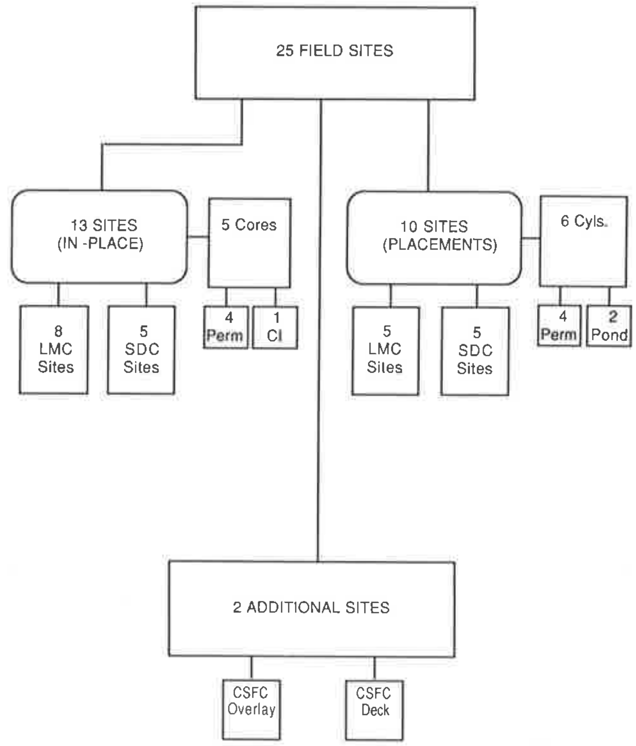

FIGURE 1 Sampling plan.

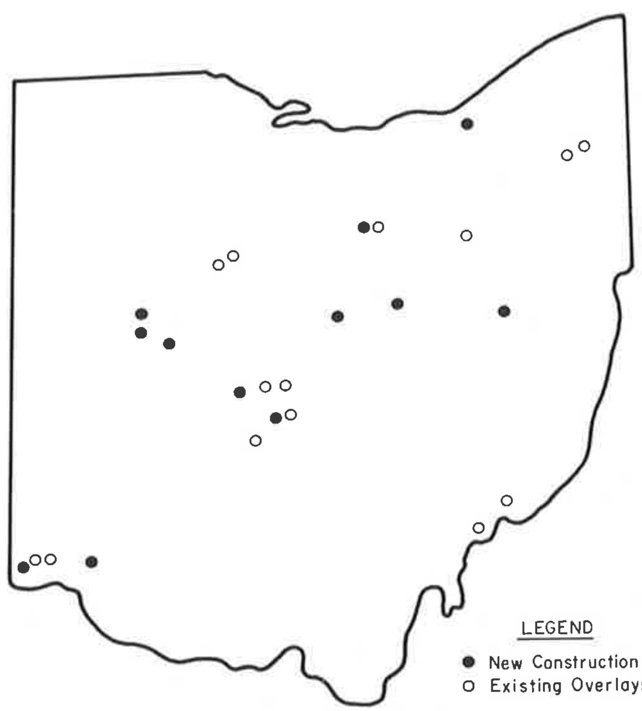

FIGURE 2 Location of sites selected for sampling.

lyzed for total (acid-soluble) chloride ion by procedures described in AASHTO T260-82 (Standard Method of Sampling and Testing for Total Chloride Ion in Concrete and Concrete Raw Materials).

The ten placements were evenly divided between five LMC and five SDC sites. (Note: at one of the LMC sites shown in

Figure 2, samples were obtained from two separate pours. These are denoted as J and K in subsequent discussions of results.) At each site a total of six 4-in.-diameter by 8-in.-long cylinders were cast by using rigid plastic molds. In most cases, this represented three cylinders cast from each of two trucks (or concrete mobiles) selected for sampling. Cylinders were initially cured on site under a covering of wet burlap and polyethylene sheeting. After completion of initial moist curing (72 hours for SDC and CSFC; 48 hours for LMC) cylinders were transported to laboratory facilities and curing was continued (moist cure for SDC and CSFC; air cure for LMC) until the specimens had reached an age of 5 weeks, at which time testing was initiated. From each set of three cylinders, two were tested by AASHTO T277 and one was subjected to a saline ponding procedure similar to that described in AASHTO T259-80 (Standard Method of Test for Resistance of Concrete to Chloride Ion Penetration).

The two additional sites shown in Figure 2 represent an in-place condensed silica fume overlay and condensed silica fume placement of a full-depth bridge deck. Procedures used for these sites were identical to those used for the other in-place sites and placements.

RESULTS AND DISCUSSION

In-Place Sites

Chloride contents of cores obtained from LMC and SDC overlays are shown in Figure 3. There is a wide range of values in each set of data. Values in the near-surface (0 to 0.5-in.) layer for LMC range from 0.05 percent (core 6-2) to 0.40 percent (core 12-2). In some cores, such as 12-2 and 14-3, there is a sharp decrease in chloride content with depth. In other cores, such as 11-2 and 7-2, the decrease is much more gradual. For the most part, high chloride contents are associated with those LMC overlays that have been in service for the longest period of time: e.g., overlays at site numbers 2 and 12 have been in service for 12 and 14 years, respectively.

Chloride profiles for cores taken from SDC overlays are similar in many respects to those from LMC overlays. Near-surface values range from 0.14 percent (core 5-3) to 0.32 percent (core 9-2). There is a relatively sharp decrease of chloride content with depth, except for core 9-2.

A comparison of chloride contents at the 1.0-in. level for LMC and SDC cores is shown in Figure 4. For these data the effect of age of overlay is readily apparent. With increasing age, chloride contents at 1.0 in.-depth increase for both LMC and SDC overlays. Unfortunately, there are no SDC overlays in Ohio of sufficient age that can be compared to the 12- and 14-year-old latex sites; consequently the question of long-term penetration of chlorides into SDC remains unanswered at this time.

Results of RCPT on in-place core slices are presented in Figure 5. For each site, individual results on each of the four core slices tested are shown. There is a strong effect of overlay age on test results. For the most part, LMC permeability tends to decrease with age. If the high values within site numbers 10 and 11 are excluded, there is a gradual reduction of charge passed from an average of about 900 coulombs at 1 year to about 400 coulombs at 14 years. Most of the results after 1

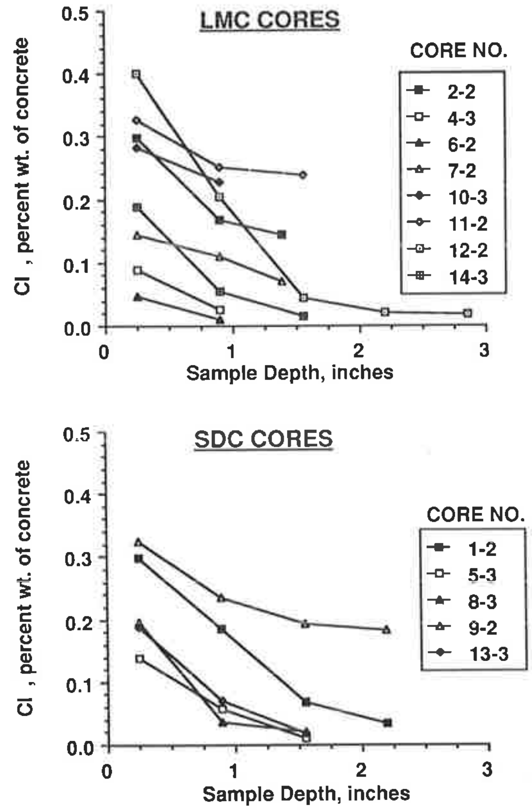

FIGURE 3 Chloride contents of cores obtained from in-place overlays.

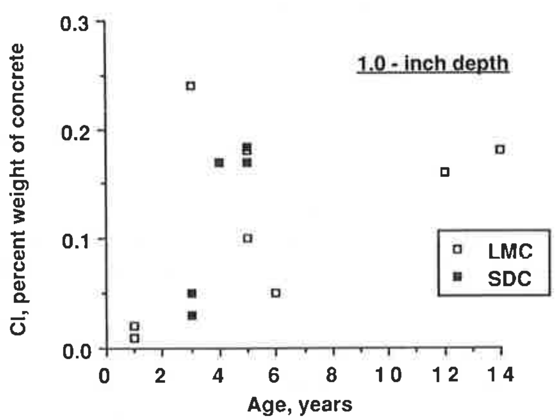

FIGURE 4 Effect of age on chloride content of overlays.

year of age lie below 600 coulombs. This age effect has been previously reported in laboratory studies (*7*, *13*).

For SDC, on the other hand, within the ages from 3 to 5 years, there is little effect on permeability. The range of ages of structures for which cores could be obtained was more limited than for LMC. Values range from approximately 600 coulombs for site 8 to an average of 1,200 coulombs for site 9, placing these SDC overlays within the low to very low categories, as defined in AASHTO T277. Within any given

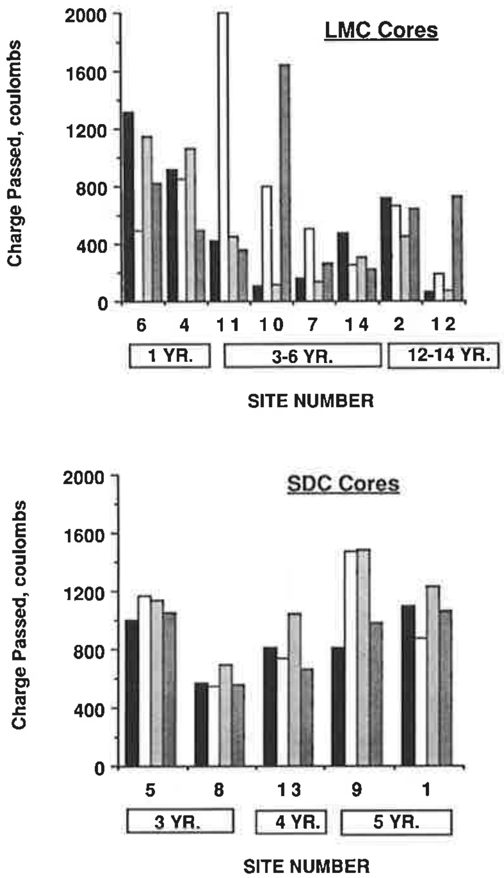

FIGURE 5 Results of rapid chloride permeability testing on samples taken from in-place overlays.

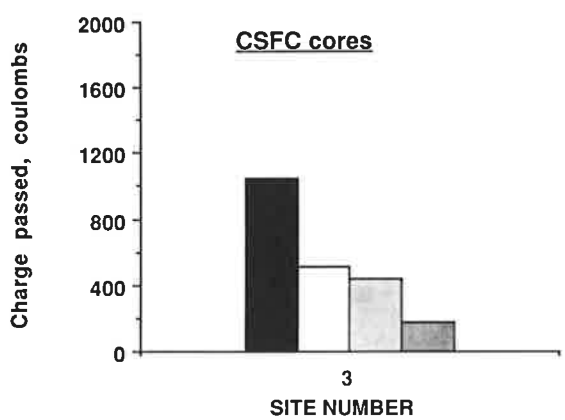

FIGURE 6 Rapid chloride permeability test results on CSFC overlay samples.

set of four SDC cores, results appear to be more uniform than for the LMC cores. For instance, the widest range of results for SDC was exhibited by site 9, where the lowest value was 810 coulombs and the highest was 1,490, representing a difference of 60 percent of the mean value. In comparison, for LMC site 10, highest and lowest values were 1,640 and 110, a difference of 175 percent of the mean value. Likewise, LMC sites 11 and 12 show large differences in charge passed between specimens in the same sets. Results on CSFC specimens, all obtained from site 3, are presented in Figure 6. Even though this overlay had been in place for only 2 months before sampling, rapid chloride permeability values were already lower than all of those for the SDC overlays and were roughly equivalent to most of those for the LMC overlays that had been in place for a number of years. These very low values for chloride permeability are in agreement with data recently presented by Luther (*14*) and Ozyildirim (*15*) on CSFC overlays.

Placements

RCPT results on cylinders cast from overlay placements are presented in Figure 7. Each bar represents the average of two test cylinders. Most of the LMC results (with the exception of site D) lie between 1,000 and about 2,000 coulombs, which is somewhat higher than has been reported in previous laboratory (*7*, *16*, *17*) and field (*18*, *19*) studies, where LMC concretes typically exhibited values lower than 1,000 coulombs. However, it is reasonable to expect that field concretes will exhibit somewhat higher permeabilities than concretes prepared under more controlled laboratory conditions. It is also worth noting that most of the RCPT values reported for field concretes pertain to cores taken at ages considerably greater than the 6-week test period used in the current study.

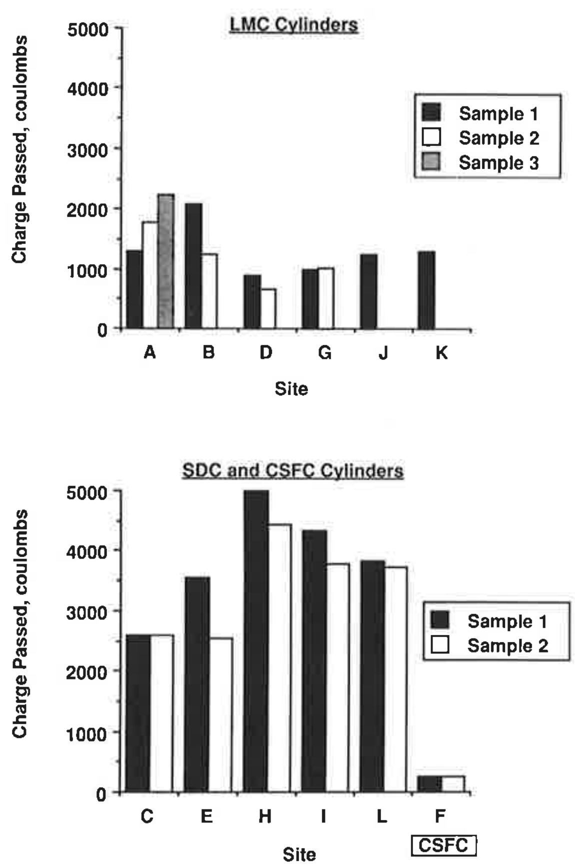

FIGURE 7 Results of rapid chloride permeability tests on cylinders cast at overlay placements.

All SDC test results are considerably greater than results obtained on LMC cylinders. The highest LMC results (obtained on sample 3 from site A and sample 1 from site B) were slightly lower than the lowest SDC results (site C and sample 2 from site E). The average of all LMC RCPT results, 1,290 coulombs, is 60 percent lower than the average of all SDC results, 3,640 coulombs. The SDC results are far above the range of 1,000 to 2,000 coulombs suggested as being typical of RCPT results on low w/c ratio concretes in the original permeability method development study (*2*). However, the current results are in close agreement with values reported by Whiting and Kuhlmann (*7*) for LSDC and SDC mixtures tested at ages of 1 and 2 months. It appears that the low permeability values said to typify LSDC and SDC-type concretes based on the initial RCPT development studies may have been overly optimistic and may be possible only under controlled laboratory conditions.

Results on the CSFC specimens, on the other hand, further demonstrate the low permeability inherent in this material. The very low values of charge passed, in the range of 250 coulombs, are typical of mixtures with the high silica fume loadings (15 percent solids basis) used in the site F deck placement. The CSFC specimens exhibited the lowest values of charge passed among all of the specimens sampled from placements included in this study.

After exposure to 3.0 percent sodium chloride solution for 90 days, companion cylinders obtained from overlay placements were sectioned at 0.5-in. intervals and analyzed for total (acid-soluble) chloride ion. These values were then graphically integrated over the sampling depths to obtain the "integral" chloride values (*2*) previously used in development of the RCPT. Results are presented in Figure 8. In general, 90-day ponding results for the SDC set lie somewhat above those for LMC. Mean value for total integral chloride for all SDC samples was 0.124, compared with 0.097 for the LMC. This represents a difference of a factor of 1.3. Statistical treatment of the two data sets indicates a significant difference (i.e., lower LMC values) at the 99 percent confidence level. As was the case for the RCPT results, CSFC overlay concretes show much lower values for total integral chloride than values obtained for either SDC or LMC.

In previous laboratory studies (*2*, *20*) fairly good relationships between results of RCPT and 90-day chloride ponding tests were established. There is little published information, however, on these relationships with respect to field concrete samples. To obtain such information on LMC, SDC, and CSFC samples, linear regression analyses were carried out on the RCPT and 90-day ponding data, by using RCPT as the independent and 90-day ponding as the dependent variable. The resultant relationship is shown in Figure 9. Considering the variability that can exist in field concrete samples, the results are encouraging. There is a gradual increase of total integral chloride values (90-day results) with an increase in charge passed (RCPT). For the most part, SDC samples, as a group, exhibit higher values of charge passed and correspondingly higher total integral chloride values than do LMC or CSFC samples. The relationship is expressed in the following form:

$$Y = 0.0704 + 1.53 \times 10^{-5}X$$

where Y equals total integral chloride and X equals charge passed in coulombs.

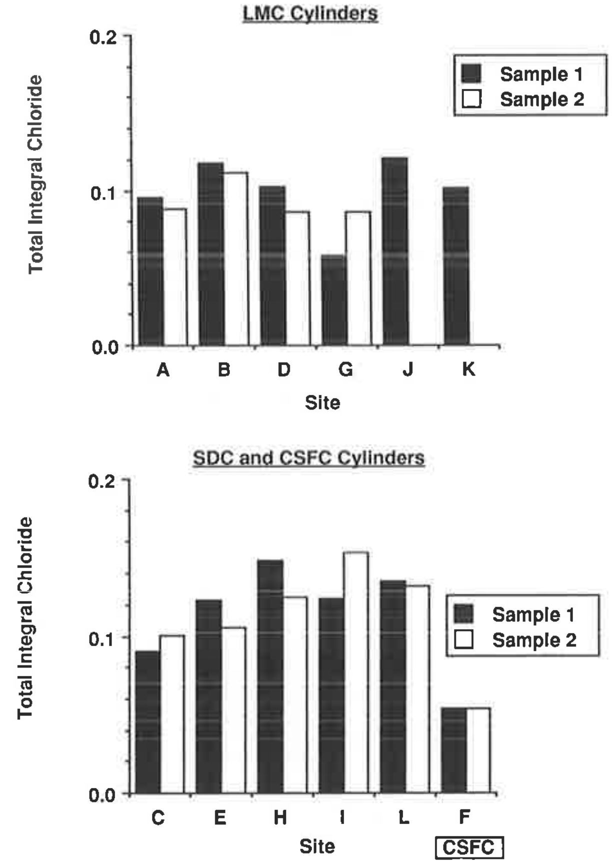

FIGURE 8 Total integral chloride contents of cylinders cast at overlay placements and subjected to 90-day ponding.

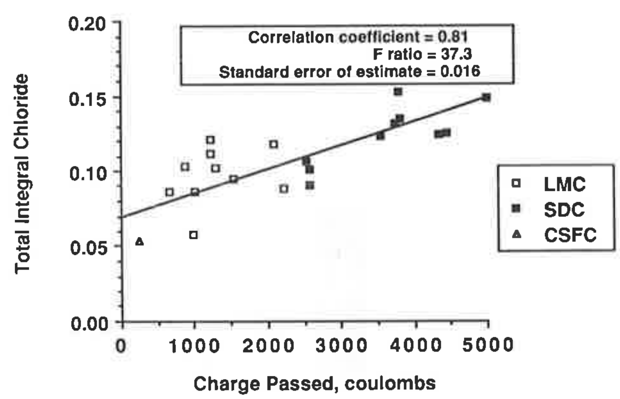

FIGURE 9 Relationship between rapid chloride permeability test results and total integral chloride contents for cylinders taken from placements.

The association between the variables is highly significant (F statistic = 37.3), the correlation coefficient is relatively high (0.81), and the standard error of estimate (0.016), amounts to approximately 16 percent of the mean value of integral chloride. The correlation coefficient for the current study is essentially equivalent to that developed in the initial laboratory study (*2*), and the standard error is less. These data demonstrate that the RCPT can be used as a fairly reliable indicator of the long-term permeability of concrete to chloride ions and that it, therefore, has considerable merit as a rapid quality control tool for this purpose.

CONCLUSIONS

Based on the results of this study the following conclusions may be drawn:

- Rigid concrete overlay systems, such as LMC, SDC, and CSFC can be used to reduce infiltration of chloride ions into reinforced concrete bridge decks with varying degrees of effectiveness. The most impermeable overlays appear to be those incorporating condensed silica fume concrete, although long-term data with which to substantiate this conclusion are, as yet, not developed.
- Latex-modified concrete represents the next level of relative permeability. Initial permeabilities typically are in the low to very low category and decrease substantially over a period of years. Variability across a structure, however, can be high, and substantial amounts of chloride can migrate into LMC given sufficient time and exposure to severe conditions.
- Superplasticized dense concrete overlays appear to be somewhat more permeable than their LMC or CSFC counterparts. Initial permeabilities are generally in the moderate range, and high permeability values have been encountered. Over a period of years, it appears that permeability of SDC will decrease to low values.
- The rapid chloride permeability test (AASHTO T277) shows good potential for use as a means of establishing relative effectiveness of rigid overlay materials. The test may be carried out on test cylinders prepared at the jobsite and agrees well with more time-consuming procedures, such as 90-day salt ponding.

ACKNOWLEDGMENTS

This research was sponsored by the Ohio Department of Transportation (ODOT) and the FHWA. The authors acknowledge the assistance of Keith Keeran, ODOT Construction Bureau, who served as project monitor for this research. They also thank Dennis Bunke and his staff at the ODOT Bureau of Testing, for their assistance in coordination of sampling and testing. Finally, the authors thank the many ODOT district engineers and inspection staff who assisted the investigators during the field studies.

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Iowa Flowable Mortar Saves Bridges and Culverts

WILLIAM E. BUSS

The development and use of flowable mortar for various applications are presented. This research used various methods of backfilling pipe culverts to compare settlement and cost of backfill methods. The research resulted in a method of backfilling culverts with sand backfill to midheight of the culvert and then with a maximum of 5 ft of flowable mortar. The remainder of the backfill is normal embankment or other subgrade treatment. This method was expanded to include installing culverts under bridges and then backfilling under the bridge with sand and flowable mortar in two stages; this method resulted in the bridge check becoming the pavement by removing the bridge curb and handrail. Another use of flowable mortar involves placing a pipe culvert in a deteriorated box culvert and filling the void with flowable mortar. The Iowa Department of Transportation has backfilled an "H" pile and wood plank retaining wall with flowable mortar. The present flowable mortar consists of 2,600 lb of sand, 100 lb of cement, 300 lb of fly ash, and water, which results in a 12-second efflux time for the Corps of Engineers flow cone method CRD-C611-80, approximately 70 gal. The Ames Central Office Materials Laboratory uses submitted sand, fly ash, and cement to develop the desired efflux time. The fly ash content may be increased if the sand is too coarse. Sand gradations are 100 percent passing ¾-in. sieve and 0 to 10 percent passing 200 sieve.

DEVELOPMENT

In 1979 the Office of Materials of the Iowa Department of Transportation, under the direction of Ralph Britson, developed a flowable mortar backfill material consisting of 212 lb of Type 1 Portland cement, 505 lb of Type F fly ash, 2,232 lb of sand, and 438 lb of water. By using this material, granular backfill, and soil backfill, a comparison was set up within a project in which nine culverts were being replaced.

Flowable mortar was used to backfill under the pavement by using the soil outside a 1:1 slope as a form to contain the flowable mortar. Granular backfill was used in the same area as the flowable mortar for other locations. Still other locations were backfilled with soil in 8-in. lifts and compacted with a vibratory plate. The methods used are compared as follows:

Type of Backfill	*Settlement (in.)*	*Cost ($)*
Soil	1.5	1,790
Granular	0	1,880
Flowable mortar	0.25	790

More testing resulted in a flowable mortar mix consisting of 100 lb of cement, 300 lb of fly ash, 2,600 lb of sand, and ±70 gal of water to produce a 12-second efflux time by the Corps of Engineers flow cone method CRD-C611-80. The aggregate gradations are 100 percent passing the 3/4-in. sieve and 0 to 10 percent passing the 200 sieve. The best results are with fine sands. Fine aggregate for concrete will work with 400 to 500 lb of fly ash because of the small amount of material that passes the 200 sieve (0 to 1.5 percent). This material will develop a compressive strength of 80 psi but is designed for backfill material that does not shrink. Flowable mortar can be removed without a jackhammer.

The following uses of flowable mortar have been designed with the cooperation of Kermit Dirks, geologist in the Office of Road Design.

CULVERT BACKFILL

Flowable mortar was designed to be used for culvert backfill with the following requirements: granular backfill for half the height of the culvert and flowable mortar for a maximum of 5 ft above the culvert. The granular backfill provides insurance that the culvert does not float and acts as a filter to keep from plugging the drainage system. The required subgrade treatment must be between the pavement and the flowable mortar. The sand and flowable mortar are contained by soil as shown in Figures 1 and 2.

This type of construction has been used on roads closed to traffic and also under traffic in which one lane is closed. For the one-lane closure method, half the pavement is removed with a bulkhead of sheet piling installed to retain soil under the open lane. The culvert is installed and backfilled with sand, flowable mortar, soil, or granular material, and the

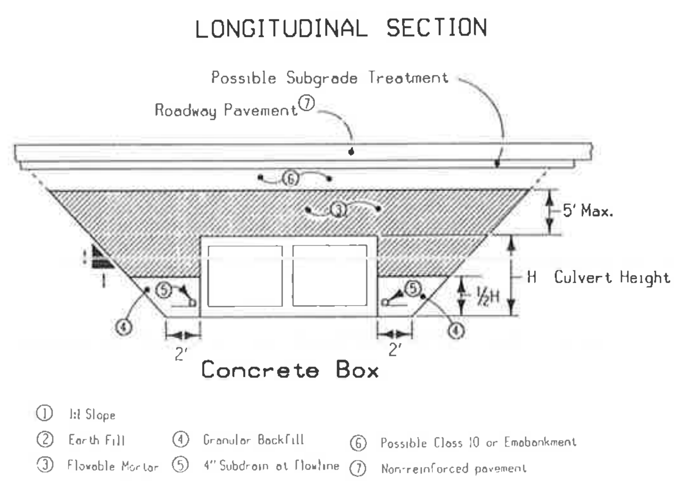

FIGURE 1 Side view of culvert backfill.

Construction Office, Iowa Department of Transportation, Ames, Iowa 50010.

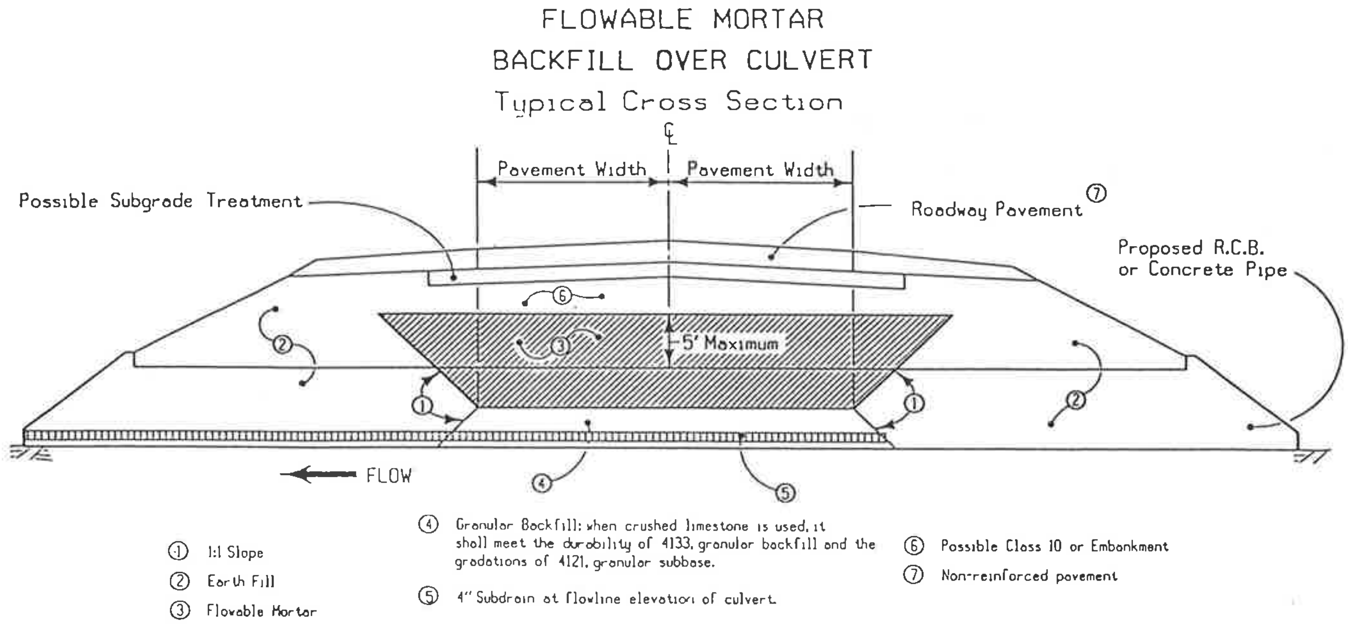

FIGURE 2 Cross-section of culvert backfill.

FIGURE 3 Overall view.

FIGURE 4 Excavation view.

pavement is replaced. The remaining half is constructed by the same procedure (Figures 3 and 4).

In locations in which a culvert is badly deteriorated, a corrugated metal pipe culvert is placed inside the deteriorated culvert and the void between the two culverts is filled with flowable mortar. If possible, granular backfill to midheight is placed before placing flowable mortar. If space is not available, the pipe culvert is blocked into position and backfilled in stages to prevent the floating of the culvert. For long culverts the flowable mortar is introduced at more than one location.

When it is to be abandoned in place, a culvert is filled with flowable mortar after sealing off both ends of the culvert. Sealing it off also fills any voids adjacent to the abandoned culvert to prevent future settlement.

BRIDGE BACKFILL

Where drainage conditions will allow and where space is adequate, Iowa has developed a system that uses flowable mortar to convert the bridge to a roadway without closing the road to traffic. (See Figure 5 for before and after views of a bridge backfill.) This process involves installing the culvert, pipes, or reinforced box culverts under the bridge. The culvert is backfilled by using soil as forms at both ends and filling the area under the bridge with flowable mortar (Figures 6 and 7).

The first stage is filled to within 6 in. of the low member of the bridge. The second stage is placed after a 72-hour delay to allow for the fast settlement of the original soil under the bridge. The second stage is placed sequentially over half of

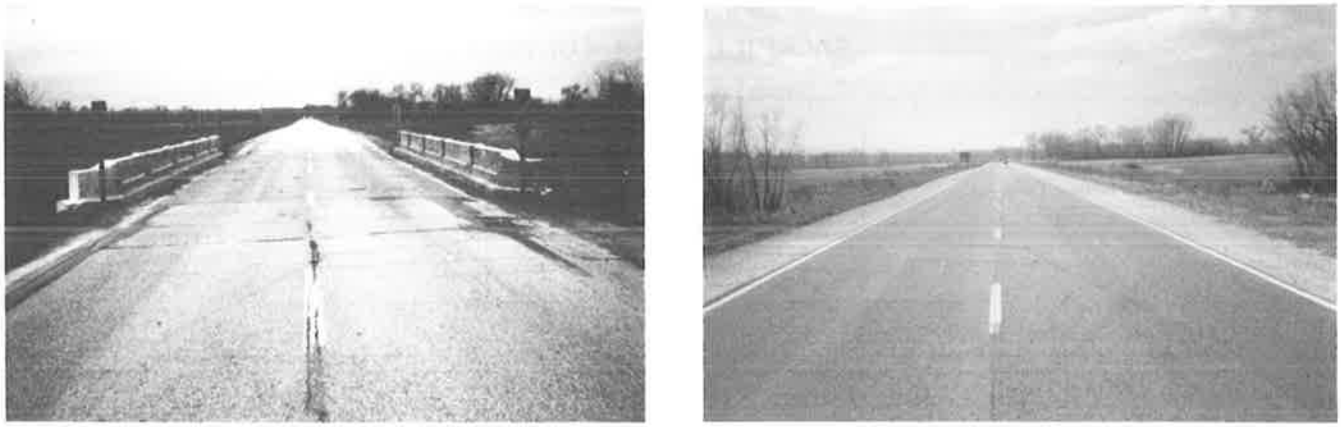

FIGURE 5 Before (left) and after (right) views of a bridge backfill.

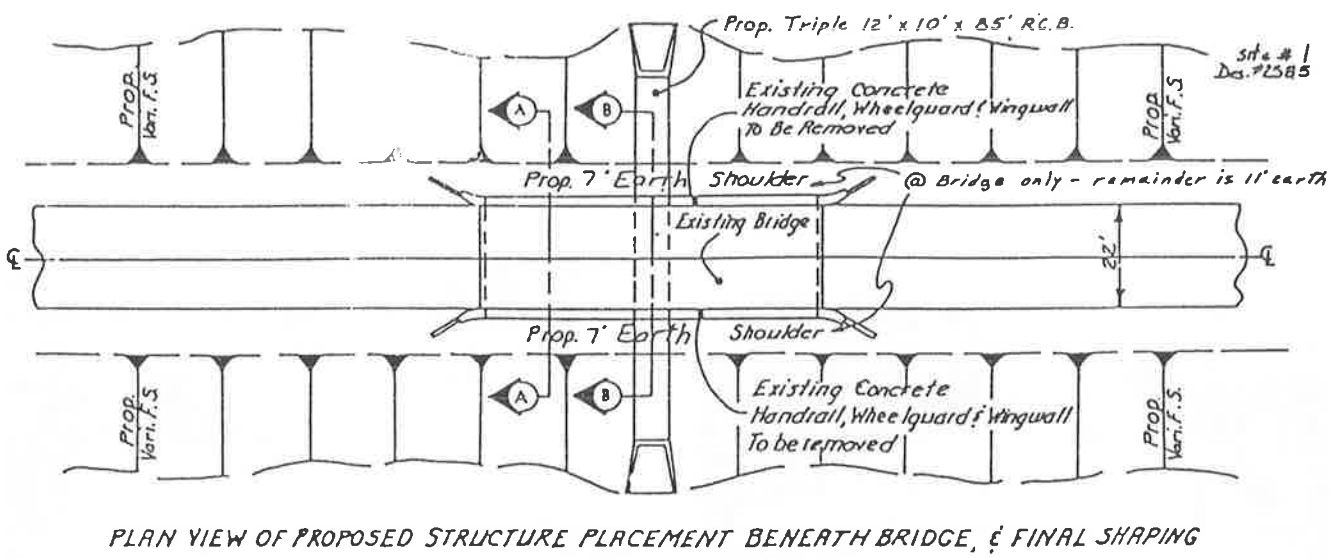

FIGURE 6 Plan view of bridge replacement.

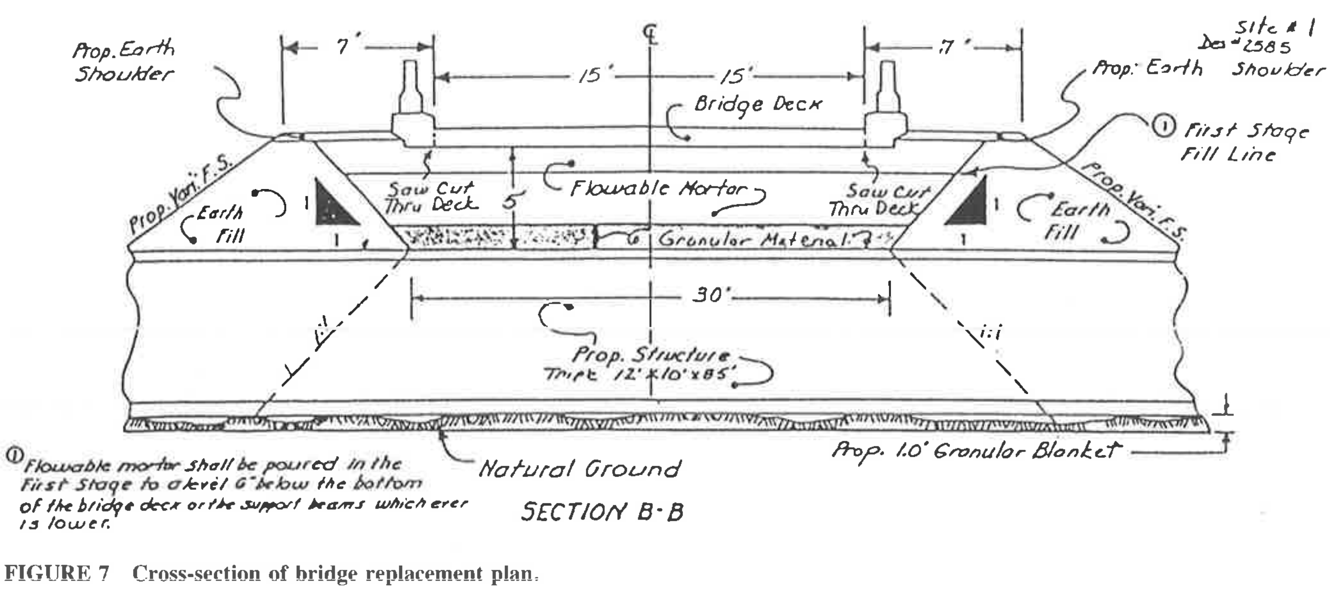

FIGURE 7 Cross-section of bridge replacement plan.

the deck at a time through holes drilled into the floor between beams, abutments, piers, and diaphragms.

The bridge handrail and curb are then removed (see Figure 8). With the construction of shoulders in the bridge area, the bridge becomes a pavement. The culvert is constructed with a limited amount of one-way traffic (see Figure 9). The placement of flowable mortar involves one-way traffic at least during the second stage.

In a 9-mile length of U.S. Route 30 between Woodbine and Logan in Harrison County, Iowa, there were six narrow bridges. Four of these bridges were modified by using flowable mortar with concrete pipe culverts. One was modified by using flowable mortar with a reinforced concrete box culvert, and one was replaced with a new bridge by using a runaround to keep the road open to traffic. A cost comparison is shown as follows:

Type of Work	*Cost ($)*
Pipe culvert installations	50,500
RCB culvert	140,000
Bridge replacement	207,000

At several locations in which a railroad passes over a highway, an at-grade railroad crossing can be safely constructed because of reduced train traffic. In this situation, the railroad underpass was backfilled with sand and flowable mortar by a method similar to the previous method, except that the flowable mortar is placed at one time (see Figure 10).

BACKFILL RETAINING WALL

At one location in Iowa, a laid-up limestone block retaining wall bulged out, indicating the initial stage of failure. In this

FIGURE 8 Removal of bridge handrail.

FIGURE 9 Traffic during second stage.

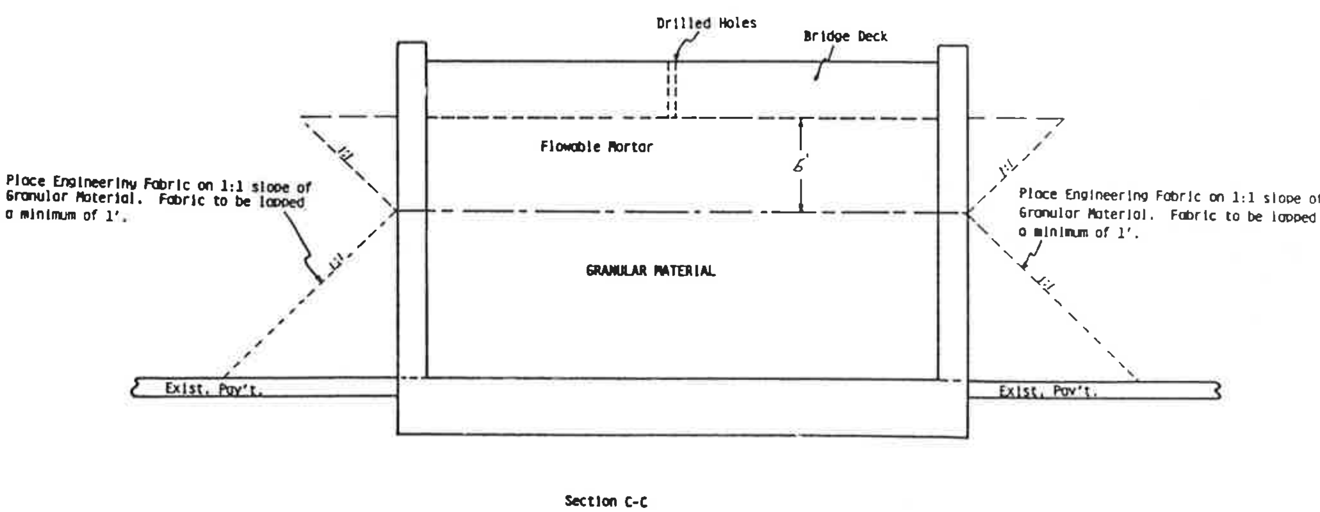

FIGURE 10 Profile through underpass.

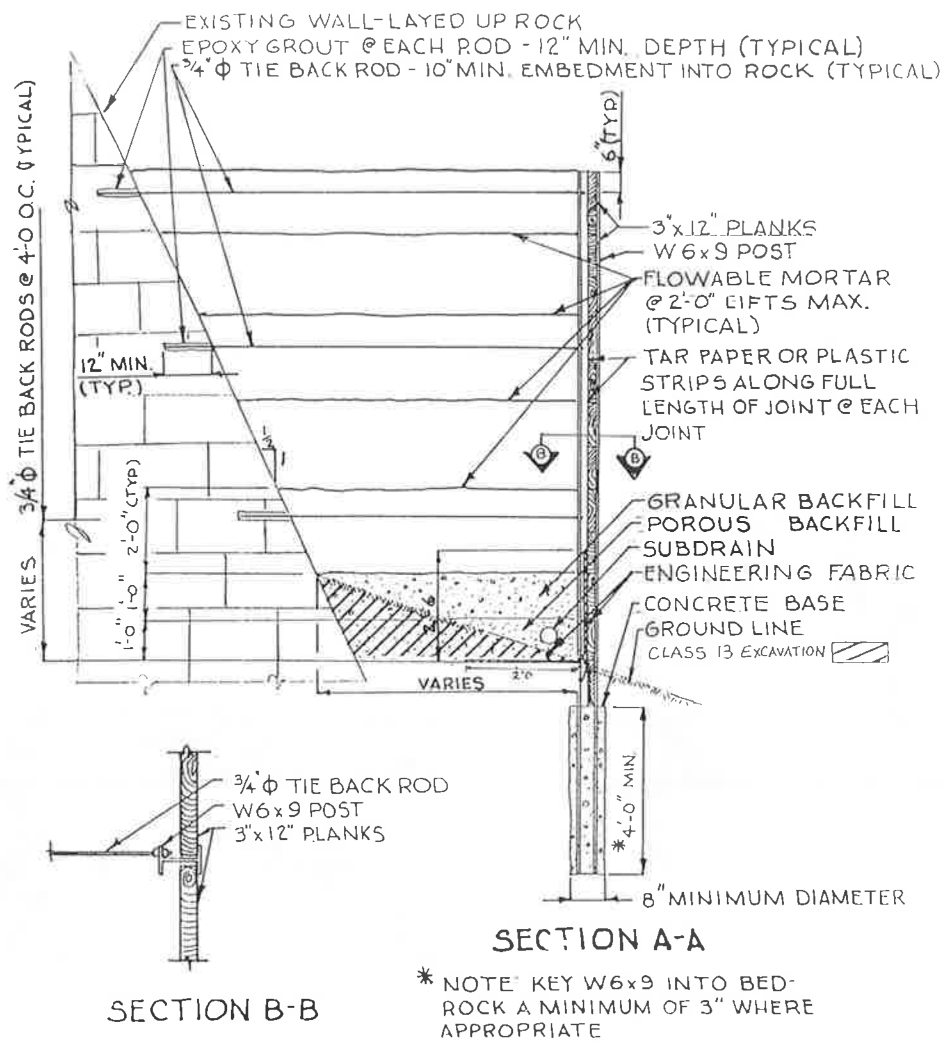

FIGURE 11 Cross-section of retaining wall.

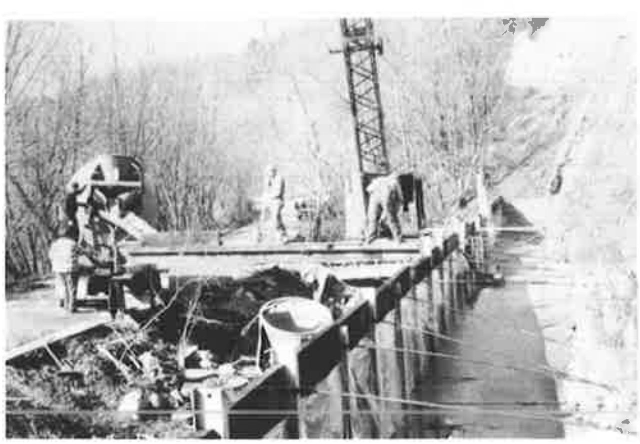

FIGURE 12 Actual placement of flowable mortar.

instance an "H" pile wall with wood planks was constructed in front of the wall and backfilled with sand and flowable mortar, which filled voids behind the limestone block wall (see Figures 11 and 12).

IMPORTANT POINTS

The following are important to remember:

- Pipe culverts will float.
- Bulkheads are designed for liquid pressure.
- Water content is critical: too much is as bad as too little.
- Delivery should be continuous for filling confined spaces.
- Drainage of water through the flowable mortar is a must.

SUPPLEMENTAL SPECIFICATION

The Supplemental Specification for Flowable Mortar (SS-1069) dated December 20, 1988, is available from the author.

The contents of this report reflect the view of the authors and do not necessarily reflect the official views of the Iowa Department of Transportation. This report does not constitute a standard, specification, or regulation.

Publication of this paper sponsored by Committee on Mechanical Properties of Concrete.

Use of Coarse Aggregate in Controlled Low-Strength Materials

THOMAS A. FOX

Controlled low-strength materials (CLSM) have evolved using only sand as aggregate filler. CLSM in the Pacific Northwest has evolved differently in that many mixtures use gravel up to 1 in. top size. The reasons for the use of gravel center around economy and performance. Concrete technology teaches that the largest top-size aggregate that can be used yields the lowest voids in the combined aggregates. Reduced voids result in a lower paste requirement, which reduces the cost for cementitious materials.

Controlled low-strength materials (CLSM) have gained appreciable recognition as fill materials because of their many inherent advantages. These advantages include flowing placement without segregation, self-consolidation, controlled density, controlled strength, ease of excavation, and economy. In recognition of the increasing use of CLSM, The American Concrete Institute (ACI) has enforced committee 229 to develop a state-of-the-art document of CLSM.

Development of CLSM has centered around mixtures using a sand filler. The interparticle voids are then slightly overfilled with a fluid paste composed of cement, fly ash, water, and occasionally additives. Components of the paste are varied in quantity to achieve the required performance in terms of strength development, self-consolidation, flow behavior characteristics, durability, economy, and ease of removal (*1*). Gravel generally has not been a component of the aggregate filler probably because of economical reasons. Throughout the country, gravel is usually at a premium while sand is generally in surplus. K-Krete, Flowable Mortar, and earlier cement-sand slurries are typical of CLSMs that use only sand as a filler.

Sand filler mixtures have performed well, giving all the properties desired. It may be difficult to achieve satisfactory flowability and there may be severe bleeding in cement-sand slurries. Bernard and Tansley reported that fly ash can be of assistance in obtaining properties of flowability with reduced bleeding (*1*).

Exceptions to sand-filled CLSM are foamed mixtures, flowable fly ash, lean concrete mixtures, and sand-gravel–filled mixtures. Foamed CLSM, rich in cementitious material fines, do not need or want aggregate fillers. Flowable fly ash capitalizes on abundant, inexpensive fly ash to act as a paste component and as a filler. Lean concrete mixtures are typically specified at fairly low slump and are not expected to have good flowability. Gravel-sand–filled CLSM can be proportioned to achieve all the necessary performance features, in which suitable gravel economy exists.

Economy, given satisfactory performance, is the key factor to the viability of CLSM. The greater the economy, the greater the applications for its use. Mother nature blessed many areas of the west with equivalent or greater supplies of gravel compared with sands. The use of gravel in CLSM can assist in gaining economy where sands are premium or equivalent in cost. Initial economy is achieved through the use of a lower-cost material, whereas secondary economy can be achieved because of lowered paste requirements from reduced voids with increasing maximum aggregate size.

Pozzolanic International, 2448 76th Ave., S.E., Mercer Island, Wash. 98040.

PRODUCTION, PERFORMANCE, AND ECONOMY CONSIDERATIONS

Production of CLSM usually must fit within the confines of normal concrete production operations. Normal concrete operations contain bunker and silo space only for specification materials used routinely in the production of concrete. The use of materials not normal to the concrete production operation requires either additional bunker and or silo space or a plant dedicated to the production of CLSM. Consequently, nonstandard aggregates that could well be used in CLSM, generally are not used because of a lack of plant capacity.

Since concrete plants usually use standard concrete materials, the objective is to proportion CLSM of suitable performance at lowest cost using available materials. With cement costing \$50 to \$80 per ton and fly ash costing \$30 to \$45 per ton, it is economically important to minimize their use while maintaining product quality. Premium prices for sand or surplus of gravel make it advisable to use as much gravel as possible.

PROPORTIONING

When proportioning CLSM with gravel, standard concrete porportioning techniques apply since the material so closely resembles weak concrete. The steps taken for proportioning can basically be taken from ACI 211, section 5 (*2*), modified for CLSM: (a) selection of slump range; (b) selection of maximum aggregate size; (c) selection of cement content; (d) selection of fly ash content; (e) estimating water content; (f) selection of entrained air content; and (g) determination of aggregate content.

Selection of Slumps

As in concrete, slump provides a measure of consistency defined in ACI 116 (*3*) as, "the relative mobility or ability of freshly mixed concrete or mortar to flow." CLSM generally will be more flowable than concrete at the same slump primarily because of the lubricating action of the high volume of fly ash spheres. General fill applications require a slump range of 6 to 8 in. Where greater flow is required, care should be taken to ensure that adequate fines (generally fly ash) are present to accept the flow without segregation. CLSM of low slump (2 in.) has been successfully delivered by concrete pump without the need for added consolidation effort (sub-footing fill, Valley General Hospital, Kent, Washington).

Selection of Maximum Aggregate Size

Similar to concrete, selections should allow for the largest practical size commensurate with the intended application. Most work with gravel in CLSM has been done in the sizes from ⅜ in. to 1 in., although larger sizes could be used under permissible conditions.

Selection of Cement Content

Selection of cement content is based on required compressive strength and unfortunately does not follow the water to cement (w/c) law. Actual requirements must be determined by trial. The following table has given good results in determining the requirements:

Cement Content (lb/yd³)	*CLSM 28-day Compressive Strength (psi)*
40–50	100
70–80	200
90–110	400
110–150	500

Where excavation is required, CLSM strength should be limited to 150 psi maximum.

Selection of Fly Ash Content

Fly ash is used in CLSM as a void filler and fluidifying agent more than for strength production. Fly ash contents as low as 100 lb/yd³ have been used with an equivalent amount of cement for placement directly from the truck chute when flowability was not required. Where average flowability is required, 250 lb of fly ash is generally used with at least 40 lb of cement. Where great flowability or pumpability is required, fly ash contents may reach 1,000 lb/yd³.

Class C fly ash may produce strengths higher than is wanted. Thorough testing should be done to determine the advisability of using class C fly ash.

Estimating Water Content

The high fly ash-low cement contents of these mixtures provide for high slump with low water contents relative to what can be expected from concrete. We have found that switching a plain cement 5-sack concrete mix at a 4-in. slump to a mixture using 50 lb of cement and 250 lb of fly ash with the same aggregates and water gives a CLSM with a 7- to 8-in. slump. Less water is required for a change of slump with CLSM than can be expected for concrete. Where 1 gal of water changes concrete slump 1 in., consider about 0.5 gal for CLSM.

Increasing the fly ash content has a dramatic effect on reducing water demand for a given slump. Trials run by Pozzolanic on a mixture with 50 lb of cement and equal portions of pea gravel and building sand showed an 11-lb reduction in water for each 100 lb of fly ash added. This value held true in the fly ash range of 100 to 500 lb/yd³.

The use of entrained air can be expected to have a water-reducing effect similar to that in concrete.

Water demand prediction techniques, such as the loose fine aggregate voids method by Willis (*4*), may be useful in determining initial starting points.

Selection of Entrained Air Content

The use of entrained air is not mandatory and is not recommended when using variable, high-carbon fly ash because of the technical control effort required. At the low compressive strength levels used in CLSM, it is doubtful that entrained air would contribute significantly to durability; however, it may have a beneficial place. Because it occupies volume, entrained air replaces more costly aggregates to provide additional economy. Unit weight is reduced, which may be an important factor in certain fill situations. Entrained air also has the capacity to promote cohesion and reduce water content as it does in concrete mixtures.

Entrapped air contents in CLSM are similar to those of concrete.

Selection of Aggregate Content

Selection of aggregates should follow conventional practice for concrete proportioning. The *b/bo* technique proposed by Goldberg and Gray (5) first published in 1942 and adopted by ACI 211 (Section 5.3.6) provides a good basis for gravel content determination. With the high fly ash contents used, *b/bo* values can be increased to accommodate larger gravel contents without adversely affecting performance. Sand can then be calculated as the volume remaining after cement, fly ash, gravel, water, and air.

EXAMPLE OF CLSM PROPORTIONS WITH GRAVEL

The following data give a representation of CLSM with and without entrained air in which gravel content water and demand are approximated from an existing 5-sack plain concrete mix. These data could be obtained from other methods as well as from practical experience.

Required

- Compressive strength ≤ 150 psi at 28 days (excavatable);

TABLE 1 EXAMPLE OF CSLM WITH AND WITHOUT ENTRAINED AIR

	Plain Concrete		CLSM			
			Non-Air Entrained		Air Entrained	
	Weight SSD (lb)	Volume (ft^3)	Weight SSD (lb)	Volume (ft^3)	Weight SSD (lb)	Volume (ft^3)
Cement: Type I/II	470	2.39	50	0.25	50	0.25
Fly Ash: class F	—	—	250	1.82	250	1.82
1″–4# Gravel	1,900	11.36	1,900	11.36	1,900	11.36
4#–0 Sand	1,400	8.51	1,454	8.83	1,340	8.13
Water	270	4.33	270	4.33	255	4.09
Air (%)	1.5	0.41	1.5	0.41	5	1.35
Total	4,040	27.00	3,924	27.00	3,795	27.00

- Slump range 6–8 in.;
- General flowability;
- With and without entrained air.

Materials

- 1-in. No. 4 gravel, specific gravity, 2.68;
- No. 4–0 sand; specific gravity, 2.64; fineness modulus (FM) 2.80;
- Fly ash (class F), specific gravity, 2.20; loss on ignition (LOI), 0.2 percent;
- Cement (Types I or II).

Procedure

- Water demand from concrete shown with a normal slump of 4 in.;
- Fly ash content for general flowability, 250 lb/yd^3;
- Cement content, 50 lb/yd^3 for $\leq$ 150 psi;
- Gravel 1,900 lb/yd^3 as in the concrete;
- Entrained air, none and total air, 5 percent.

The following example gives a representation of CLSM with and without entrained air (see Table 1). Water demand and gravel content are approximated from an existing 5-sack concrete mix. Data for water demand and gravel content could be obtained by other methods and practical experience.

Required

- A flowable, cohesive, nonsegregated CLSM for street-cut backfill, placed directly from the concrete truck chute. Must be excavatable.
- General flowability;
- Slump range, 6–8 in.;
- Strength $\leq$ 150 psi, therefore ~100 psi @ 28 days;
- With and without entrained air.

Materials

- 1-in. No. 4 gravel; G_s = 2.68;
- No. 4–0 sand; G_s = 2.64; FM = 2.80;
- Fly ash (class *F*); G_s = 2.20; LOI = 0.2;
- Cement (type I or II).

Procedure

- Water demand for 6- to 8-in. slump from reference concrete;
- Gravel content from reference concrete;
- Fly ash content = 250 lb/yd^3 (general flowability);
- Cement content = 50 lb/yd^3 (~100 psi @ 28 days);
- Entrained air, none and total air, 5 percent.

ENGINEERING PROPERTIES

CLSM with gravel behaves similarly to mixtures that use only a sand filler in terms of compressive strength, erosion, flow, permeability, and excavatability. Subsidence may be one area in which gravel mixtures perform better than those with sand only. This better performance is probably because of the reduced water contents of the gravel mixtures, as consolidation occurs by water leaving the mass.

Filling the 12-ft-diameter by 120-ft-deep exploratory shaft and 10-ft-diameter by 30-ft-long subsurface tunnels for the Mount Baker Ridge Tunnel in Seattle, Washington, was accomplished by using a CLSM with ⅞-in. top-size gravel. The calculated fill volume was approximately 800 yd^3 with filling to refusal reached at 786 yd^3 of CLSM, discharged directly into the shaft from concrete truck chutes in 4 hr. Oliver Harding, project engineer for the Washington Department of Transportation, reported a subsidence of ⅛ in. in the 120-ft depth (personal communication, 1984). Subsequent excavation operations resulted in a subsidence of several inches in the hill through which the tunnel passes. The subsidence of the hill occurred without subsidence of the CLSM-filled shaft, which now extends several inches above ground level.

SUMMARY

Whereas CLSM historically has used only sand as a filler, gravel is indeed a viable material for use as a filler. Economics will likely determine whether gravel will be used. Proportioning of gravel CLSM can be accommodated by current concrete proportioning practices such as those in ACI 211. Performance of the gravel mixtures can be expected to be similar to those made with sand only. Subsidence may be reduced as a result of low-mix water requirement.

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Development of a Contract Quality Assurance Program Within the Department of Transportation

CHERYL LYNN

To assure the quality of construction products and processes, the Virginia Department of Transportation has established three levels of construction control. First, contractors themselves provide oversight and quality control as set out in the Department's Road and Bridge Specifications and in their contract stipulations. Second, the Department's construction inspectors provide quality acceptance by determining whether contractors are adhering to specifications and either accepting or rejecting the work in progress. Finally, the Department's quality assurance program examines the methods for inspecting projects and determines where new procedures are necessary to keep the construction process under departmental control. In 1987, the quality assurance effort for highway construction in Virginia was redesigned and retitled the Contract Quality Assurance Program, and steps were taken to put the program on a sound statistical footing and to improve its reliability among field personnel. Sample size requirements were calculated, and a stratified random sampling plan was instituted. A list of inspectable items was developed and prioritized to assist inspectors in managing their time and to ensure agreement on what field inspection entailed. New reporting procedures were developed to change the focus of the program from "inspecting inspectors" to evaluating the inspection process, thus removing the punitive aspects of the previous program.

The Virginia Department of Transportation (the Department) historically has been both active and diligent in its efforts to ensure that high-quality materials and efficient processes are used in the construction of its facilities. To ensure the quality of materials and workmanship, the Department has employed for many years a large complement of field inspection forces. At its inception, the Department's field inspection program provided almost all the necessary engineering services to contractors and, although it no longer provides them, it still provides comprehensive oversight of all construction and most maintenance activities. In 1965, as part of an evolutionary change in most roles for both the Department and the FHWA, the Department began its Inspection-In-Depth program (IID) in an effort to assume some of the responsibility for the construction inspection oversight previously handled by federal employees. In the most general sense, the purpose of both basic inspection and IID was to ensure the accuracy, adequacy, and effectiveness of procedures, methods, controls, and operations used by the contractor and the Department related to the construction of highway facilities (Procedures Memorandum, Management Services Division, January 1, 1977).

In 1986, a study of the statistical adequacy of the 20-year-old IID program was begun by the Department's Management Services Division, which is responsible for quality assurance. As a result of the initial study, it was decided that the original conception of the program no longer met all of the Department's needs and that a new Contract Quality Assurance Program (CQAP) should be designed and implemented. The IID's goal of providing careful and efficient oversight of the construction process was applied to the design of CQAP.

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BACKGROUND

Three generally accepted components, or responsibilities, are necessary for meeting the technical and legal requirements in testing and inspection: (a) quality control, (b) quality acceptance, and (c) quality assurance (see Figure 1).

Quality control is the responsibility of the contractor, who must institute procedures to ensure that the results of a project meet certain specified standards (H. Newlon, personal communication, October 1986). Quality acceptance is the responsibility of the buyer (the Department). Quality acceptance addresses both testing (attributes) and inspection (workmanship). The adequacy of a contractor's quality control can be determined by statistical sampling and analyses, particularly for those elements that involve attributes. This level of quality acceptance is, in effect, the second line of defense (assuming that a contractor has adequate quality control). Quality acceptance through inspection of workmanship is, in effect, the first line of defense for workmanship.

Quality assurance generally refers to procedures for validating the effectiveness of the combined quality control-quality acceptance system. This is the component to which the IID program is directed. In effect, this program has become the second line of defense for workmanship, just as testing is the second line of defense in the case of attributes.

These three components of quality monitoring have always been much in evidence on departmental construction sites. Until the formal development of quality control, acceptance, and assurance systems in the 1960s, the Department's project personnel often performed all of the quality acceptance functions and significant portions of the quality control functions. On federal-aid projects, quality assurance functions were provided by the FHWA through random sampling (*1*). Later, this function was performed by the Department for the FHWA. The IID program was established following a reduction in on-site inspections and tests by the FHWA.

Responsibility for the various aspects of quality acceptance

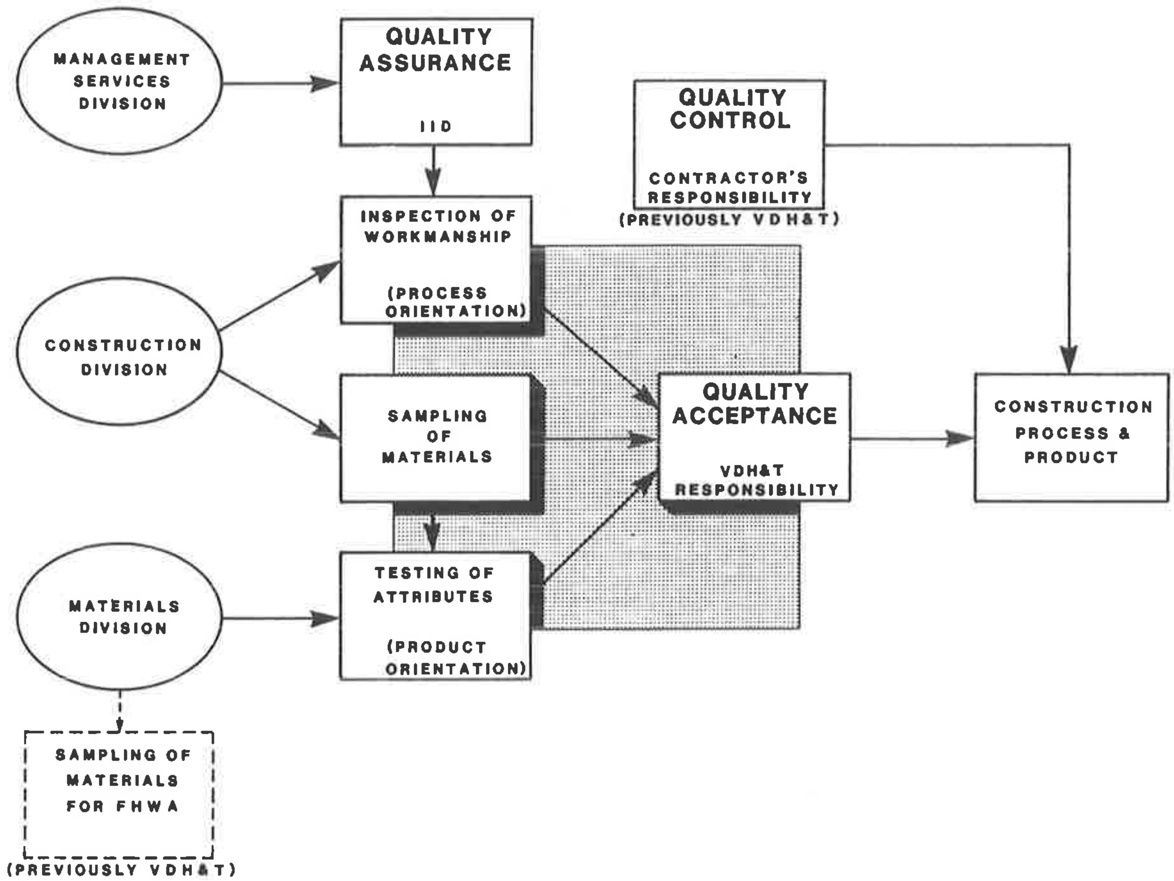

FIGURE 1 Department's quality control, acceptance and assurance system.

and quality assurance is vested in various divisions in the Department. The inspection of workmanship, mainly a process-oriented activity, is under the control of the Construction Division. The construction inspectors also provide samples of materials to the Materials Division, which has the responsibility for testing them to ensure that the materials meet specifications. The IID program, an additional control for both inspection and quality acceptance, operated from 1965 to 1986 under the auspices of Management Services and its predecessor divisions.

For several years before its intensive study of quality assurance, the Management Services Division had been attempting to improve IID site visits and site reports to make them more useful to the Department. Studies of the IID program had identified a number of statistical and procedural problems that had to be rectified before the program could adequately assess the quality of current practices. In an effort to evaluate the statistical accuracy of IID and to explore new uses for the data, both Management Services personnel (Statistical Analysis of the 1985 IID Program. Internal memorandum, Virginia Department of Transportation, 1986) and Turshen and Miller from Virginia Commonwealth University (*2*) conducted studies using the previous years' IID results. They concluded that there were no significant correlations between such variables as number of inspectors per project or dollar value of the project, which had been thought to be related to project complexity, and the number of deficiencies noted by IID inspectors. They also concluded that IID sample sizes were too small to allow for statistical analyses and that the lack of random sampling reduced the extent to which IID results could be generalized. As a result of these studies, in August 1986, Management Services requested the assistance of the Virginia Transportation Research Council (the Council), the Department's research arm, in evaluating and revamping the program.

Initially, the Management Services' request dealt exclusively with IID (quality assurance). As originally intended, the joint effort by Management Services and the Council constituted an attempt to improve the existing quality assurance program by using the same techniques that Management Services had applied successfully elsewhere in the Department. However, in examining the IID program, Council personnel became increasingly convinced that the success of IID was dependent on the success of the Construction Division's inspection program. These two programs were so interrelated that the former could not be adequately evaluated without aspects of the latter being considered. For this reason, it was proposed that the evaluation examine both programs.

The request from Management Services came at an auspicious time. Both IID and the construction inspection program had shown great potential for providing information crucial to the proper management of the construction process. The generation of this information and the monitoring and oversight of the quality acceptance program it represents were especially important in view of the significant expansion that occurred in the state's construction program in 1987 and 1988. Clearly, for the Department (and the state) to be sure that it was getting what it paid for, these new projects had to be carefully monitored to ensure adherence to contract provisions and state specifications. Also, since the Department was committed to getting many of these projects under way very quickly, the success of each inspection activity became even more critical.

The evaluation was timely for several other reasons. Because the new construction effort was the most significant undertaken by the Department in many years, the size of the inspection force had to be greatly increased, and any needed changes in the current inspection program that might be revealed by the evaluation could be initiated before new employees became entrenched in practices that might not be critical to ensuring quality. An analysis of what was to be inspected, how it was to be inspected, and how the data obtained were to be recorded was appropriate. New procedures for evaluating the importance of various construction elements could be developed without as much opposition from the inspection establishment since most of the Department's construction practices were in the process of change and adaptation to the new work load. Thus, IID quality assurance practices and procedures could be evaluated to ensure that they concentrated on items most critical to good inspection.

RATIONALE BEHIND CONTRACT QUALITY ASSURANCE

The original IID program had two separate functions. First, IID was designed to provide quality assurance; that is, inspections were carried out to ensure that the Department's construction inspection process provided the proper oversight (as defined by the Department) of all projects. There seemed to be some disagreement as to whether the IID program was designed to evaluate the performance of the inspection system or the performance of individual inspectors. Because of this disagreement, IID was often seen as adversarial in its relation to inspectors in the field. In addition, IID was originally designed by the FHWA to provide "a thorough on-site review and evaluation of a specific contract item or combination of items which constitute a significant step in the construction process" (*1*). Thus, IID originally provided a more intensive level of inspection since it monitored both the construction and inspection programs.

In terms of intent, there are several similarities between CQAP and the old IID program. The CQAP provides intensive inspection and may assist the on-site inspector in cases in which the contractor is hesitant to correct a deficiency. CQAP also oversees the inspection process as implemented by construction inspectors just as IID did.

However, there are several important differences between IID and CQAP. First, since IID was sometimes misused to pass judgment on individual inspectors or individual field managers, field personnel did not welcome such scrutiny and rarely made use of the results. Emphasis within CQAP is deliberately drawn away from the individual inspector and toward control of the construction process. The purpose of the program is first to determine whether individual components of the construction process meet departmental specifications and, if they do not, whether corrective action is being taken. By highlighting control of the process, CQAP can be used to improve efficiency and evaluate construction activities, specifications, and training. The only instance in which CQAP may be used to judge individuals in the future is if it is eventually used in determining whether individual contractors prequalify to bid on Department construction projects.

A second difference between IID and CQAP is that CQAP is statistically based. Statistically, the number of IID inspections performed each year was relatively small. Thus, little could be said about the efficiency and accuracy of inspections. Also, to be able to allow statements about the inspection program as a whole, the sites inspected would have to be randomly selected (or at least randomly chosen within previously established strata). The method by which projects were chosen in the original IID program was biased toward the selection of projects with substandard performance. This criterion was chosen to ensure that projects with problems received attention since budgeting restrictions limited the number of possible IID inspections. Sample sizes for CQAP were carefully calculated to ensure statistical validity, and sites were selected randomly. Finally, under IID, there seemed to be little explicit agreement between field inspection and IID personnel as to what an inspector's job actually entailed and what constituted good performance on the part of an inspector. The inspection process, then, was not defined comprehensively, and decisions concerning the concentration of effort were left to an inspector's discretion. Since IID might not be measuring what inspectors actually did, IID visits were viewed suspiciously by inspectors. The first step in developing the CQAP involved developing a comprehensive list of inspectable items and activities so that both inspectors and CQAP reviewers would agree on how the construction process should be monitored.

PURPOSE AND SCOPE

The primary purpose of this project was to examine the rationale behind the old IID program and to put the new CQAP on a sound statistical basis. Fulfilling this objective also involved examining the policies and procedures used in administering the program.

To conduct an evaluation of IID, an examination of the current field inspection program was necessary. Thus, a secondary purpose of the study was to evaluate the content of the current inspection program, inasmuch as its content would affect the success of a restructured CQAP program.

METHOD AND RESULTS

Clearly, this project was an ambitious one, requiring the cooperation of many divisions and individuals. In this section, the steps taken to develop and implement the new CQAP are described along with the results of each step. The study was conducted in ten steps:

1. Notification of Department personnel that both an evaluation of the inspection program and the development of a new quality assurance program were being undertaken, and solicitation of input.
2. Development of a comprehensive list of inspectable highway construction items.
3. Prioritization and weighting of inspectable items.
4. Validation of the prioritized list of inspectable items.

5. Development of a statistical sampling plan for the program.
6. Development of computer software to implement checklists.
7. Development of a checklist scoring system and report formats.
8. The hiring and training of CQAP reviewers.
9. Development of measurable characteristics of priority inspection items.
10. Development of new policies and procedures for inspection and CQAP.

Notification of Department Personnel That an Evaluation of Inspection and IID was Being Undertaken and Solicitation of Input

As a first step in the process, the Management Services Division developed a procedures manual for the new CQAP project (*3*). Copies were circulated among all levels of management and among representatives from all field positions dealing with inspection and quality assurance. Comments were received from reviewers, and the procedures manual was amended accordingly. At the same time, the nine district engineers (the highest level of field managers) and their assistants were briefed by Management Services personnel.

Development of a Comprehensive List of Inspectable Highway Construction Items

It is clear that as the complexity and number of the Department's construction projects have increased, the role of an inspector has also become more complex. Up to this point, there has been no real effort to define comprehensively in operational terms an inspector's job.

The major tool of inspectors, in addition to the Department's Construction Manual (*4*), is the Department's *Road and Bridge Specifications* (*5*). This reference book supplies information on all types of construction and is extremely complex, covering many topics that describe the precise engineering standards used in the construction program. The *Construction Phase Inspection Handbook* (*6*), on the other hand, gives a broad outline of activities that inspectors may follow in doing their jobs, but it covers neither what items are to be inspected during those activities nor what constitutes a satisfactory inspection.

After examination, it appeared that there was a clear need for a document falling between the *Road and Bridge Specifications* and the *Construction Phase Inspection Handbook* in terms of comprehensiveness and specificity. This new document (or checklist) was envisioned as containing those inspectable items that applied to the project at hand, expressed in measurable terms, so that inspectors could clearly determine whether criteria had been met. Items to be included for a particular project could be standardized by project type and then tailored to meet the needs and characteristics of individual projects. Some indication could also be given in this document as to which of the applicable items was more or less critical to construction quality. A listing of these critical items for each project would give inspectors guidance in managing their time and in concentrating on the most crucial items.

With this intention, an attempt was made to create a list of inspectable items. Initially, personnel from Management Services who had extensive experience with both the inspection process and the IID program converted the *Specifications* into detailed inspection items, incorporating the standards to be met in each item as listed in the *Specifications*. Next, since there were obviously too many items with which any individual inspector would be familiar, items were grouped by standard areas and by physical items to be inspected. When this process had been completed, a checklist of approximately 1,800 items covering all aspects of the construction process had been created. These 1,800 items fell into approximately 75 categories (a portion of a checklist covering one category, Excavation and Embankments, appears in Table 1). Although 1,800 items is a large number of items, it was felt that in a checklist this number was manageable since not all projects would involve all of the items and not all inspectable activities would occur simultaneously.

Prioritization and Weighting of Inspectable Items

Inspectable items do not have equal potential to reduce or improve construction quality. Clearly, inspectors make determinations concerning more important and less important inspectable items on a daily basis as part of their job. It was felt that a method of making these decisions based on a consensus of inspectors and managers was needed. The Council frequently uses a modified Delphi technique when diverse groups need to achieve a consensus, especially with regard to prioritization. It includes aspects of the Delphi procedure and the nominal group technique (NGT). It is felt that the combination of these two methods results in an efficient, objective, and fair technique for prioritization. A hallmark of the Delphi technique is that it is applied exclusively by mail by the administration of several rounds of questionnaires. In each new round, the results of the prior round are provided along with an indication of the overall group assessment. Respondents remain anonymous and are free to revise their previous judgments. The purpose of the feedback is to induce a consensus. The method succeeds if there is convergence on one or several recommendations.

A major defect of the Delphi procedure, however, is that because the participants never meet face to face, there is no opportunity for the direct interchange of ideas or clarification of the issues by fellow participants. The interaction and explanatory materials are controlled by the group coordinators, and this magnifies their influence on the group outcome. Most face-to-face meetings use the usual committee approach, which has been shown to be a largely ineffective method of achieving a consensus. This approach often inhibits discussion because dominant individuals may exert a disproportionate influence on the deliberations, and members may contribute according to their self-perceived status. Additionally, members often make covert judgments but are reluctant to express them as overt criticisms because of the social pressure to conform. Finally, maintaining the group relationship requires a good deal of time and effort, which reduces the group's ability to deal with substantive problems and consider alternatives thoroughly.

NGT provides for structured interaction among participants without allowing the adversarial interchanges often seen in

TABLE 1 TYPICAL CHECKLIST OF INSPECTION ITEMS: SECTION 303–EXCAVATION AND EMBANKMENTS

_____ 1. Are embankments being placed uniformly and with the specified lift thickness? (PS = 1)*

_____ 2. Is the embankment being compacted uniformly and rolled to the outside of the fill? (PS = 1)

_____ 3. Is a density test conducted on embankment material with a fill length less than 500 feet at least every fourth 6-inch lift? (PS = 2)

_____ 4. On fills from 500 to 2,000 feet in length, is a density test conducted on at least every 10,000 cubic yards of embankment material and on every 6-inch lift within 5 feet of subgrade? (PS = 2)

_____ 5. Do the grades appear to be within the tolerances specified in Section 303.16 of the Specifications? (PS = 3)

_____ 6. Has the contractor obtained prior approval from the engineer before opening up a waste area or borrow pit? (PS = 3)

_____ 7. Has the topsoil stripping been confined to the area over which excavation is to be actively prosecuted within 15 days following the stripping operation? (PS = 3)

_____ 8. Do the records show the amount of unsuitable material removed and the person that authorized the removal? (PS = 3)

_____ 9. If the contractor used any borrowed material prior to the use of all regular excavation, did he obtain the permission of the engineer? (PS = 5)

_____ 10. Have all loose rocks larger than 4 inches in diameter been removed from the cut slopes? (PS = 5)

_____ 11. Were slopes widened or flattened to improve the safety of the project before any material was wasted? (PS = 5)

_____ 12. Was authorization given in writing to allow the contractor to waste or sell any surplus material? (PS = 5)

*PS = priority score. Highest priority items receive a 1 and the lowest receive a 5.

group discussion. Since anonymous balloting is used to rank projects, the false consensus that is often seen with voice voting is avoided, and the perception of fairness among participants is maintained.

The modified Delphi technique used to prioritize inspectable items involved several steps:

1. Final preparation of lists of inspectable items. Since it was felt that each Delphi panel would be able to consider only about 600 items at one time, each list of inspectable items developed by Management Services' personnel was composed of three subjects: (a) concrete, asphalt, and soils; (b) environment; and (c) general construction (including contract management). Each panel's checklist included up to 20 sublists covering discrete content areas.

2. Selection of the Delphi panel. Panelists were selected based on their expertise in the particular topic covered by the panel and on their level of interest and their willingness to express opinions concerning prioritization. Each panel included members from several divisions and geographic areas; thus, all aspects of the items could be discussed. Each of the three panels had 8 to 15 members, including persons who had been selected to later become the six CQAP reviewers. Thus, the Delphi process was used to help reviewers become familiar with the inspector checklists.

3. Mail-out round. The first round of rankings, in which all 600 of each panel's items were initially included, was conducted by mail so that participants could consider each item carefully. Panelists were asked to rate each item on a 1-to-5 scale, in which 1 indicated that the item was extremely important in assuring a quality construction product and 5 indicated that the item was extremely unimportant.

The phrase "assuring a quality construction product" was defined as including construction practices that affect the timely continuance of work and those directly affecting the quality of materials and the end products. From this initial round of rankings, about one-fifth of the items were given a 5 and were excluded from further consideration (unless panelists requested that an item be reincluded). In this way, each round of rankings created the lowest priority level, leaving all remaining items open for consideration in the remaining upper priority levels.

It was also decided that items in each sublist would be ranked independently rather than compared with other items in the sublist. There were about 1,800 items under consideration. With ranking within sublists, about 375 would have been ranked as top priority, a number that could be beyond the capability of an inspector to inspect.

4. In-person panel session. After the first mail-out round of rankings, panelists met face to face to consider their lists of inspection items. At the beginning, the purpose of these meetings was described, and panelists were allowed to ask questions. The results of the mail-out round were presented, and the panelists were asked if they wished to salvage items that had previously received the lowest priority rating (and thus would not be considered in future rankings). Panelists were then asked to rerank the remaining items by selecting their top (highest-priority) and bottom (lowest-priority) 40 items. It was hoped that three rounds of ranking could be completed during two 3-hour sessions. However, because of the difficulty in selecting the rankings and scoring the results, the best any group did was two rounds (the other two groups completed one round each). Thus, the remaining rounds were completed by mail.

5. Additional mail-out rounds. Based on the discussion at the in-person panel sessions, several groups requested the opportunity to combine and re-edit items. Thus, the first mailing after the panel session solicited this input. In subsequent mailings, members were asked to rerank remaining items (for instance, in the third round of rankings, panelists were asked to prioritize their top and bottom 30 items).

After all ranking had been completed, the lists were reassembled and reordered based on their priority ratings. Panelists were then asked whether they agreed or disagreed on the priority of each item once they had been able to examine the checklist in its entirety. If someone disagreed with an item's ranking, all members were repolled, and if a majority concurred, the priority was changed. This occurred in very few cases.

Validation of the Prioritized List of Inspectable Items

After prioritization by field employees and midlevel management, validation of the rankings by upper-level management was needed. It was possible that field management had stressed activities that top management would not. For instance, in initial rankings, some field personnel gave a low score with regard to ensuring quality construction to items dealing with mandatory contracting with disadvantaged business enterprises. If this trend had continued throughout the rankings, or if all panelists had given these items a low score, these priority rankings may not have had management's approval.

For this reason, the list of prioritized items was sent to the top management in the construction and engineering areas. After lengthy consideration, these top managers accepted the priorities without change.

Development of a Statistical Sampling Plan for the Program

To correct one of the major flaws in the previous IID program, the new quality assurance plan had to be statistically valid. This meant that a statistically adequate number of samples had to be drawn in a random fashion, representing the relevant characteristics of the construction program.

The first step in developing a sampling plan was to determine what questions needed to be answered through the analysis. This step involved deciding whether the Department was interested in simply estimating the true state of affairs or whether they meant to compare conditions among various groups or years. Management also had to decide on the level of confidence to be used and the degree of accuracy or resolution required. Additionally, if comparisons were to be made, a decision had to be made whether the direction of the difference was important (e.g., whether one group is higher or lower than another rather than just different). There are different (but related) formulas for each hypothesis or analytical situation, and each question had to be associated with a different sample size or sampling technique. In general, the more complicated the question one wishes to answer, the more sophisticated the sampling plan must be (and often, the larger the sample).

A limited set of questions was proposed to be answered for the program's first year, with the intention that the number be increased in subsequent years. Initially, the following were the three questions to be answered:

1. What is the average number of deficiencies per construction project in the Commonwealth of Virginia?
2. What is the average number of deficiencies in each district?
3. Will the number of deficiencies measured next year be significantly different from those this year? (This question, of course, could not actually be answered until the next year, but the current year's sample had to be drawn so that it would be possible to answer the question at that time.)

The factors that go into determining sample size are variability (a factor that is inherent in the data and thus is out of the control of the experimenter), accuracy, and confidence (factors that are chosen by the analyst and thus are within control). These factors are represented in the following formula (*4*):

$$n = \frac{(Z_{1-\alpha} + Z_{1-\beta})^2 (SD^2)}{(M1 - M2)^2}, \text{ corrected } n = \frac{Nn}{(N + n)}$$

where

α, β = the confidence levels chosen;
$Z_{1-\alpha}, Z_{1-\beta}$ = the normal curve values corresponding to the chosen levels of confidence;
SD^2 = the variability estimate used;
$(SD^2_1 + SD^2_2)$ = two samples to be compared;
$M1$ = the true mean;
$M2$ = the estimated mean from the sample;
$M1 - M2$ = the minimum meaningful difference that one can tolerate between the sample mean and the true mean, or the minimum difference one can detect between two samples, for example, 1987 and 1986 (this factor represents accuracy or resolution); and
$\frac{Nn}{N + n}$ = a finite population size correction factor, where n is the uncorrected sample size and N is the population size.

For each question the program was designed to answer, a sample size was calculated and distributed proportionally across the nine construction districts and nine project types. Then, to ensure that the study was statistically capable of answering all the questions, the largest sample size for each district and project type was selected (see Table 2).

As these sample sizes were applied to the program, a number of problems were noted. First, sample sizes were proportionally very large. This was due to the very high variance estimates taken from the 1985 and 1986 IID results. (Actual variances within the new program have been noted to be much lower, thus reducing future sample size requirements.) Second, since the inspectors were not actually taking data until the fourth quarter of 1987, the sample based on the second, third, and fourth quarters (i.e., the 1987 construction season) was not wholly applicable. In addition, by the fourth quarter, many of the projects listed as ongoing were actually completed, reducing the population size. Also, since projects were awarded based on type, several project numbers could be assigned to each job. For instance, one job might include widening, resurfacing, and repairing a bridge. Selecting project numbers necessitated that an inspector visit a job site more than once with no assurance that the project activity he or she wished to inspect was actually ongoing.

The following changes were recommended for the 1988 CQAP sample:

- Jobs rather than project codes should be selected as the unit to be sampled. Thus, the inspector could visit a project and inspect all ongoing activity at once.
- The samples should be pulled on a quarterly basis, rather than annually, to ensure that all projects selected for a particular quarter were ongoing.
- Sample sizes could be reduced based on more accurate variance estimates.

Steps 6–8 of the Study

Steps 6–8 of the study (development of computer software to implement checklists; development of the checklist scoring system and report formats; and hiring and training CQAP reviewers) were initiated as part of the actual implementation of the program. Software was developed to allow all CQAP reviewers to use lap-top computers to evaluate projects and print results after reviews of local construction managers and personnel. Data collected on reviews could then be sent to Richmond in computer-readable form for easy entry into the statewide analysis system.

The scoring system originally selected for use with the checklists required each reviewer to score each item on a scale

TABLE 2 PROJECTS TO BE SAMPLED IN 1987 BY DISTRICT AND CONSTRUCTION TYPE

	DISTRICT								
TYPE	BRIS	SALEM	LYNCH	RICH	SUFF	FRED	CULP	STAUN	NVa
Previously Ongoing	31	18	9	19	30	8	13	7	18
New Construction	2	4	3	10	7	1	1	3	4
Recon-struction	58	38	39	27	10	30	22	40	17
Widening/ Resurfacing	1	4	1	2	3	1	1	0	3
Resurfacing Only	2	0	7	2	1	0	0	0	0
Bridge-New/Major	10	10	7	7	8	3	3	7	4
Bridge-Rehab/Repair	3	1	5	2	1	1	2	1	1
Safety	7	8	5	13	12	9	6	9	11
Misc.	4	1	0	2	2	0	1	4	2
TOTAL	118	84	76	84	74	53	49	71	60

TOTAL = 669

from 1 to 5 where 5 was excellent, 4 was above average, 3 was average, 2 was below average, and 1 was unacceptable. There was, however, some misinterpretation of the highest scores, which led to consistent disagreement between reviewers and low interrater reliability. (Essentially, reviewers had difficulty relating the excellence of the work to meeting contract requirements. Under previous definitions, meeting specifications constituted excellent performance.) After two quarters of data collection under this system, the scoring system was modified and compressed to a four-point scale where 4 meant "exceeds contract requirements," 3 meant "meets contract requirements," 2 meant "below contract requirements," and 1 meant "unacceptable." Interestingly, exceeding contract requirements is not always advisable. For instance, placing reflectorized barrels on 12-ft centers in a work zone exceeds contract requirements; however, it also results in higher costs for the Department.

Once procedures were in place, CQAP reviewers were selected from among the Department's most experienced inspectors. In addition to their training period, these reviewers also participated in the modified Delphi process, allowing them to become more familiar with the checklists and to influence priorities based on their extensive experience.

Steps 9 and 10 of the Study

The implementation of steps 9 and 10 (development of measurable characteristics of priority inspection items and development of new policies and procedures for inspection and CQAP) is just beginning. Management Services has convened an advisory group of experts in construction practices and management to provide guidance for the program. This group will also assist in the refinement of checklist items and development of operational standards for each inspectable item. This guidance should also help CQAP reviewers achieve consistent and reliable ratings from all reviewers and all projects.

DISCUSSION OF RESULTS

Although this system is still in its infancy, its uses are already becoming clearer. As an objective and statistically based program, its results can be used to pinpoint areas in which the Department's construction process is "out of control" and assist in evaluating solutions designed to improve monitoring and contractor cooperation. The checklist can also be used to train new inspectors since it outlines not only the process of CQAP review but also the inspection process itself. In the long run, once the program has been proven to be statistically reliable and valid, it may be possible to include some factor representing a contractor's quality rating into the prequalification formula for future construction processes. In this way, contractors can be made accountable not only for completing a contract but also for providing a quality product and cooperating fully with Department personnel.

All of these uses, however, depend on proving the statistical validity of the review process and results. Whenever researchers or managers create a test or a measure of performance of any kind, several factors must be considered in evaluating how appropriate and useful the test is. These characteristics fall into two categories: those that are not statistical by nature, such as content or face validity, and those that are statistically based, such as reliability and statistical validity.

1. Face validity. Although validity is often a purely statistical term, there is a component of validity that is wholly intuitive and can be judged at the outset. Face validity is how valid or appropriate the test seems to the users and the developers. It includes the following:
   - Does the test or method seem to measure what one intends to measure (in this case, the control of the construction process and the contractor's implementation of the process)?
   - Does each item or section reflect the stated program objectives and does each comprehensively cover the activities one wishes to measure?
   - Will the rating of each item discriminate between projects that are "in control" versus those that are "out of control" (or will all projects score about the same)?
   - Is the vocabulary used in each item accurate and appropriate for the persons taking the test and are all items phrased in a readable and grammatically correct way?
   - Are all items of an appropriate difficulty? Is it too hard or too easy to meet the criteria?

For the most part, through development of the checklists, having them edited, going through the Delphi process, piloting the lists in the field, and gaining the backing of management, each of these questions has been answered.

2. Statistical validity. Statistical validity involves proving objectively that the test or measurement being used actually tests the behaviors or activities it is intended to measure. The key word is objectively. Measures of performance must be related to another objective indicator of quality: one that is well defined and has already been proven to measure construction quality. With regard to the CQAP checklists, there may be several outside measures one can use to determine validity. The question that may be asked is "Do projects that score high on the checklists produce a higher-quality product (or use a more efficient process) than those that score low?" To get at this, the question might be posed as "Do projects that score high on the reviews finish on time more often than those scoring low?" Program analysts might also determine if the project finished within budget or if it had fewer delays during construction. In the long run, analysts might examine the service life of the facility as a function of its CQAP score or use some other outside measure of quality (such as materials testing).

3. Reliability of the measure. Whereas validity of the measure is related to what is being measured, reliability relates to how it is being used. Reliability tests determine whether the same thing is being measured every time the scale is used or whether each person using it is actually measuring the same thing in the same way. If there are differences in reviewers' scores, it must be determined whether they reflect differences in the projects or just differences in the reviewers. Thus, this item is crucial.

The problem is that most measures of reliability are based on having different raters use the test to rate the same thing. If their ratings are different, this is because of the biases of the raters rather than differences in what they are rating. Unfortunately, the only way to obtain accurate reliability estimates is to have each reviewer rate the same projects and then compare their results. (There are measures of reliability that compare even-numbered items with odd-numbered items on a checklist to determine internal reliability. These measures can be used to provide additional information, but they will not yield much insight into the interrater reliability problem.) Since reviewers are assigned to different geographic areas and do not rate the same projects, a clear measure of reliability has not been available. One possible solution to this problem would be to create a series of videotaped project reviews. All reviewers could then rate the various videotaped projects, and their results could be compared. Currently, the Council is considering the feasibility of such an effort.

4. Item analysis. In describing face validity (characteristic 1), some of the features discussed can be determined by simply looking at the checklists. However, there are statistical methods for objectively checking whether each of the qualities required under face validity is met. These statistical tests can be applied only when a sufficient body of data has been gathered. Once the data are available, the following indices can be generated:

- Difficulty index. The difficulty index indicates the average difficulty of each sublist and of the checklist as a whole. For all of the indices used in item analysis, the first step is to separate projects scoring high overall (the top third) from those scoring low overall (the bottom third). For each item, the proportion of each of the two most extreme groups answering the item correctly is calculated by dividing the number answering correctly by the total number in the group. The higher the difficulty index, the easier the checklist; the lower the index, the more difficult. Ideally, meeting the specifications listed in the checklist should not be too easy or too hard as measured by this index.
- Discrimination index. The discrimination index determines the extent to which the checklist (or checklist item) discriminates between projects that are in control versus those that are not (i.e., between those projects scoring high overall and those scoring low). This index is calculated by comparing the proportion of projects in the top group scoring high on an individual item with those in the low group also scoring high on that item. Clearly, if all projects, even those that are out of control, score high on an item, then the item does not discriminate between good and bad types of projects.
- Phi coefficient. The phi coefficient is a point biserial correlation between a high score on a checklist item and a high score overall. It is an index that measures internal consistency and indicates how well each checklist item predicts total review performance. Obviously, the more related an item is to overall performance, the more useful it is in discriminating between projects.

The idea behind using these indices in item analysis would be to develop the shortest and most concise checklist possible that would adequately discriminate between projects that are in control and those that are not. When one finds individual checklist items that have low discrimination indices, they would either be removed from the checklist or rephrased, possibly to be more specific concerning the activity the Department wishes to promote. When low phi coefficients are noted, it may be desirable to remove the item, move it to a checklist that is more closely related to it in content, or revise it to relate more clearly to the overall objectives of the particular checklist. The handling of items with overall high or low difficulty indices is trickier. If an item is high in difficulty but reflects the specification accurately, it would be illogical to remove or modify it (unless the wording was resulting in low scores). Perhaps contractor education on that particular item is needed. For low-difficulty items that reflect specifications that in other situations would be removed, no adjustment is needed since the purpose of the program is to encourage meeting the specifications.

Once these analyses are complete and the statistical validity of the CQAP has been documented, its results can be used to monitor the state of the construction process statewide and provide an additional tool to the Department to assist in creating better, more efficient, and longer-lasting facilities for the public.

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Concrete Tool for Laying Concrete Pavements: Zero Clearance

Y. CHARONNAT, M. L. GALLENNE, AND P. DELIGNE

Traditional equipment for construction of concrete pavement requires two additional lateral runs, which is caused by the position of molding between the tracks of a tractor. It is not necessary to place the molding at the rear of the tractor. The use of a floating table is the solution to this problem. This concrete tool is equipped with a rack of hydraulic vibrators, a rigid floating table, and two lateral formworks. It is pulled by two arms. Various experiments were performed at the Laboratoire Central des Ponts et Chaussées (Public Works Research Laboratory). The results showed that, (a) concrete behaves in the same manner as other compaction materials; (b) self-leveling and self-stabilization of concrete were possible with a floating table; and (c) vibration and delivery of concrete are two parameters that require regularity. These results also show that this concrete tool is useful for laying concrete pavement. The concrete tool does not require the width that is necessary when traditional equipment is used; thus it operates successfully at zero clearance.

The construction of a cement concrete roadway with traditional equipment requires two additional high-quality runs (evenness and bearing capacity) across the road to be constructed. This type of construction is expensive and can lead to rejection of the idea of laying a concrete roadway. Techniques used for spreading and stabilizing materials do not have this inconvenience. Laying is carried out by using a "floating table" drawn behind the tractor. Having analyzed this situation, the Laboratoire Central des Ponts et Chaussées (Public Works Research Laboratory) and the Viafrance Company designed a special piece of equipment for cement concrete, known as the "concrete tool."

ANALYSIS OF EQUIPMENT

To analyze equipment, it is necessary to state properties of the mix. Two types of equipment can then be compared: the traditional, (i.e., bearing table), and the innovative, (i.e., floating table).

Properties of Cement Concrete Material

Cement concrete is a viscoelastic material with Bingham characteristics (Figure 1) (*1*), which means that the material develops a flow effect when placed under stress. If this stress is removed, the material returns to its initial state as a solid material. When applied to laying a cement concrete roadway, this property is explained in the following manner (Figure 2) (Y. Charonnat, unpublished data, 1987):

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1. Before entering the vibration zone (non-stressed) concrete is in a solid but bulked condition. In this state, it has a very low bearing capacity (self-weight).
2. In the vibration zone (the basis of the stress), concrete becomes liquid. Under the Archimedes effect, air has a tendency to rise to the free surface and the density of concrete increases. Because of its liquid state, concrete takes on the shape of the receptacle in which it is found; thus concrete is moldable.
3. As soon as it is removed from the vibration zone, concrete returns to its solid state, but with two very important modifications: it has taken on the shape of the mold in which it was contained and it has acquired a high degree of compactness (approximately 2.4), which gives it a bearing capacity of several hundredths of megapascal (1.5 psi) (Figure 3) (*2*).

Equipment Required for Laying Concrete

Among traditional types of equipment, we shall examine only those that use all concrete propeties, i.e., those for which the mold gives the final shape to the works.

When concrete is delivered, spread, and leveled over the width of roadway to be constructed, several successive operations take place. First concrete is vibrated. This step is generally carried out by using vibrators that generate the stress that causes liquefaction. The material is then "conditioned" in a mold made up from a table and side form. Once removed from the mold, the laid concrete should have acquired its shape and service compactness.

The following requirements impose certain conditions on the equipment:

- Vibrators should be sufficiently powerful to liquefy concrete across the entire width and throughout the thickness of the roadway to be constructed (*3*). The vibration time should be sufficient to ensure that as much air as possible is evacuated from the concrete while it is still in a liquid form under the mold gate.
- The table should be the same shape as that required for the structure, and its position should be defined in terms of the elevation of the structure to be laid (*4*). The position and movements of the table will be largely responsible for the evenness of the roadway.

Bearing Table

General Description

The bearing table is fixed to the frame of the machine that takes up the pressure load of material. Normally four jacks

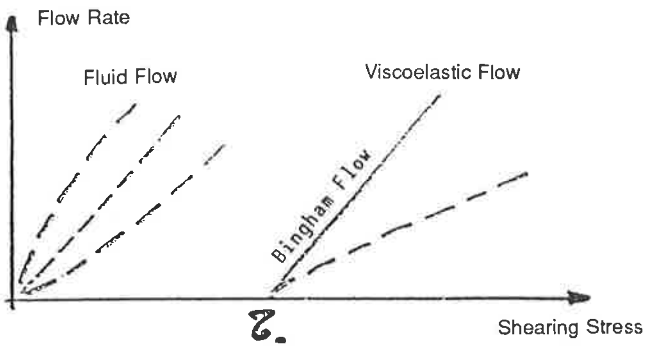

FIGURE 1 Concrete is a viscoelastic material with Bingham characteristics.

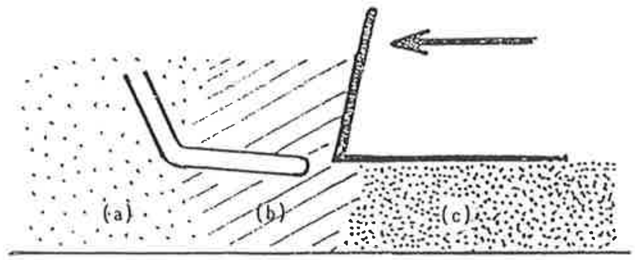

FIGURE 2 Various concrete phases: (a) solid, bulked state, (b) liquid phase, (c) return to solid state after removal from vibration zone.

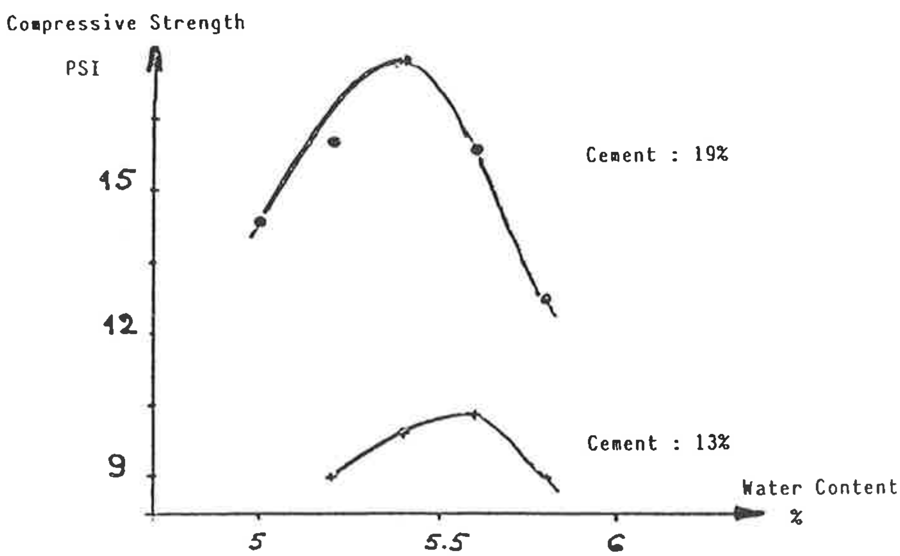

FIGURE 3 Bearing capacity of fresh concrete depends on water and fine element content.

connect the frame to the translation tracks placed laterally to the roadway under construction. Jack positions respond directly to the guide mechanism. The spacial position of the table is thus totally defined (Figure 4).

Positioning and Guiding

The machine driver carries out two successive operations for adjustment. The first involves placing the mold in a parallel position to the roadway to be laid. The second operation involves an opening to the front of approximately 0.1 degrees to improve the surface condition. This angle is adjusted when inspections are made on the laid slab.

The guide reference generally consists of two contact wires. For reduced wires (less than 6 m or 6 yd) a single wire and cross-fall corrector may be used.

Bearing Table Movements

Soil over which the tracks run requires, but often does not have, the necessary evenness required for a roadway. The table, however, must maintain a constant position in space.

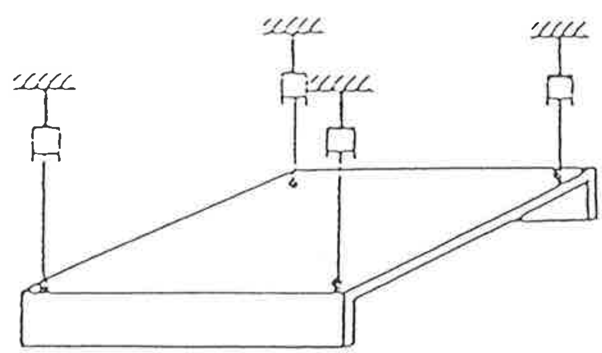

FIGURE 4 Diagram of bearing table.

For these two conditions to be compatible, it is necessary that a jack absorb all the vertical track movements. It is therefore imperative that these movements be detected as early as possible so that they may be absorbed by jacks, which are moved in one direction or another. Since sensors are all-or-nothing detectors, movement commands are translated into short orders, which are the probable cause of reduced length faults that often occur on concrete pavements.

Bearing Table Placed to the Rear of Tractor

It is not necessary to position the table between the tracks; instead, it is possible to place it to the rear of the tractor. The result of this is the ability to lay a roadway wider than the tractor tracks. To pick up vertical pressure loads, the table tractor connection should be rigid. This layout amplifies all tractor movement jacks with a great travel length, which limits the possibility of curves.

Floating Table

General Description

The floating table is integrally assembled on two arms (Figure 5). These arms are connected to the tractor frame by a device that permits vertical rotation of each arm on a vertical axis, slight rotation on a horizontal axis, and longitudinal movement (Figure 6).

The vertical position of the connection points to the tractor are made mobile through the use of a jack whose movement can be blocked or controlled by a guide system. In its working position, the floating table, located at the rear of the tractor, sits entirely on a layer of material that it is spreaoing. Its self-weight picks up the vertical thrust (see section on *Influence of Thickness of Material in Front of Mold*).

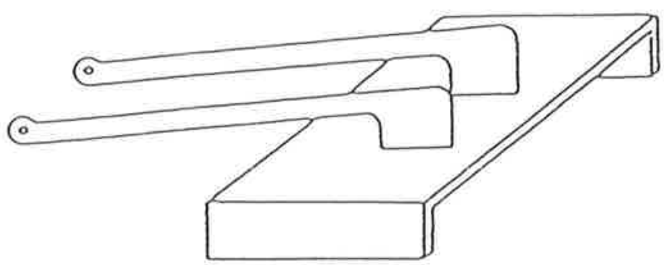

FIGURE 5 Diagram of floating table.

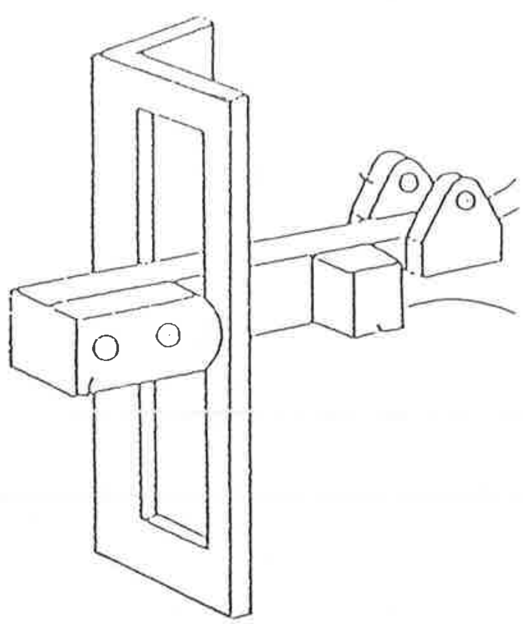

FIGURE 6 Connecting points on tractor.

Positioning and Guiding

The table position adjustment for a given thickness to be laid is carried out by adjusting connection points to the tractor. This operation allows the cutting level of material in its liquid state to be determined and to calibrate the quantity of material entering under the table.

Spreading with the floating table is often carried out without a guide. When a guide system is used, the guide reference is either two contact wires or one wire and a cross-fall corrector. The guide action occurs at the connecting point of the arms.

There are several theoretical sensor assembly possibilities. Two particular points require checking: the connecting point that is linked to tractor movement, and the table spreading edge that determines the thickness of the layer. The intermediate position is generally chosen between these two points, offering a compromise that is currently judged as acceptable.

Floating Table Movements

The reaction of the table to stress at a connecting point (Figure 7) causes the following:

- The vertical movement of the connecting point modifies only the opening angle of the table; the rear load point (table spreading line) remains unchanged.
- Because the length of the arms is much greater than the length of the table (four to six times longer), the instant vertical movement of the spreading edge of the table will be four to six times lower.

This transfer function translates into a slowness in reaction of the table (Figure 8), resulting in a reduced number of faults (amplitude or slope: see section on *Passage of Obstacles*).

THE CONCRETE TOOL

The concrete tool has been designed as an adaptation to the tractor. It is made up only from elements that are required for carrying out this type of operation. The tool has been patented in the United States under number A 3602115.

Tractor

The only imperative requirements for traction equipment are as follows:

- A regulator controlling the supply of material;
- A transversal distributor;
- An even and adjustable speed, upwards from 1 m/min (3 yd/min).

These characteristics are common on usual equipment (e.g., finisher, motor grader, and so on).

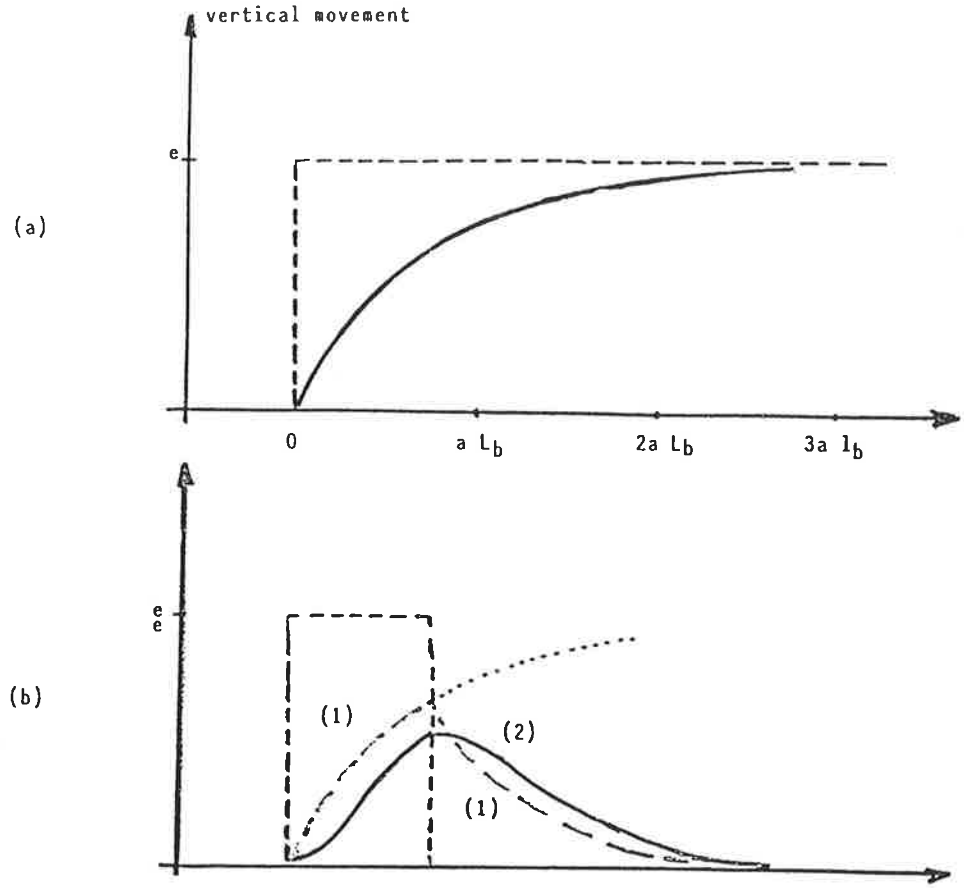

FIGURE 7 Reaction of table to stress at connecting point: (a) incremental (theoretical) response. Stability returns when mold has covered distance proportionate to length of arm, (b) Response to insufficient length fault. (1) Theoretical response, (2) observed response.

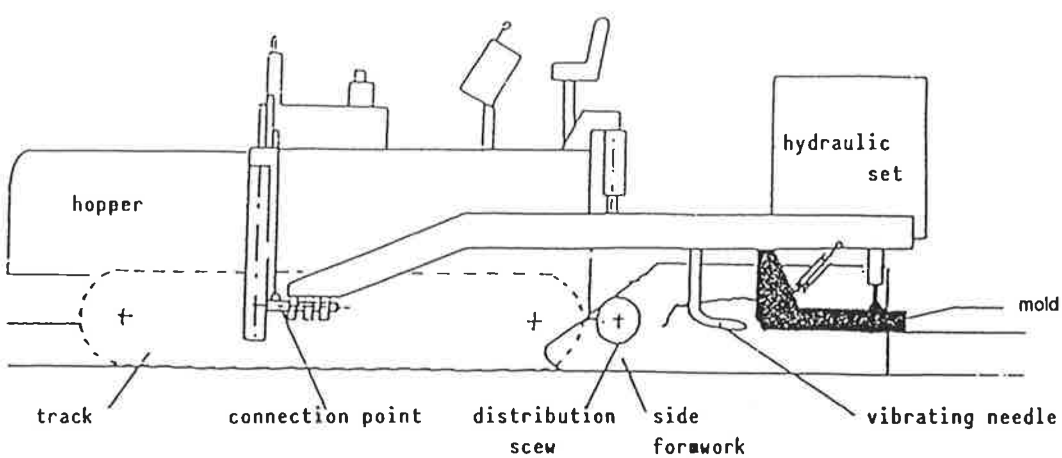

FIGURE 8 Tractor-concrete tool unit.

Concrete Tool Equipment

The concrete tool is equipped with the following:

- A rack of hydraulic vibrators. The number of vibrators depends on the width of the concrete works. Generally, one vibrator per 50 cm is used (1.5 ft).
- A hydraulic set able to supply ten vibrators. This set has a vibration power adjustment device for each of the vibrators.
- A rigid floating table approximately 1 m long (3 ft). It spans 2.5 m (7.5 ft) but can be extended to 4 m by the addition of adapted elements (12 ft).
- Two lateral formworks, capable of vertical movement, whose task is to laterally enclose the concrete.

EXPERIMENTATION

Before it was used on site, the concrete tool was tested on the spreading track of the Laboratoire Central des Ponts et Chaussées (LCPC) (Public Works Research Laboratory), Nantes, France (Figure 9) (*5*).

The slab is 150 m long and 8 m wide. A measuring trolley with a 5-cm pitch permits the level of the spread layer to be measured to a precision within a 0.5 mm.

Influence of Connecting Point Position

To test the influence of the connecting point position, various tests were carried out. The first series of tests consisted of determining the stabilization level of the mold at various connecting point positions. The second series consisted of checking this same stabilization level by placing the mold in various positions. Figure 10 represents all the results obtained during this first set of trials. If E is the thickness of the course and H is the level of the connecting point, then $E = 0.6\,H + 4$, where E and H are expressed in inches. This form of expression is identical to that generally used for materials laid by compaction.

The precision in relation to thickness for a level connecting point is approximately 1 cm (13/32 in.); these fluctuations could be caused by simultaneous variations of other parameters.

Through the second series of trials, we were able to verify that the self-stabilizing property was equally true for cement concrete.

Influence of Consistency of Concrete

The range of consistency of concrete used for the trials (maneuverability from 9 to 25 sec) is much higher than that generally used on site (15 to 25 sec) (Figure 10).

Adjusting the vibration frequency in terms of consistency reduced the recorded approximation. For consistencies varying between 8 and 17 sec, the frequency chosen was 150 Hz. For consistencies between 18 and 25 sec, the frequency chosen was 100 Hz. We note that the approximation has been reduced to ±0.5 cm (7/32 in.) (Figure 10).

Angle of Incidence of the Table

During tests establishing the various thicknesses, the angles taken by the table were measured. The results obtained from five test slabs are grouped together in Figure 11. The angle of incidence taken by the mold is proportionate to the thickness of the spread layer and depends on the quantity of air remaining in the concrete. The section equation deduced from the results is

$$\alpha = 22.5\,E$$

where α is expressed in degrees and E is expressed in feet. The approximation on α is ±0.25 degrees. The angle of the table is practically proportionate to the thickness of the laid material.

This conclusion shows that concrete is compressed under the table by air compression. If it is assumed that the length of the unvibrated concrete under the table is approximately

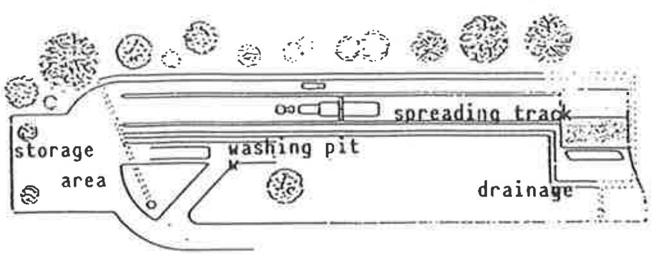

FIGURE 9 Spreading machine test site.

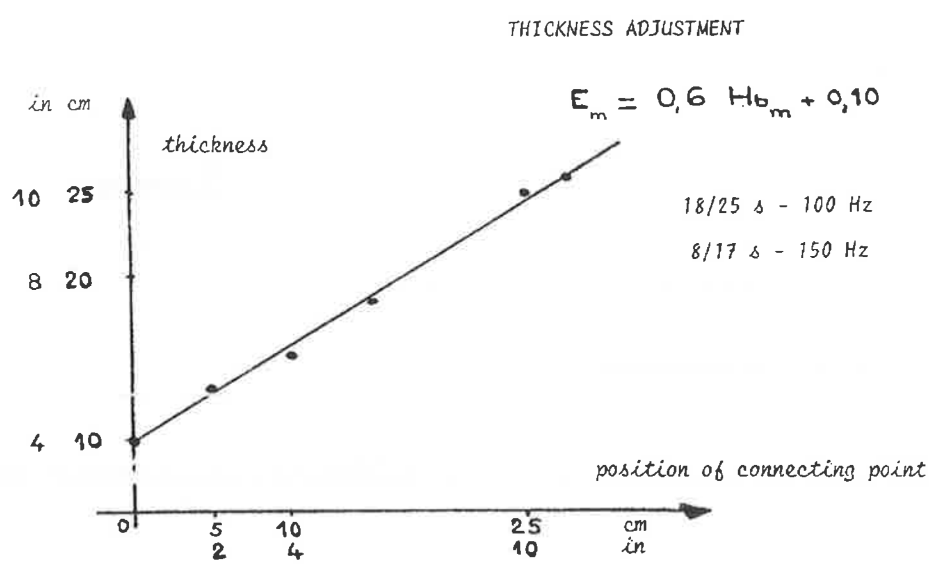

FIGURE 10 Thickness of the layer and connecting point position. (a) Dispersion around section is influenced by variations in water content, (b) adjustment in vibration frequency to consistency of concrete allows a reduction in dispersion.

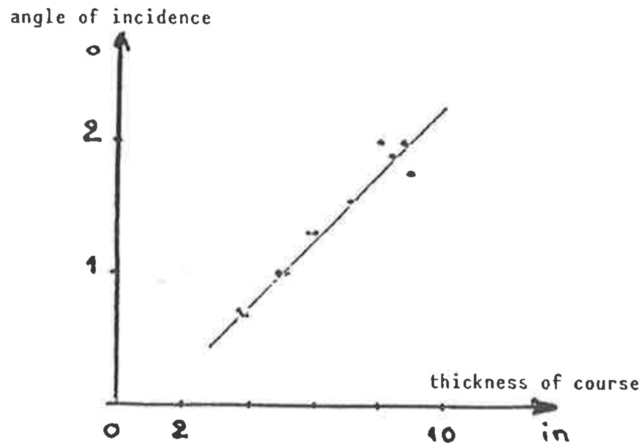

FIGURE 11 Floating mold compacting effect.

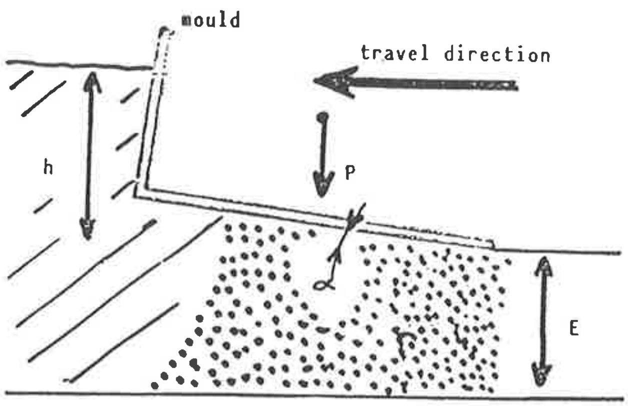

FIGURE 12 Mold balance.

1 m (1 yd), this corresponds to 13 percent of air content at the gate under the mold (8 percent for the bearing table). By reducing the air content at the gate under the mold (vibration efficiency), the variability of α is reduced and the evenness of layer thickness is increased.

Influence of Thickness of Material in Front of the Mold

The influence of thickness of the material was tested during a second trial (*6*). Examination of the various forces leading to the balance of the mold (Figure 12) shows that the angle of incidence taken by the table is a function of the weight of the mold and the height of the material in front of the mold (hydrostatic pressure).

$$\alpha = K(P - kh)$$

where

- α = angle of incidence,
- P = mass of the table,
- h = height of the material from the surface of the constructed slab, and
- K and k = constants for work conditions.

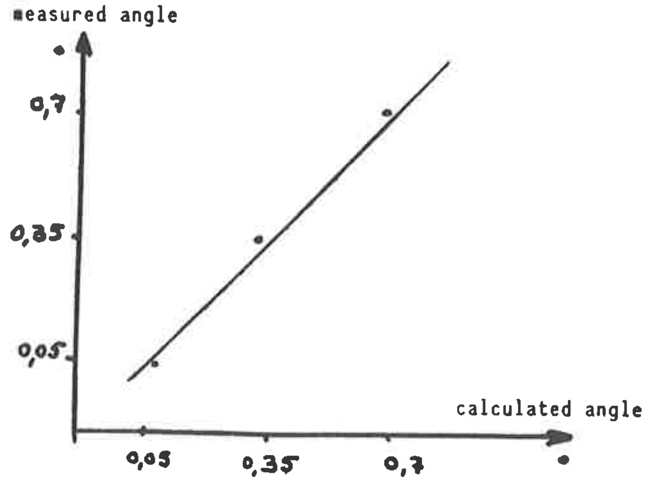

FIGURE 13 Modeling the angle taken by mold.

(Variations in the consistency of concrete noted during the trials did not have any particular influence.)

The mold is subjected to a number of loads: the mass or the bed (P), the hydrostatic thrust (kh), and the compression of the air. For P, varying from 1.4 to 2.9 long tons and h from 0.33 to 0.44 m (1 to 1.1 ft), α varied from 0.15 to 0.7 degrees. The equation thus becomes

$$\alpha = 0.35 (P - h)$$

where

- α is expressed in degrees,
- P is expressed in long tons, and
- h is expressed in feet.

Figure 13 gives the correspondence between calculated α and measured α. The α angle, calculated according to the $\alpha = K (P - \text{kh})$ formula, is very close to the measured angle.

Tests were carried out to attempt to control the level of material in front of the table (*7*). Variations in depth of 28 cm (~1 ft) were produced for a distance varying between 2 and 4 m (2 and 4 yd). The thickness values of the slab had a 3.6-cm (1.5 in.) fluctuation.

A level sensor was installed on the table to check the height of material. This sensor automatically controls the start-up of conveyors and transversal distribution screw. The height of the material within a range of 7 cm (3 in.) with a distance of 20 to 40 cm (8 to 16 in.) could then be controlled. Fluctuations on the installed layer are not perceptible.

Passage of Obstacles

During tests carried out at LCPC, calibrated obstacles were placed below the tractor tread travel path. Runs were then made, both with and without guides. The contours of the faults were then noted.

For a first approximation, to describe the table movement without guides, the following equations can be used:

1. For the amplitude of a movement toward the top:

$$h = aHo\,(1 - e^{-1/aLb})$$

2. For the amplitude of a movement toward the bottom:

$$h = aHo\ e^{-1/aLb}$$

3. For a length (isolated bump-type fault):

$$L = Lo + Lc + 3\ aLb$$

where

Ho = height of the obstacle,
Lb = length of the arm,
Lc = length of the track,
Lo = length of the obstacle,
a = coefficient dependent on the angle of incidence (= 0.7),
1 = distance covered by the table, and
h = vertical movement of the table.

The limits to the resulting fault are hard to establish as the vertical movement is very slow at the beginning and end of the fault.

A 3-m (3-yd) long and 4-cm (1.5-in.) thick obstacle is translated into a 2-cm (13/16-in.) height fault for this system. Under actual working conditions, we have noted a height of 2.2 cm (approximately 14 to 16 in.) and a length of over 10 m (10 yd). We thus find the self-leveling function, well known in the compacting technique of laying materials.

Behavior of Concrete Under the Table

Test results show that concrete behaves in the same manner as other compaction materials and that all laws applicable to these materials apply as well to concrete.

The table has two functions: the first is to give the shape and the second is to give the final density. The second function is very important as it completes the work of the vibrators. Nevertheless, it is necessary to try and limit this regulator function (which permits an increase of a few density points) because vibrators normally should provide maximum densification levels.

WORKS CARRIED OUT

Several projects have been undertaken with this concrete equipment, two of which have given interesting results.

Works on Minor Roads

The width of the works was 2.50m (2.5 yd), and slab thickness was 12 or 15 cm (4.5 or 6 in.), depending on the zones treated. The surface area of 6,000 m^2 was concreted (2.5 mi^2).

The equipment was adjusted based on the test results.

Because the unit had good maneuverability, these works were carried out without any difficulties. The works (Figure 14) had the following characteristics:

- 10 percent longitudinal fall,
- 7 percent support cross-fall,
- Curve radius: 30 m (30 yd), and
- Work carried out in strips or between strips.

The result was completely satisfactory.

STRENGTHENING WORKS

The width of works was 4.00 m (4 yd), and slab thickness was 25 cm (10 in.). Each part of the roadway was made up from two adjacent strips. There was a 5 percent longitudinal fall and a 2.4 percent cross-fall. The maximum laying speed was 1.6 mm/min (1/16 in./min) without any supply-delivery problems. Core samples were taken from the roadway. An average

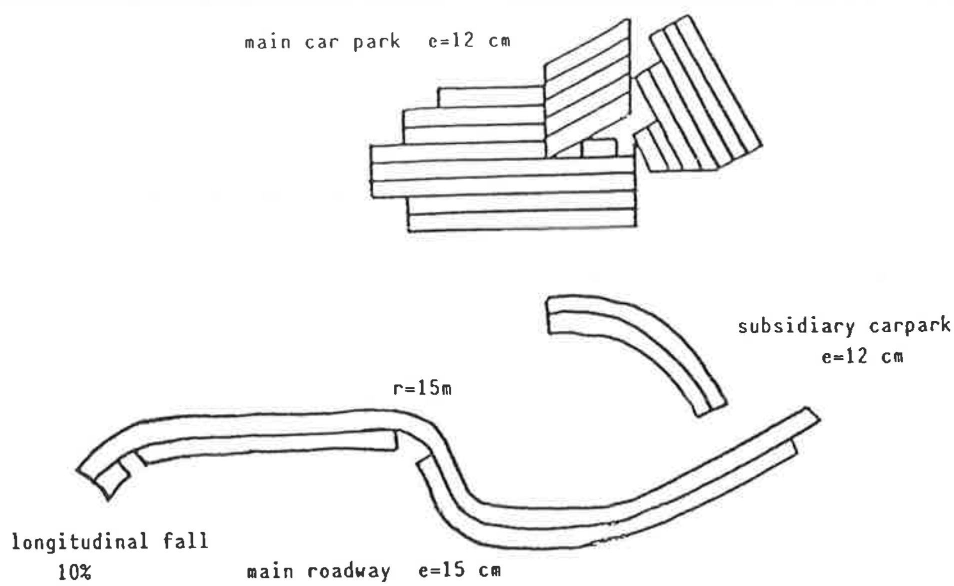

FIGURE 14 Works on small roads: maneuverability of unit enabled work to be carried out without difficulty.

FIGURE 15 Construction of slab. Note sealed appearance of upper surface and straightness of slab edge. Maximum thickness laid was 32 m.

volumetric mass of 2.35 kg/dm³ (147 lb/ft³) was obtained from the 51 core samples taken (2.38 kg/dm³ or 149 lb/ft³ on the test samples) with a standard deviation of 0.03 kg/dm³ (2 lb/ft³).

ADVANTAGES OF THE CONCRETE TOOL

The obvious main advantage of the concrete tool is that it is not necessary to widen either side of the roadway to be constructed. Other advantages include the following:

- The tractor is driven over a section of roadway that has a stable loadbearing capacity. For new roadways the tractor is driven over the sub-base and then is driven over the old roadway to strengthen works.
- The width of the works is not limited to the width of the tractor track. The tool has been designed for 4-m (4-yd)-wide concrete works.

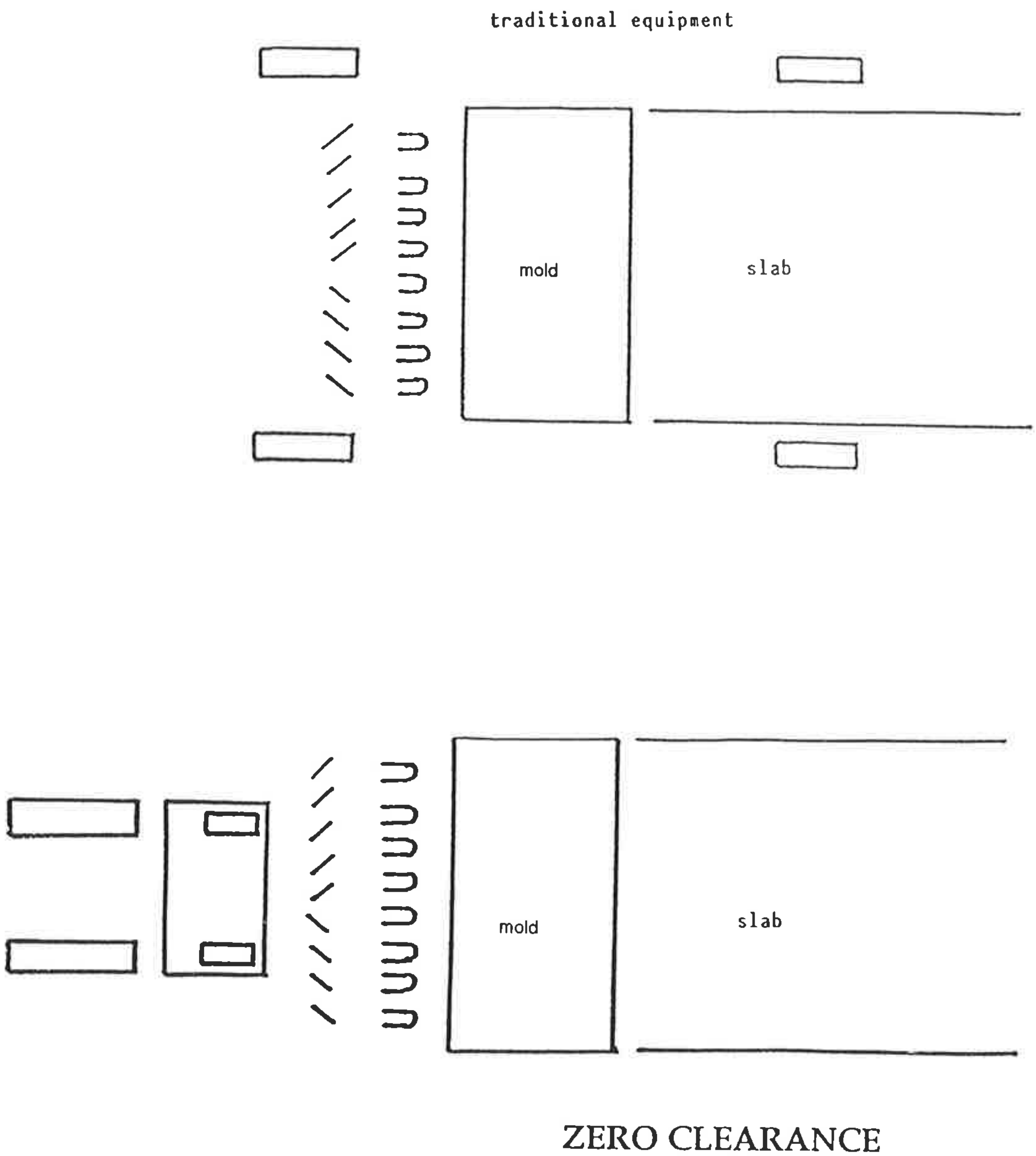

FIGURE 16 Minimal space is required for concrete tool use.

- Investment costs are reduced because the tool is limited to use with materials required for laying concrete.

CONCLUSIONS

Tests carried out with the concrete tool demonstrate that it is entirely feasible to lay a cement concrete roadway with a floating table (Figure 15). Already completed works have shown that the concrete tool-finisher machine is maneuverable, both in tight curves and on steep falls.

Various trials have allowed measuring material and equipment parameters against the quality of the finished layer. Two points demand particular attention: the vibration of the concrete and the ability of the machine to deliver the concrete to the front of the mold. As for other aspects, the concrete tool has the same qualities as those of traditional finishers.

The main advantage of this tool is that it is possible to lay a concrete slab without lateral widening (Figure 16). Other equally important advantages can be ascribed to this equipment, such as extended work width possibilities and reduced costs. This last feature is a result of the adaptability of the concrete tool to all types of tractor.

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Time and Weather Provisions in Construction Contracts of State Highway Agencies

JIMMIE HINZE AND JAMES COUEY

A thorough examination was conducted of the provisions of the construction contract documents of all state highway agencies (SHAs). The study focused on the varying practices of these agencies concerning the inclusion of time and weather provisions in their construction contracts. Results show a wide variability between the various SHAs on most topic areas examined. Although regional differences can explain some of this variability, other provisions appear to be different for no apparent or obvious purpose. A greater degree of consistency between the various state construction contracts of these agencies is advised. Contractors must also be aware of these differences so that they are fully informed about the implications of entering into agreements with SHAs.

Whenever a construction contract is awarded, the owner has specific objectives that the contractor is expected to meet. These objectives can be roughly categorized as containing elements of cost, quality, and time. By awarding the contract to the lowest bidder, the owner has at least some assurance that the cost objective—of having a project delivered at an acceptable price—will be realized. Of course, this cost can be considerably altered by change orders resulting from design changes and also if site conditions differ from what was expected. These additional costs can be best minimized by insuring that designs are complete before the bidding phase and by conducting careful site investigations before completing the bid documents. The quality objective, of having a project delivered with the desired inherent quality, can also be best obtained by carefully preparing complete construction contract documents and by maintaining an effective quality assurance program. The time objective is generally defined as having a project delivered within stated time constraints. The time constraints stipulated in a construction contract may be unequivocal; however, this will not guarantee that the project will be delivered on time. Judicious planning of the construction activities on the part of the construction contractor is essential to the timely delivery of a project. However, even the best-made plans can be stifled when a highway or bridge project is subjected to severe or unanticipated weather conditions.

The issue of how time is dealt with in a construction contract is important, particularly in contracts in which weather can have such an extreme influence on a project that it actually halts or delays construction activities. Of course, an owner can contractually require the construction contractor to deliver the finished project on a firmly established date in spite of any delays caused by weather or other phenomena. A prudent owner, however, does not make these requirements since this type of provision would result in the inclusion of extremely high contingencies in the bids. Thus, the objective of having a project delivered at the lowest reasonable cost would be jeopardized. Therefore, it is important that an owner stipulate unequivocally the terms of the time constraints for the construction of a project but at the same time recognize that a reasonable allowance must be made to alter these terms when unanticipated conditions such as severe weather necessitates it.

Time is a crucial issue in many construction contracts. In fact, it is so important that it is a common contracting practice to include a liquidated damages provision if the contractor does not complete the contract within the time constraints stipulated. These liquidated damages are generally stated as being assessed on the basis of a specific sum of money for each day that the project delivery is delayed. Since the amount of the liquidated damages is often high, particularly on large projects, it is imperative that the contract language be clear where time provisions are concerned.

Although time is obviously an important aspect of virtually any construction contract, little is published about the means by which various owners treat this topic. The textbooks published on the subject of scheduling focus on the algorithms and manual methods of solving problems associated with arrow diagrams, precedence diagrams, and PERT charts. Additional topics usually included in these texts concern cash flow analysis, time-cost trade-offs, resource allocation, resource leveling, and computer applications. Unfortunately, little is included on the subject of how time is actually addressed in construction contracts. Perhaps this is a topic on which little is actually known in terms of general "real world" practice. This lack of information on actual construction contracts prompted the study reported here.

QUESTIONNAIRE SURVEY

To gather more information about the means by which various owners treat time in their construction contracts, a survey questionnaire was developed. This questionnaire was designed to be completed by an owner's representative who was familiar with the methods being used to contractually address such issues as contract time, liquidated damages, weather, lost workdays, seasonal conditions, and progress schedules (survey forms are available from authors). As part of the questionnaire, the respondents were also asked to provide copies of any related standard construction contract provisions that they used.

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The questionnaires were sent to all state highway agencies (SHAs). A total of 50 responses were received, representing a 100 percent response rate. In addition, copies of standard contract provisions were received from 13 of the respondents. With the high rate of response, the findings regarding time and weather provisions as summarized in this paper can be assumed to be a reasonable portrayal of the practices and policies generally implemented by all of the SHAs.

RESULTS: CONTRACT TIME

Generally, the contract time is the time allowed for the construction of a project. More specifically, it may be regarded as the time allowed for construction from the notice to proceed to the point in time when the project is substantially complete. Although the definitions for these occurrences may be regarded as being relatively specific, a considerable portion of these definitions is open to wide interpretation. This is exemplified by some of the practices noted by some of the SHAs. In any case, the timely delivery of the project is important, and it is common to state this in the contract documents with such statements as "Time is an essential element of the contract," and "It is important that the work be completed within the time specified."

The SHAs vary considerably on the definition used for the contract time. The construction contract time is primarily defined in terms of calendar days, working days, completion dates, or some combination of these, as shown in Table 1. The least ambiguous terms for the definition of the contract time is the stipulation of a specific completion date. Fourteen percent of the SHAs use a completion date as the exclusive means of defining contract time, whereas another 26 percent use this when calendar days or working days are not considered appropriate. Specific projects on which completion dates were justified by the respondents were those that were to coincide with other events such as the opening of the World's Fair in the state or region.

Although they may appear to be the least ambiguous, completion dates are clear only on the issue of when the project is to be complete. On the other hand, it is not at all clear how much time is actually allotted for the construction phase. For example, the bid date will be stipulated as a specific date, but the date on which the notice to proceed will be issued is not specified. The starting date may be defined as coinciding with the "date of the engineer's order to commence work . . ." or, as provided in another document, "the latest date specified for the beginning of construction operations . . . or the eighth day after the date of notice of contract approval, whichever is later." Another provision stipulated that the notice to proceed could occur as much as 45 days after the bid opening. Thus, the starting date is not accurately defined. Therefore, although the completion date method may accurately define the completion date for a project, it gives only a general indication of the time allotted for construction.

The definition of contract time that gives the best indication of the amount of time allotted for construction is the use of calendar days. The use of calendar days, as used exclusively by 12 percent of the respondents, leaves the least chance of making an error in interpreting its meaning. This is in contrast to the use of working days. Working days, as used by 34 percent of the respondents, are not as clearly understood. In general, working days are defined as consisting of all days exclusive of Saturdays, Sundays, and legal holidays.

The most common means of defining the contract time is by working days. This information is based on 34 percent of the respondents who use this method exclusively and an additional 30 percent who use this method along with other methods. Although the general definition of working days may be clear, additional factors may confound the definition. For example, over one-third (38 percent) of the SHAs using working days in their contracts will define Saturdays, Sundays, and holidays as working days if work is performed on those days. A typical example of such provisions states "Saturdays, Sundays, and holidays will be counted as working days when the contractor utilizes such days for construction work." Note that contractors working in states with such provisions could not hope to "catch up" on a faltering schedule by simply deciding to work additional hours on weekends.

TABLE 1 DEFINITION OF CONTRACT TIME

Response	Number of responses	
by calendar day only	6	(12%)
by working day only	17	(34%)*
by completion date only	7	(14%)
by calendar day, working day, or completion date	8	(16%)*
by calendar day or working day	7	(14%)*
by calendar day or completion date	5	(10%)
TOTAL	50	

*Of these respondents, 3 counted Saturdays if they were worked and 9 counted Saturdays, Sundays or holidays if they were worked.

WEATHER CONSIDERATIONS

By the very nature of the projects constructed for most SHAs, weather plays a crucial role in the construction time. A question was asked to determine if delays for normally anticipated weather were included in the contract time. A majority of the respondents (30 respondents or 60 percent) stated that normally anticipated weather delays are included in the contract time.

Of the 20 respondents (40 percent) who did not include delays caused by normally anticipated weather in the contract time, 14 (70 percent) stated that extensions were granted because of such delays. One typical provision stated that "No working days will be charged for work performed on subsidiary items when weather or other conditions beyond the contractor's control are such that work cannot proceed on the controlling operations." The six SHAs that do not include normally anticipated weather delays in the contract time and that do not grant extensions for such delays clearly place considerable risk on the contractor. A typical provision from one of these six SHAs provides that "the contractor shall take into consideration normal conditions considered unfavorable for the prosecution of the work and place sufficient men and equipment on the project to complete the work in accordance with the time limit." This added risk borne by the contractor will undoubtedly be reflected in the bids if the contract time is not commensurate with the conditions to be anticipated.

Because weather plays such an important role in highway construction contracts, a question was asked if normally anticipated weather was defined in the contract. It is interesting to note that 36 (72 percent) of the respondents indicated that such definitions do not exist in their contracts, whereas 11 (22 percent) indicated that they do exist. The definition of normally anticipated weather is frequently based on information of past records of weather for the region in question. Such information is available for most regions of the country from the National Oceanic Atmospheric Administration. From this information, the number of anticipated adverse weather days can be determined for each month. Most agencies using this type of data base their judgments of adverse days on temperature and precipitation. Because of cold conditions, particularly in northern states, winter months have the greatest number of adverse weather days.

Although adverse weather conditions are universally accepted as a primary deterrent to making progress on highway or bridge construction projects, other factors can also delay a project. These factors are often included in the construction contracts as constituting just cause for claiming excusable delays. Examples of valid reasons for delay include those resulting from late deliveries or shortages of materials stemming from "some unusual market condition caused by industry-wide strike, national disaster, area-wide shortage, or other reasons beyond the control of the contractor." Other causes for excusable delays include owner-caused delays or work suspensions, earthquakes, floods, cyclones, tornados, embargoes, government acts, and lockouts. Contracts will frequently include factors that do not constitute valid reasons for delay, such as "slow delivery of materials or of fabrication scheduling for reasons of late ordering, financial considerations or other causes which could have been foreseen or prevented."

Of course, to contractors on projects measuring time in terms of working days or calendar days the more important issue is the definition of lost workdays. From the definitions provided, it is noted that workdays are considered lost when either some percentage of the work force cannot be put to work or only some percentage of the workday can be worked (see Table 2). One provision defined lost workdays as those "on which the contractor is prevented by inclement weather or conditions resulting immediately therefrom adverse to the current controlling operation or operations, as determined by the engineer, from proceeding with at least 75 percent of the normal labor and equipment force engaged on such operation or operations for at least 60 percent of the total daily time being currently spent on the controlling operation or operations . . ." Another provision clearly defined workdays as those "on which the work can be effectively prosecuted during 6 hours or more of the contractor's daily working schedule. One-half day will be assessed for each working day on which the work can be effectively prosecuted for at least 2 hours but not more than 6 hours of the day."

TABLE 2 DEFINITION OF LOST WORKDAYS BECAUSE OF UNUSUALLY SEVERE WEATHER

Response	Number of responses	
by % of work force present/absent	1	(2%)
by % of day worked	16	(32%)
50% or less of day worked (4 responses)		
60% or less of day worked (1 response)		
70% or less of day worked (1 response)		
5 hrs or less of day worked (5 responses)		
6 hrs or less of day worked (1 response)		
Not specified (4 responses)		
by combination of % work force present/absent and % of day worked	7	(14%)
not defined	24	(48%)
not applicable	2	(4%)
TOTAL	50	

Whenever a lost workday is assessed, the completion time will be altered. Thus, an agreement must be reached between the contractor and the owner concerning the claim for a lost workday. Most contracts will stipulate the time during which unusually severe weather is to be reported. According to the responses to a question on this topic, the most common reporting interval for lost workdays is at each occurrence. A significant number of SHAs require this reporting to take place within a week of the occurrence (see Table 3). Note that eight SHAs do not require such reporting until the end of the contract. This late reporting may be a disadvantage to a contractor who may rely on the acceptance of certain lost workdays. Such a contractor may incur heavy liquidated damages in the event that the contract time has run out and the owner refuses to accept the contractor's request for an extension as a result of the lost workdays.

Some SHAs appear to make a subjective determination of when lost workdays are to be assessed. Several respondents indicated that they were "flexible" when deciding what constitutes lost workdays. These respondents stated that they would evaluate the impact of occurrences on each specific job. In general, lost workdays are assessed when the work progress is impaired to a significant degree and when the cause is not under the control of the contractor.

When the contract time is established in terms of working days, it is often convenient to have a systematic approach of converting to calendar days. One of the methods of converting from workdays to calendar days is by using a conversion factor, such as 1.40. This conversion constant treats all days equally, regardless of season. A conversion method that reflects seasonal differences is the seasonal weighting of days. When asked about this method, 30 percent (15) of the respondents stated that they used it, whereas 70 percent (35) said they did not. One good example of the seasonal weighting of days is shown in Table 4. From the table, one can quickly assess the equivalence established between working days and calendar days. For example, two working days during February are equivalent to 28 calendar days, whereas two working days in April are equivalent to 5 calendar days. These "weights" obviously reflect the fact that less performance can be expected during the winter months. The information presented in Table 4 was provided by an SHA that does not grant extensions for weather delays, but the SHA uses the weighting of days to extend the performance period for change orders that affect project duration. One SHA contract provision stipulated that extensions "will be given for loss of time due to weather conditions for the number of days lost . . . in excess of 5 calendar days." Thus, this provision includes five "weather days" per month that are considered part of the contract time.

TABLE 3 FREQUENCY WITH WHICH LOST WORKDAYS CAUSED BY UNUSUALLY SEVERE WEATHER SHOULD BE REPORTED

Response	Number of responses	
each occurrence	15	(30%)
weekly intervals	12	(24%)
monthly intervals	8	(16%)
at end of contract	8	(16%)
no response	4	(8%)
not applicable	3	(6%)
TOTAL	50	

Weather in certain months often is so severe that it is not reasonable to expect any appreciable amount of work from the contractor performing outdoor activities. In this type of case, the winter months are often defined as "free days," meaning that no contract time is consumed during these months. The survey asked if a winter exception period was included in the construction contracts. The results showed that 70 percent of the SHAs use the winter exception periods. Information in Table 5 shows the varying winter exception periods used by various states.

Contractors should be aware of the practice noted by six (12 percent) of the SHAs whereby adjustments will be made to the contract time if work is actually performed during the exception period. As seen in Table 5, the SHAs not incorporating a winter exception period in their contracts tend to be those in the southwest region of the country, the Atlantic states, the Gulf Coast states, and those states bordering the Pacific Ocean. Although a few states do not use winter exception periods in their contracts, states such as Arizona include winter exception period provisions on contracts for work in only those portions of the state that are adversely affected during the winter. Still others may use one winter exception period for asphalt paving work and a different winter exception period for other work such as surfacing.

LIQUIDATED DAMAGES

Failure of a contractor to deliver a project within the time constraints stated in the contract might constitute a material

TABLE 4 EXAMPLE OF SEASONAL WEIGHTING OF DAYS

Month	Work Days	Cumulative Work Days	Conversion Factor	Cumulative Calendar Days
Jan	2	2	15.500	31
Feb	2	4	14.000	59
Mar	7	11	4.429	90
Apr	12	23	2.500	120
May	18	41	1.722	151
Jun	18	59	1.667	181
Jul	18	77	1.722	212
Aug	18	95	1.722	243
Sep	18	113	1.667	273
Oct	15	128	2.067	304
Nov	5	133	6.000	334
Dec	2	135	15.500	365

breach of the contract in the absence of other provisions. To avoid lengthy litigation concerning the damages to be paid for such a breach of the contract, it is now a common practice to establish this cost sufficiently early so that the amount is included in the bid documents. The owners can increase the enforceability of the damages clauses with such statements as, "Time being an essential element of the contract, it is hereby agreed that the department will be entitled to damages for failure on the part of the contractor to complete the work within the prescribed time." It is common to also add that the liquidated damages amount is to be construed "not as a penalty but as liquidated damages to compensate for the additional costs incurred." By establishing the liquidated damages provision, all parties to the contract will know the costs to be incurred for delivering the project later than is stipulated by the terms of the contract. The amount of these damages, referred to as liquidated damages, are generally applied to each day that the project completion exceeds the agreed completion date. The rights of the owner are further protected by such provisions, stating that "permitting the contractor to continue and finish the work or any part thereof after elapse of contract time will not operate as a waiver on the part of the division of any of its rights under the contract."

TABLE 5 WINTER EXCEPTION PERIODS BY SHA

Exception Period[a]	State
Nov. 1 to April 30	Alaska
Dec. 1 to March 31	Colo., Conn., Ky., Mich., S. Dak., W. Va., Wyo., Va., N.H.
Dec. 16 to March 15	Del.
Dec. 1 to Feb. 28	Idaho, Nebr., Nev., Oreg., Utah
Nov. 15 to March 31	Iowa, Wisc.
Nov. 15 to May 15	Maine
Nov. 30 to April 1	Tenn.
Nov. 1 to March 31	Mass.
Nov. 15 to April 15	Minn., Mont., N. Dak.
Dec. 15 to March 15	Mo., N.C.
Dec. 1 to April 30	Ohio, Vt.
Dec. 15 to April 15	R.I.

[a]Of the 35 SHAs using winter exception periods, six (12 percent) adjust the contract time if work is performed during this period. Fifteen SHAs do not use winter exception periods.

The liquidated damages provisions are generally defended on the grounds that the public is denied use of the facility, that public safety is jeopardized by the delayed completion, and that the SHA will incur additional administrative costs as a result of late delivery of the project. It is understandable that these costs would be difficult to quantify on a project-by-project basis.

The SHAs were asked about the assessment of liquidated damages. Forty percent (20) of the SHAs assess liquidated damages on the basis of calendar days, 32 percent (16) of the SHAs assess them on the basis of working days, and the remaining 26 percent (13) use either calendar days or working days. When a contract stipulates the completion time in terms of working days and the liquidated damages are also assessed on the basis of working days, the contractor should fully understand the contractual implications. For example, such a contractor should determine whether the added expenditure of working on weekends would be an advantage or if these weekend days would then be counted as working days, thereby further reducing the available days allotted for completion of the project.

Although no question was asked about the amount of liquidated damages to be charged for each day of late project delivery, some of the respondents provided copies of their standard specifications. From an examination of these specifications it is clear that some SHAs establish the amount of liquidated damages on a project-by-project basis. Some of the SHAs, however, have standard schedules by which the amounts of liquidated damages can be easily determined. In all cases, these amounts were dependent on the size of the project (see Table 6). Although the amounts to be charged may be set by

TABLE 6 SCHEDULE OF LIQUIDATED DAMAGES BY SHA

	Daily Charge of Liquidated Damages in Dollars					
Range of Contract Value (1000's of Dollars)	Colorado	Minnesota	N.Dakota	S.Dakota[a]	Virginia	Wyoming[a]
0-25	85	150	50	50	50	63
25-50	140	150	100	100	75	105
50-100	205	250	150	200	100	154
100-500	280	400	225	300	150	210
500-1,000	420	500	300	400	200	315
1,000-2,000	560	600	400	500	300	420
2,000-4,000	840	900	500	600	400	630[b]
4,000-8,000	1120	900	500	600	500[c]	--
8,000-10,000	1400	900	500	600	500	--

[a]The daily charge is per working day, not per calendar day.
[b]A charge of $630 applies to contracts up to $5 million, $840 is assessed per day for contracts from $5 million to $10 million, and $980 is assessed per day on contracts over $10 million.
[c]Liquidated damages in this range and larger may be otherwise specified in the contract.

a schedule, the contract may also state that all parties to the contract agree that the stipulated daily charge is presumed to be reasonable.

PROGRESS SCHEDULES

Although the assessment of liquidated damages occurs after a project should have been completed, the assessment of such damages provides little consolation to the owner who is denied the use of the facility. Many owners would prefer to have an ongoing understanding of the progress being made on a project. Many owners even include provisions in the contract that empower them to terminate the contract if satisfactory progress is not being made on the project. To have an enforceable contract provision of this type, the owner must have an effective means by which to monitor progress.

A variety of monitoring methods are available to owners, including the use of milestone dates, narrative descriptions, Gantt charts, and critical path method (CPM) schedules. These methods vary considerably in their ability to accurately portray construction progress and to accurately predict project completion. The various SHAs were asked which types of progress schedules they required on their construction contracts. The responses indicate that all of the monitoring methods are used (see Table 7). Note that only 8 percent of the SHAs make exclusive use of the more sophisticated CPM schedules, 38 percent of the SHAs use only Gantt charts, and 14 percent have no contractual requirement concerning a progress schedule. Note that the owner's right of termination for failure to make adequate progress is considerably weakened when no effective means of monitoring construction progress is in use.

Several respondents indicated that they generally gave the contractor the choice of using bar charts or CPM schedules. In most cases, the bar charts are chosen by the contractors. Some respondents indicated that, although the bar charts were required on most projects, on the larger and more complex projects CPM schedules will be required and, in some cases, may need to be managed by a consultant.

A total of 43 SHAs were noted as requiring some form of progress schedule from the contractor. These SHAs were asked how weather was reflected in the schedule. Earlier it was noted that 20 SHAs do not include normally anticipated weather in the contract time. Conversely, 30 SHAs do include normally anticipated weather in the contract time. Only nine respondents indicated that weather is reflected by some means in the schedule (see Table 8). Taking into account the information provided earlier and in Tables 7 and 8, one might conclude that an anomaly exists in the information provided. It appears that several SHAs include normally anticipated weather in the contract time and require that the contractor provide some type of progress schedule, but do not have weather reflected in the schedule. However, several respondents indicated that the severe weather need not be reflected in the schedule if a winter exception period exists in the contract.

Obviously, the issue of weather is considered to be important by most SHAs, but it is not considered sufficiently important to address the subject in the progress schedules. The few SHAs that do address weather in the progress schedules use either a separate weather activity or an appropriate weighting of activities in their schedules. These SHAs tend to be those that use CPM schedules for project monitoring.

TABLE 7 TYPE OF PROGRESS SCHEDULE REQUIRED

Response	Number of responses	
Gantt chart only	19	(38%)
critical path method only	4	(8%)
Gantt chart or critical path method	18	(36%)
narrative/milestone	2	(4%)
no schedule required	7	(14%)
TOTAL	50	

TABLE 8 HOW WEATHER IS REFLECTED IN THE SCHEDULE

Response	Number of responses	
not reflected	28	(65%)
by separate weather activity	2	(5%)
by weighting activities	7	(16%)
no response	2	(5%)
not applicable	4	(9%)
TOTAL	43	

CONCLUSIONS

A basic conclusion from the analysis of the responses of the various SHAs is that there is little uniformity among SHAs in their construction contracts in regard to their practices concerning time. Some SHAs are relatively sophisticated in their practices, whereas a few use practices that weaken their contractual position. Obviously, some projects are sufficiently small in scope that rigorous adherence to all policies might not be productive or meaningful. The responses to the questions indicate, however, that in some states informal adherence to policies may be a practice on projects of all sizes.

The findings also provide a clear warning to contractors who plan to work for various SHAs. That warning is that the provisions in the construction contracts on issues such as time are not consistently addressed; often they are different in adjacent states. Topics to scrutinize in the contract documents include the definition of contract time, the impact that normally anticipated weather delays have on the contract time, the definition of lost workdays, the reporting interval for requesting the assessment of lost workdays, the existence of a winter exception period, the amount of liquidated damages to be assessed for late completion, the type of progress schedule required, and any other time-related issues that are addressed.

RECOMMENDATIONS

In light of the findings of this study, construction managers at every SHA are encouraged to conduct a thorough internal evaluation of how time is addressed in their contracts. The standard procedures as outlined in the standard specifications of the SHAs may not be closely followed in actual practice. In such instances the standard specifications should be changed or, if supported by an internal review, the practices should be changed.

In general, SHAs should make a strong attempt to enter into only those construction contracts that are clear and fair. A clear and fair contract will guarantee that a good working relationship will exist between the contracting parties. In addition, the issue of risk should be fairly addressed. It may be desirable for an owner to contractually require the contractor to assume all the risks associated with a project, but this is counterproductive. Contractors who are asked to assume greater risks simply adjust their bids in accordance with the perceived risks, thereby increasing the overall costs of construction. Construction work has significant inherent risks without shifting added risks onto the contractor.

From this research several specific recommendations can be made to SHAs with regard to time and weather provisions in their construction contracts. For each project to be constructed, a careful decision should be made regarding the appropriate means of defining the contract time. The contract time should either include delays for normally anticipated weather or provide a means for granting extensions when such delays occur. If delays for normally anticipated weather are included in the contract time, a clear definition for such weather conditions should be provided. In addition, a clear measure must exist for defining lost workdays. The assessment of lost workdays should be made on a timely basis, preferably within a month of their occurrence. If winter exception periods are to be used, they should be clearly defined. In general, it is advisable that work performed during the winter exception period or on weekends (for contracts defined in terms of working days) not be assessed against the contract time. The liquidated damages provisions should be unambiguous; in most cases, the assessment on the basis of calendar days is preferred. Greater attention should be given to the contractor's obligation to provide a progress schedule. An owner should require the type of schedule that provides the level of monitoring that is warranted by the project. It should also be made clear how the contractor is to reflect the weather factor in the schedule.

Finally, it would appear to be appropriate to establish greater collaboration among the various SHAs so that information can be shared more effectively. In the litigious environment that currently exists, true benefits might be realized by communicating to a greater extent with the construction divisions of other SHAs. Research might also reveal interesting findings as to the relative effectiveness of specific policies and practices. With the high cost of construction and the high cost of some claims, it behooves every owner to prudently examine the value of every provision included in the construction contract documents.

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Potential Applications of Robotics in Transportation Engineering

FAZIL T. NAJAFI AND SUMANTH M. NAIK

Deficiencies in transportation facilities must be addressed urgently. The adoption of new technologies into the transportation sector is necessary to the productive and economic rehabilitation, repair, and maintenance of the infrastructure of existing transportation systems. The continuous need for construction and rehabilitation and repair and maintenance of transportation network systems, specifically under uncontrolled environments, is an issue addressed in conjunction with robotic applications. The nature of robot tasks is discussed in this paper to provide general knowledge about this technology. Also discussed are robot attributes and the manipulation of robots for tasks such as welding and painting of highway bridge components. Current research in other robot applications such as sealing, grinding, sandblasting, inspection of concrete pavements, handling of precast concrete beams, fabrication of steel and reinforcing bars, excavation, tunneling, roadside management, hazardous material handling, railway track maintenance, nuclear plant clean-up, and the use of robots in harsh environments are also described. Although the United States currently has a lead in artificial intelligence, related computer technologies, and some areas of robotics, large-scale field applications of robots is the biggest challenge to this infant technology. There is a great need for a comprehensive, unified robotic research information system for the dissemination of information on robotics in transportation.

In the United States today, there are almost 4 million mi of streets and highways. Some of these were built long ago by using old, conventional techniques and materials. Many of them are already deteriorated and need rehabilitation and replacement. Deficiencies have been developing in this transportation system at a rate faster than funds have become available to keep it serviceable. For example, there are approximately 575,000 bridges in the street and highway system, of which more than 44 percent are considered deficient in one way or another. Similar problems exist for the railroad system, comprising 166,000 mi of mainline roads and more than 100,000 mi of other rail lines, all with track structure and bridges requiring maintenance and rehabilitation (*1*). These observations show that current and near-future research trends in the transportation sector will focus on innovative strategies such as robotics for the improvement of infrastructure facilities.

Transportation projects have always involved construction operations that are highly equipment intensive. Although some of these operations have made use of automation technology in varying degrees, developments in robotic applications, such as in Japan, should prompt more research in the United States in advanced technologies like robotics and artificial intelligence. These technological innovations have definite potential in bringing about dramatic increases in the productivity and quality of operations.

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In the foreseeable future, most of the traditional civil engineering projects can be expected to evolve and adapt to changing market conditions. Even in established disciplines such as transportation, water supply, waste treatment, and geotechnical engineering, pressures from urban growth and resource depletion will (a) force technological advancement toward tackling increasingly constrained conditions in subway and sewer tunnels, (b) minimize disruption in highway and utility structures, and (c) maximize the flexibility and utility of buildings. Advances in materials and design concepts, coupled with the need to reduce the risk of failure associated with natural calamities such as earthquakes, floods, hurricanes, and fires, will continue to mandate advances in structural engineering. This paper focuses on the issues involved in robot applications in transportation engineering; the nature of robot tasks and attributes of robots; the use of robots in construction inspection, tunneling operations, roadside management, the handling of hazardous materials, railway track maintenance, and space applications; and the use of expert systems in robot task planning. The main theme of the paper is to recognize the tremendous future in advanced technologies for civil engineering among which robotics is a challenging yet highly beneficial technology.

The use of robotics in civil engineering currently is still in its infancy. The Japanese have successfully introduced a few robots in the building environment. These robots have the potential to shrink personnel requirements, boost productivity, and relieve human workers of hazardous and repetitive work (*2*). Recently, the 5th International Symposium on Robotics in Construction in Tokyo, Japan, emphasized that research has been forthcoming in the field of construction robotics. Several successful robotic applications have been illustrated to demonstrate the feasibility of this technology in the domain of construction. Similar robot applications can be introduced into transportation construction and infrastructure rehabilitation operations as discussed later in this paper.

NEED FOR APPLICATION OF ROBOTICS IN TRANSPORTATION ENGINEERING

Transportation engineering is now poised to adopt advanced technologies and innovative strategies to improve productivity and performance. A recent workshop sponsored by the National Science Foundation to examine the state of the art and research opportunities in transportation showed that the ultimate objective of research in the areas of transportation facilities is to develop technological innovations that will result in substantial improvements in the quality of transportation facilities, including productivity and performance (*3*). Fundamental research on transportation facilities has also been outlined for repair, rehabilitation, and maintenance with the use of robotics and other automated systems (*4*). The type of work commonly called "3R" (resurfacing, restoration, and rehabilitation) as applied to highways (*5*) has the greatest benefit in robot applications, apart from areas such as assessment of facilities conditions, use of new materials in highway components, and vehicular navigation, control, and location (*6*).

Current technologies and practices to rehabilitate existing facilities have been derived largely from those of new construction. However, the market for these two types of activity and the types of work involved are different in some important respects. Whereas new construction comprises a mix of project sizes ranging from multibillion-dollar megaprojects to small bridges, rehabilitation is much more consistently a small-scale proposition—patching, replacing, strengthening, sealing, painting, lubricating, and so on. Therefore, the natural occurrence of economies of scale, the incentives toward mechanization and use of improved material and techniques, and the corresponding development of larger, efficient organizations have not yet taken place in rehabilitation (*5*).

If possible, rehabilitation efforts must take place without closing down transportation facilities. Furthermore, work space is typically confined and may often be inaccessible or invisible to work crews using standard construction technology (*5*). If work has to take place outside of peak hours, issues such as overtime compensation for workers, additional lighting facilities during dark hours, and scheduling of activities need to be addressed. These factors have not been reflected strongly so far in productivity and performance analyses in the transportation sector because the volume of rehabilitation work has been small on an overall basis. But the need for new construction shows a relative decline compared with the need for rehabilitation work. Highway needs formed nearly 86 percent of estimated infrastructure needs of the transportation sector (*6*). For example, highway needs studies prepared biennially by the Department of Transportation for the period 1975–1981 show that traffic (in vehicle miles traveled) grew at an annual rate of 2 percent during that time, with higher-than-average growth occurring on the interstate system. In that same interval, the interstate system, which carries a disproportionate share of the traffic, exhibited a decline in pavement condition overall, despite the addition of new mileage to the inventory (*5*). This trend will continue for some time, and it will become necessary to introduce advanced technologies such as robotics to quicken the pace of efficient infrastructure improvements and new construction.

Several potential areas of application of automation are recognized in this paper. Varying degrees of automation can be introduced in equipment ranging from automated data acquisition techniques to cognitive robots, as discussed below. However, it is useful to introduce some of the characteristics of modern robots before undertaking a discussion of their applications in transportation engineering.

Nature of Robot Tasks

Robots are classified in general into three classes, based on the amount of human intervention necessary to control them. The first of these, called teleoperated robots, are fully dependent on human operators for planning, perception, and manipulation. The second of these, called programmed robots, perform predetermined, definite tasks by preprogrammed instructions. The third of these, called cognitive robots, are capable of sensing, modeling, planning, and acting, independent of human operators.

The Robotics Institute of America defines a robot as a reprogrammable, multifunctional manipulator designed to move material, parts, tools, or specialized devices through variable programmed motions for the performance of a variety of tasks. Although this definition is valid to a point in the factory, it excludes the high- and low-end capabilities of devices that are relevant in unstructured and uncontrolled environments and underemphasizes the importance of mobility and force that are essential in construction (*7*). Highway construction and rehabilitation operations take place in unstructured environments. Since robots working in such environments require varying degrees of human intervention, the possibility of introducing all three classes of robots and their hybrids has to be expected (*7*).

Attributes of Robots

The attributes of robots that are important from the civil engineering standpoint are their manipulation, effecting, control, sensing, and mobility operations (*8*).

Manipulation

Robots usually have an arm that can reach and grasp an object and move it from one location to another. This action is facilitated by a wrist. Examples of common manipulation systems are shown in Figure 1: (a) three axial translations in rectangular coordinates; (b) rotation and biaxial translation in cylindrical coordinates; (c) two axial rotation and translation in polar coordinates; (d) three axial rotation in revolute coordinates; and (e) anthropomorphic or articulated (*8*).

Effectors

Effectors are devices that enable the robot to do any particular task. In civil engineering applications, effectors can be used for grasping, jointing, welding, painting, and other similar tasks. These effectors are operated by the manipulators. Typical effectors are the welding gun, spray painting nozzle, bolt-tightening grippers, sealing and jointing devices, and grinding disc (*8*). The grippers are used for "pick-and-place operations" and are of different types, such as finger grippers, suction grippers (for flat and smooth objects), magnetic grippers (for metallic objects), and tube grippers (for hollow circular tubes) and are shown in Figure 2 (*8*).

Control

Robots are classified in six groups based on the level of control or intelligence (*9*) as

- M1: Manual control (teleoperation);
- M2A: Fixed sequence built into the robot mechanical system;
- M2B: Variable control, according to preprogrammed instructions;

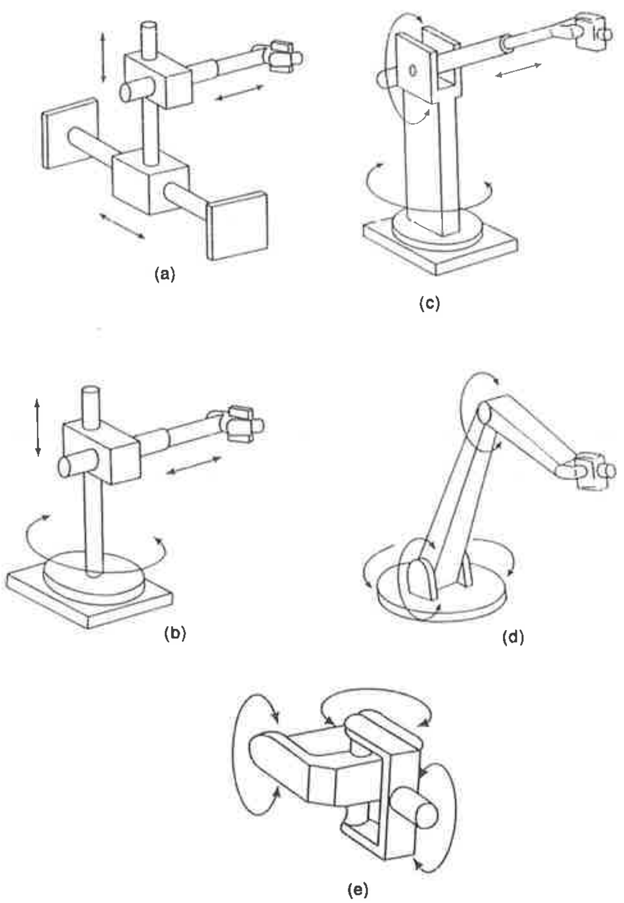

FIGURE 1 Typical configurations of industrial robots (a) rectangular, (b) cylindrical, (c) spherical, (d) jointed, (e) wrist (*8*).

- M3A: Playback, in which the control unit "learns" the desired sequence of arm movements when the arm end is guided by the operator on the path of its intended activity;
- M3B: Numerical control, using a computer language that the control understands; and
- M4: Artificial intelligence capabilities built into the robot, involving a process of learning from vision, contact, and hearing (7).

Sensors

Sensors are most useful in M4 type robots when the control unit can modify a manipulator's activity based on the information received from the environment during its performance. Various physical parameters such as sound, touch, and sight are converted into electronic signals that can be recognized and acted upon by the control unit of the robot (*7*).

Mobility

Robots being used in the manufacturing industry are mostly stationary. But robots for civil engineering purposes have to be mobile, since the places of operation are unstructured and are in different locations.

POTENTIAL APPLICATIONS OF ROBOTICS IN SPECIFIC AREAS OF TRANSPORTATION ENGINEERING

Robotization of transportation operations will be a challenging task. Unlike manufacturing environments that are structured, field operations in transportation involve unstructured and unpredictable environments. Consider the task of excavating a roadside trench for installing fiber optic cables. While excavating the upper layers of the soil, the system may encounter fine soil. But on digging deeper, the system may encounter hard rock pieces for which it is not prepared. This discovery would require a cognitive robot that can adjust its capabilities to changing situations. Coupled with uncertainties, robots have to be sensitive to other factors, such as varying temperature conditions at the site, range, and magnetic fields.

Robots operating under field conditions should be able to tolerate extreme conditions of roughness of terrain. To perform successfully a robot built for field operations should be able to survive in field conditions. So, future robots for transportation should be capable of moving over and around obstacles and also of climbing grades.

In transportation, numerous application areas exist and are outlined below.

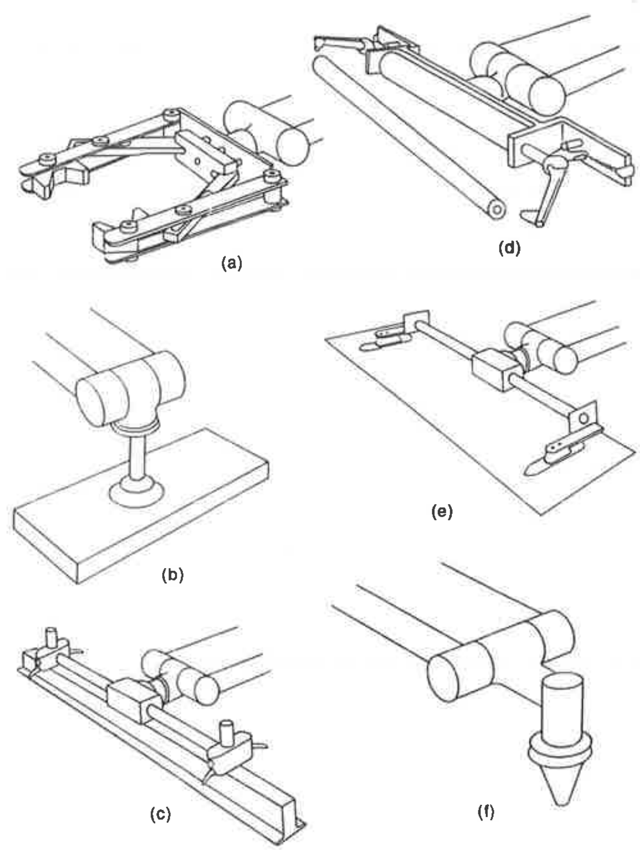

FIGURE 2 Robot Effectors (a) finger gripper, (b) vacuum gripper, (c) magnetic gripper, (d) tube gripper, (e) magnetic gripper, (f) grinder (*8*).

Steel Bridge Construction and Maintenance

Welding of Bridge Components

Robots routinely have been used in manufacturing plants for accurate welding of objects, with a good degree of success. Robots similarly could be used for welding purposes during steel bridge construction. Using one such robot developed in Japan, girders are manufactured by profile cutting of plate steel to the desired contours and then welding the cut sections together to form the I-beam. To provide adequate anchoring of concrete decking to the steel girders, a regular array of mushroom-headed steel studs is welded to the uppermost flange of the girder, as shown in Figure 3 (*10*). When the steel welding operation is performed manually, the stud positions must be marked before the welding operations can start. The marking is time-consuming: it is estimated that it takes one-third of the time attributed to the whole operation. When the process is automated, it does not require premarking. The welding (usually) takes a long time since there are typically 400 studs welded to one girder. Throughout the operation, the welder has to carry the gun, which is tiring and hazardous (*10*).

A tracking mechanism is needed to move the robot along the girder because girders are usually several meters long. In this case, the tracking means required to guide the robot along the girder can be obtained by direct use of the girder, as shown in Figure 4 (*10*). This welding application can be extended to the welding of splices and braces.

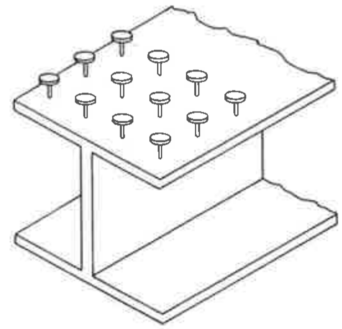

FIGURE 3 Array of steel studs welded to girder flange (*10*).

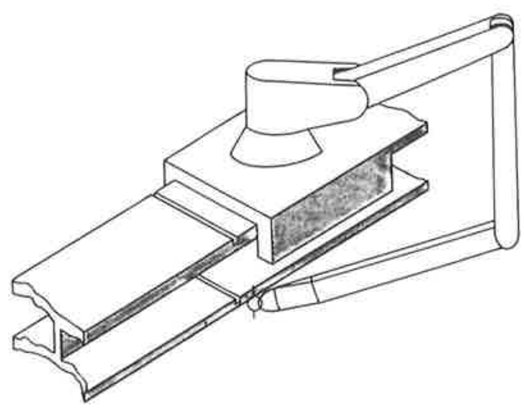

FIGURE 4 Robot setup for welding steel girders (*10*).

Bolting Connections

In railway bridges and in steel bridge work involving bolt and nut fastening, a robot similar to the one used for welding operations can be used. The effector at the end of the arm can be facilitated for gripping the bolt and tightening it into the nut, which is previously welded at the exact intended location by the welding effector.

Corrosion Protection

One of the most promising and cost-effective applications of robots has been shown to be surface finishing work. Basic surface operations include the following:

- cleaning and shaping (applying mechanical treatment to a raw surface to obtain better quality or utility); and
- coating and spraying (spreading a liquid substance on a structural surface to obtain better quality).

These repetitive and often hazardous tasks require protective equipment, continuous control, and high accuracy (*11*). As pollutant loads increase in air and water, and the threat and effects of acid rain increase, corrosion protection of steel facilities becomes a critical rehabilitation need.

The sandblasting process requires high-pressure spraying of air, water, or dust, which poses high risk to personal injury and requires expensive protection gear (*12*). Medical and statistical evidence claim that sandblasting processes pose a serious health hazard associated with lung silicosis to human operators. Sandblasting contractors estimate that the replacement of human labor with an autonomous robotic machine would result in an economic gain of up to 40 percent of the average human labor cost (*13*).

Sandblasting work is a highly specialized business with a multimillion-dollar turnover, in which productivity and efficiency are affected by human performance. Sandblasting jobs are tedious—typical maximum worker productivity is for only 4 hours a day. Overall daily productivity goes down by about 20 percent if the temperature is over 30°C (*13*). Robotization of sandblasting would result in the displacement of sandblasting crews by robot operators and possible savings in money for the contractor. These factors have been used as justification for introducing robots in surface finishing work (*11*).

Concrete pavement slab finishing work can also be a good application because this work involves long curing periods that force workers to work around the clock in tedious circumstances. A better-quality finish can also be expected with the use of robots.

Painting of Bridges

Painting bridges at high elevations requires costly scaffolding, which can be avoided by using a robot similar to the welding robot described earlier. The use of a robot also reduces the hazard of worker death by falling, which is a major cause of deaths in civil engineering operations.

A robot system that can climb up formwork rods to work at high elevations has been developed by the Fujita Corporation in Japan. The Climbing Jack Robot system is a robotized hydraulic jack equipped with a level detector and operating control device that grips and climbs a steel pipe rod. This system has been successfully applied in bridge pier construction. Currently, nine bridge piers have been constructed using this robot (*14*).

There is a great need for surveying and diagnostic tools that can provide condition and damage assessment information at reasonable prices to aid rehabilitation and repair efforts. It is clear that such tools have to be automated to provide quickness and accuracy in the operations. The ability of the tools to measure subsurface data will be invaluable in detecting and avoiding imminent failure of concrete pavement components. These operations are dependent on local factors and they demand equipment that is "flexible," "intelligent," and accurate. It is clear that automated data acquisition techniques coupled with robotics is the best answer to this need. Robots are also needed to carry out inspection work in inaccessible and dangerous locations. Some potential applications are discussed below.

Inspection of Concrete Pavements

Researchers at Carnegie-Mellon University, Pittsburgh, Pa., have developed a robot to evaluate existing reinforced concrete pavements. With its sensors, the robot checks the condition of reinforcing steel in the pavement and transmits data to a computer or video screen. Engineers are immediately able to detect the condition of the reinforcing bars in a nondestructive manner (*15*). This robot has sensors embedded in its base plate to indicate reinforcing bar location and size and the quality of concrete and how well it meshes with the subsurface (*15*).

By using sensors, robots can also be made to detect cracks, potholes, and warping on concrete pavements. These methods can be used without having to disrupt traffic for long.

In prestressed concrete beams, it is difficult to detect the exact location of steel elements. Robots could be used to hold up the steel elements in proper profile during the entire concrete pouring operation. Other applications are point-of-placement concrete quality testing and continuous weld quality monitoring.

Currently computers are doing some of the potential robot applications in this area. Ground-penetrating radars are used to detect voids under a jointed concrete pavement and a microprocessor-based system is used to conduct a road survey. This survey can be done at the rate of 5 lane-miles of pavement per hour with only minimal interruption of traffic (*16*). This survey information can be made available to robots for efficient location of voids. Recognition of displaced dowel bars, reinforcing bars, and prestressing tendons are other microcomputer-based technologies that can make robotization more versatile.

Fabrication of Steel and Reinforcing Bars

Robots are routinely used in the manufacturing industry for cutting operations. Steel elements for fabrication of bridge trusses can be accurately cut to the required specifications and configurations by using a robot.

Reinforcing steel fabrication is also possible with a robot. A reinforcing bar (rebar) robot will take rebar from a spool and cut, hold, bend, and weld the rod to any shape designated by a computer (*15*). These steps eliminate tedious and slow work usually associated with unwinding and cutting reinforcing bars, which also generates considerable cost savings.

Concrete placing has also been robotized. The Takenaka Corporation in Japan has developed the Horizontal Concrete Distributor, which has solved diverse problems involving moving the end-hose and relocating pipes, tasks that are labor intensive. The operating system of the robot is intelligent and sufficiently flexible to respond to any command given by the operator regarding the direction of movement. This robot has been applied widely and has enabled placing a total of 120,000 m^3 of concrete at more than 20 sites since 1981 and is still being improved for higher efficiency (*17*).

Other robots designed for concrete slab-finishing, e.g., MARK II (*18*) and SURF-ROBO (*19*), are also being employed successfully. The MARK-II robot was developed by the Kajima Corporation in Japan and is, technologically, fully developed to the current state of the art. It can be operated even by workers instead of engineers and is expected to find wide use.

SURF-ROBO has the capacity to finish a concrete floor surface of 300 m^2 per hour with the same finish and accuracy of a plasterer. Ten such robots are being used by the Takenaka Corporation in Japan at practically every one of their construction sites.

Excavation Work

Excavation is an excellent application of robotics because of its significance in scale and economic importance. Excavation operates on a universal and generic material (soil) and its goal and state can be adequately described by models of geometry and kinematics. Such diverse robotic excavation applications as off-shore ocean floor construction, pipe excavation for utilities, and repair of bomb-damaged runways are emerging (*20*).

A Robotic Excavator (REX) was first designed by researchers at Carnegie-Mellon University. REX integrated sensing, modeling, planning, simulation, and action, specifically to unearth buried utility piping. REX reduced the excavation hazard posed by explosive gases, decreased operation costs, and increased productivity. Soil was gently cleared as excavation progressed until the buried object was discovered (*20*).

A second-generation excavator developed later is a refinement on REX and is called GENEREX. This machine has better refinement in sensing because of the use of multiple sensors to acquire raw visual, tactile, and magnetic sensory data (*20*).

Magnetic sensing capabilities are important for detecting underground human-made objects such as buried pipes and cables. Robots equipped with such sensors will be able to distinguish buried pipes and cables from other buried objects such as tree roots and debris.

This application has special potential since major excavation activity is proposed to be done on the roadways all over the United States to replace conventional utility cables with fiber optic cables. Introduction of robots for excavation in such activities will reduce the cost per mile of these cable installations and make them more attractive to the utility companies and consumers.

Another important advantage in robotizing excavations is in the reduction of the number of deaths of workers because of cave-ins of excavated material. More than half of all construction accidents involve trenching and excavation, and there are hundreds of such fatalities every year; in addition, there are thousands of accidents in which people are injured and untold thousands of accidents in which no one is hurt but from which there is significant economic loss (*21*). These accidents happen because of assumptions that if a trench or excavation bank stands after the excavation is dug, it will continue to stand. Earth banks may have varying stand-up times ranging from a few minutes to several months, but cave-ins are not easily predictable.

In situations where deep excavations are needed, there is a need for long reach. Manipulators with a long reach have been designed for civil engineering applications (*22*). These manipulators have a reach of up to 52 m, with a capacity of 200 yd^3 per hour and are equipped with video data acquisition techniques (*22*). Such robotic arms can be used in large excavation work involving deep trenches since they help avoid hazards and also have a good output per hour.

Another possibility is to incorporate laser grading apparatus with these robotic arms to enable automated excavation grade control. These machines substitute for lower-cost machines (such as bulldozers and motor-graders) and low-skilled operators while yielding high-quality improvements (*23*).

A robot for real-time measurement of the degree of soil compaction has been developed for new pavement construction (*24*). This is a self-propelled robot with the unique capability of position measurement for guiding itself. A design has been proposed for a robot that performs pavement-cutting work (*25*) that can decrease the accident risk, noise level, and pavement surface slurry contamination.

Tunneling Operations

Tunneling operations often take place in dangerous situations involving cave-ins, the presence of hazardous gases, flooding water, and suffocation of workers. Workers perform strenuous and difficult jobs during tunnel excavations that are slow, thus lowering the quality and productivity rate. During tunneling operations, it is essential to furnish fresh air at the rate of 200–500 ft^3 per minute per worker and to remove noxious gases and fumes produced by explosives. It is also necessary to remove the dust caused by drilling, blasting, mucking, and other operations because silica dust causes lung diseases in workers (*26*). These tasks are not only costly but require enormous safety precautions.

Tunnel excavations can be performed quickly by a preprogrammed robot that ensures accurate alignment and extent of digging. Further, robots can work around-the-clock without getting exhausted.

The Japanese have developed automatic recorders attached to advanced soft-ground tunneling machines to record excavation volumes, advance rates, jack pressures, and other parameters to guide a "blind" tunneling operation (*2*).

Shotcreting is another robot application in tunneling. In the Austrian tunneling method, shotcrete applications take as much as 30 percent of the total time. Kajima Company in Japan has developed and implemented a computer-controlled applicator by which high-quality shotcrete is placed quickly (*11*).

Robots with vision sensors can detect defects in linings and jointing. End effectors can be used for spraying, grouting, and caulking operations in tunnel lining work.

A new automated tunneling method (M2) has been developed that is suited to underground conduit laying in congested heavy-traffic areas (*27*). This enables automatic transportation of excavated soil and lining material in the tunnel by an unmanned system. A 170-m resin tunnel under a public road has been successfully constructed in Japan by the M2 method.

A robot developed to inspect the inner linings of piping at a nuclear power plant helped shorten inspection work schedules by about 40 percent and helped reduce costs by 30 to 40%. This remote-controlled system reported the conditions of in-pipe scales and the condition of the inner lining (*28*).

Roadside Management

Each year the Department of Transportation in various states spends thousands of dollars in roadside maintenance. These operations include grass cutting, weed control, herbicide spraying, drainage inspection, and culvert maintenance. Some of these operations can be robotized for productivity improvements in automated roadside maintenance systems. Work inside culvert pipes having small diameters is difficult since they are not easily accessible. The inner linings of culvert pipes can be inspected by a robot that can identify the exact locations of damage. The same robot can be used to correct the damage in those locations by using appropriate end effectors.

Herbicide spraying is usually done manually or with the help of tractors. The chemicals used in the spray can cause harm to the lungs, skin, and eyes of workers. Robotization of this operation can reduce this hazard while improving productivity. Grass-cutting capabilities can also be incorporated into this robot, which can be equipped additionally with a laser grading device—another robot application for roadside maintenance.

Some other applications suggested for robots include underwater repair, changing lamps on lampposts, washing signs and luminaires, servicing vehicles, performing security patrols, and fabricating signs in the shop (*29*).

Robots also have the advantage of working silently compared with conventional equipment, thus reducing noise pollution around worksites.

Handling of Hazardous Waste Material in Preparation for Transportation

The issue of hazardous material handling is generating a lot of attention and research in the United States. Research has shown that robots can be used for cleanup of radioactive sludge from nuclear plants (*2*) and for maintenance and remote operation of nuclear spent-fuel processing plants (*30*). These applications can be extended to hazardous material handling operations associated with hazardous material transportation. As new roadways are built, chances of encountering hazardous waste dumps and landfills increase. Operations under these conditions require expensive protective gear and complex precautionary measures that can be substantially reduced if robots are used for excavation. Computers are helping to maintain an efficient hazardous materials transportation information system to report incidents of hazardous material releases (*31*). A computerized aerial photographic analysis has proven to be a highly cost-effective and accurate predictor of the location of hazardous waste contamination. Aerial photographs are used as inputs to a digital mapping system and the overlays are generated to identify specific configurations of the clean-up sites (*32*).

Railway Track Maintenance

Railway tracks must be inspected and maintained regularly to provide safe and efficient operation of the railway system. Robots can be used to inspect sleeper bolt connections, fish plates, cracks in old rails, and railway bridge structures. Vision sensors would be of use in this application. Robots can also be employed to weld corroded and cracked rails.

Some other robot applications in railways are sequence operations in shunting yards; building and repair operations of traffic structures, including application of multiple spindle screw machines, sleeper bolt screws, etc.; transportation, transfer, and storage processes in automatic transportation systems, for the automation of stock-keeping or handling of installations (e.g., for wagon unloading), and service processes for transportation facilities with a special emphasis on cleaning devices (*33*).

Interfacing with Digital Mapping Systems

Surveying and mapping have traditionally played a central role in all phases of an engineering project. In the 1980s, the need for computerized graphic data bases has prompted the development of geographical information systems (GIS) and satellite surveying techniques such as the Global Positioning System (GPS) for real-time operations. It is predicted that the development of real-time surveying and mapping techniques will achieve top research priorities in the next decade. Supported by vision systems such as GPS, vital functions such as construction layouts, inspection, change detection, and quantitative measurements as well as robot guidance can be done (*34*). Digital mapping systems such as ENMIT allow for graphic information on roadside management planning. ENMIT facilitates identification of damaged roadside vegetation and wetlands from a large geographical data base (S. M. Naik and W. C. Hall, Unpublished data, 1989). The information derived from such a data base can be used to guide robots meant for roadside maintenance. State agencies proposing to invest millions of dollars in GIS software and hardware in the near future will find that robots can derive guidance for field operations from these systems.

Space Applications

The successful use of a robotic arm to retrieve a nonfunctional satellite from its orbit for repair by a U.S. space shuttle has demonstrated the need for robots in space transportation operations. As civil engineers prepare to discuss their role in space exploration and exploitation (*35*), the need for robots in planning, maintenance, and operation of facilities in space will be further emphasized.

USE OF EXPERT SYSTEMS FOR ROBOT TASK PLANNING

Robotization of transportation operations will definitely bring about changes in the planning approach of engineers. Because equipment operators are inexperienced in the handling of robots and because they and managers are familiar with traditional techniques only, managers are concerned about their ability to oversee robot-related tasks in the construction process. Managers may fear low productivity from robots and delays that may affect the cost and duration of projects.

Computer-aided design and drafting has revolutionized the civil engineering industry. This technology has successfully been performing some of the potential robot applications discussed in this paper. The processes of analysis, evaluation, and synthesis of design data can be shared in an interprocess communication system (*36*). Automated data acquisition for field operations is performed with the use of minicomputer-based recording and monitoring instruments. Automated monitoring of construction quality control, production rates, and quantities is being done with the use of time-lapse photography, laser guidance systems, remote sensing, and GIS. These technologies can be used as supplements to robots to make them more versatile. Methods of using artificial intelligence techniques such as knowledge-based expert systems have been suggested to generate construction plans suitable for the constraints and conditions of robotized construction. These are used to create a model to serve as a data structure for poorly designed construction plans and schedules (*37*).

Pavement management systems are becoming increasingly useful microcomputer-based tools. These maintain a large data base of meaningful information on roadway deficiencies. Condition survey information based on foremen's evaluations of highway deficiencies are used in conjunction with parameters such as average daily traffic, pavement type, the level of resurfacing needed, and the condition of the shoulder to help keep roadway maintenance costs low.

The most significant research effort in computers related to robotics has been the advancement in machine vision. The advantages of vehicle detection through image processing are several. Image processing can simultaneously detect traffic, derive traffic measurements, perform surveillance, and recognize special vehicles. It accommodates future advanced truly dynamic control strategies for both arterial networks and freeway corridors (*38*).

Recent research in machine vision has presented three-dimensional machine vision, enabling measurement of light intensity and, thereby, distinctions between colors and textures. In spite of the complexity of machine vision systems, the field is booming today. A representative vision module, designed to supplement a robot system, costs from $15,000 to $40,000 and is expected to cost less as the industry matures (*39*).

The growing field of expert systems has continued to offer many benefits to the transportation industry. Scheduling, cost control, estimation of quantities, intersection design, feasibility analysis, structural analysis, and environmental management are just a few of the applications of expert systems in transportation.

Expert systems can enable robots to decide between multiple alternatives in motion and choice of action. An expert system framework for decision making on implementing robotics in construction has been proposed (*40*). The system's reasoning synthesizes available decision-support tools and considers technical and economic criteria for its decisions. Implementing this system will be useful when, in the future, construction robotics designers need feedback.

MANAGERIAL AND SOCIAL PROBLEMS OF INTRODUCING ROBOTICS IN TRANSPORTATION ENGINEERING

The introduction of robots in transportation engineering will definitely cause significant changes in the transportation sector as a whole. Although productivity, performance, and cost savings show potential dramatic increases with robotization, there are also several inhibiting factors.

Construction is an unfamiliar area for robot manufacturers. An April 28, 1983, *Engineering News-Record* report on the 13th International Symposium on Robots states, "of the 260 robots on display, . . . not one was earmarked for construction applications" (*41*, p. 30). Furthermore, conference exhibitors were unaware of any jobsite applications in the United States.

Another problem is that robot manufacturers are reluctant to push forward this technology. They insist that many basic and developmental problems need to be resolved before robotics can play any significant role in construction. To begin with, manufacturers do not have a complete understanding of the demands of the highway construction environment. According to one manufacturer, the Swedish ASEA company, "construction jobs are not always the same. So there's not a great deal of repeatability. Most construction jobs require a certain amount of on-site judgement, which a robot cannot provide. And if I leave a robot out in the rain, I lose $100,000 by the end of the week" (*41*, p. 32). One university researcher observed that development in the construction industry is driven by the marketplace and application is limited to locations of high hazards such as radiation, toxic wastes, and explosives (*41*).

Further, there is very little research in the construction industry. The main reason why Japan has been able to robotize construction operations is because robotics research is done by the construction industry itself. Companies like Shimizu Construction Company, Kajima Corporation, and others have developed robots for construction tasks and are using them successfully in the field (*18*, *25*). Robot manufacturers have not been shown that construction is an attractive field for robotization. They agree that they are too busy chasing the industrial market to go after unknowns in construction (*41*). Social and economic barriers such as fragmentation of the industry, risk and liability, and lack of standards and codes are also cited as obstacles to this advancement (*6*). "Intelligent" robots developed to adapt to changing environments rather than to structured environments are preferred over present-day robots because they allow the benefits of applying artificial intelligence and expert systems research to best emerge. Major factors inhibiting robotization can be unemployment of personnel displaced by robots, work capitalization for specialty subcontractors, unpredictability related to a new technology, risks of litigation, and inadequate funding for further research.

Major factors encouraging robotization can be the increasing awareness of the need for better productivity and performance for survival in a fiercely competitive international and domestic market, the dearth of human skill, the slow growth of research, and marketplace challenges for better performance.

Management is also becoming more flexible as managers bring better educational backgrounds and skills into managerial positions. However, a cautious response to robotization from worker unions has to be expected as the field of civil engineering adopts and adapts itself to robotization.

More fundamentally, the introduction of new technology (such as robots) brings with it the prospect of revamped construction procedures at the site, which promise radical improvements in productivity and working conditions. Not only will labor skills need to be upgraded, but also the labor-machinery interface will need to be revised; indeed, the entire structure and planning of construction activities may require rethinking.

SHORT-TERM GOALS FOR ROBOTICS IN TRANSPORTATION

As recognized in this paper, research in robotics is forthcoming, although slow. Further, most of the research is being done in Japan and by their construction industry itself. This needs to be emulated by the construction industry in the United States.

Further advancement in machine vision, expert systems, natural language processing, neural networks, and network communications are near-term goals beneficial to robotics. The construction material industry should manufacture structural components suitable for assembly by robots in a manner similar to that of the automotive industry. This type of system would facilitate the increase in the variety of robots assembled by the robot manufacturers. An efficient nationwide network of "build-to-assemble" component industries and local distribution of such components would parallel the distribution of robots.

The most important short-term goal is the synthesis of all robotics-related data to form a comprehensive knowledge base. This system should contain knowledge of not only the technology but also the input data suitable for available robotic systems. The types of data include numeric, graphic, and image with access to on-line construction data bases. Such a unification of knowledge would disseminate awareness about robotics among transportation professionals.

The idea of using robots is slowly being accepted by transportation research committees. A forthcoming conference on advanced technologies in transportation engineering has set aside a separate tutorial session on robotics. This trend should be strengthened so that all research organizations recognize the urgency to stay abreast in this promising technology.

CONCLUSIONS

The benefits of adopting advanced technologies in transportation have been inadequately recognized so far, resulting in overlooking possibilities to make substantial and important changes to infrastructure maintenance and rehabilitation work.

At current levels of progress, this work will be expensive in terms of time and cost. Hence, there is a great need for a radical change in the methods, materials, and equipment used in the construction, rehabilitation, and maintenance of transportation facilities. Successful automation in various areas has demonstrated the reliability and cost-effectiveness of using robots. Hence, much of the future growth in transportation must be in adopting new technologies, among which robotics is important and attractive. To make progress, robot manufacturers must envision that robots must handle construction jobs under uncontrolled and harsh environments, such as the extreme winter cold in Alaska, and sandstorms and 120°F heat in the Middle East. They work at high elevations during winter, and on road clearing and repair operations under snowy conditions. These are situations in which workers can get welcome relief from harsh conditions and the risk of fatal accidents. Robot manufacturers are right when they present the basic and developmental problems that need to be resolved before automated data acquisition, robotics, and process control technologies evolve to the point at which they will play a significant role in the transportation sector. Many of these problems could form exciting long-term basic research topics if researchers are made aware of the potential of these technologies. Focusing the attention on high-technology research in this area requires considerable mutual understanding with researchers in other high-technology fields, with the major initiative coming from civil engineers. A sincere effort is needed on the part of the government, research agencies, universities, and private industry to evolve a systematic and vigorous research effort to advance the trend of research in transportation engineering robotics.

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Determination of Construction Equipment Rental Rates in Force Account Operations for Federal and State Government Agencies

JENNIFER JORGE AND ZOHAR HERBSMAN

Construction equipment rental rates vary widely according to factors such as type and age of the equipment, capacity, estimated operating costs, availability, and the geographic and climatic conditions at the job site. In 1986, the Florida Department of Transportation retained the services of the Engineering and Industrial Experiment Station at the University of Florida for the purpose of developing a model method for the determination of construction equipment rental rates for the state of Florida. Research of the cost factors involved in the determination of equipment rental rates was conducted in the form of a nationwide survey of state departments of transportation and of several federal government agencies, analysis of the methodology used by rental rate guides, and a study of federal guidelines concerning equipment cost reimbursement. The method thus developed, if implemented, would establish compensation rates for equipment force account work. This method could replace both the negotiation process—which may lead to inequitable compensation for the same equipment services—and the use of nationwide guides of average rates in which the user has no control over the costs and formulas applied. Research clearly indicates that the use of the developed method could result in considerable cost and time savings and in fewer claim disputes by providing standard, realistic, and equitable compensation to contractors for their use of construction equipment in force account work for governmental agencies.

Competitive bidding is the method most commonly used by state departments of transportation (DOTs) for the purpose of awarding construction contracts. In this case, the contractor is paid by a lump sum fee or by a unit price agreement. In both instances, the government agency pays a flat fee, without requiring specific invoices of all the components involved in the calculation of fees. However, there are many cases in which government agencies award noncompetitive construction contracts and pay contractors for actual costs incurred in the execution of contractual operations. Work so awarded is commonly known as force account work when it can be placed under any of the following three categories:

1. Payment based on a cost–cost-plus system, used for small projects, emergency operations, and other cases to which competitive bidding does not apply.
2. Payment for supplemental agreements, for additional necessary work beyond the scope of the original project.
3. Payments for claims obtained by contractors through some judicial procedure. This type of arrangement involves a reimbursement method similar to the cost-plus method.

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The common denominator in these three categories is that, in each case, government agencies reimburse contractors for actual costs of labor, materials, and equipment. Cost control for labor and materials is a relatively simple matter of bookkeeping; however, equipment costs are decidedly more difficult to determine. Construction equipment rental rates represent a considerable portion of the costs incurred by contractors when they submit claims for force account work to DOTs. Most DOTs use rental rate manuals to determine equipment reimbursement. Past records show that depending on the manual used, the rental cost for the same piece of equipment can vary from 10 percent to well over 400 percent and that such large variances occur for practically all types of equipment.

RESEARCH PROGRAM DEVELOPMENT

The purpose of this research was to specify the necessary parameters and to recommend a method for determining rental rates for construction equipment for the Florida Department of Transportation (FDOT). The objectives of the research program were as follows:

1. To evaluate the cost components involved in the calculation of construction equipment rental rates.
2. To review FDOT procedures for determining construction equipment rental rates.
3. To conduct a nationwide survey of the methods currently used by departments of transportation in all states.
4. To investigate the methods used by other governmental organizations such as the FHWA, the U.S. Army Corps of Engineers, and the U.S. Navy; and private organizations such as equipment dealers and large construction companies.
5. To develop guidelines and recommendations for the determination of rental rates for equipment used in FDOT force account projects. These guidelines would serve as the basis for the development of an analytical model that would determine equipment rental rates according to ownership costs, operation and maintenance costs, operating hours, equipment overhead, and profit. The result would be a rental rates guide for the FDOT that could be updated on a regular basis to reflect changes in policy and actual market conditions.

Equipment cost is the major cost item in many construction projects. Because the problem of setting equitable and realistic compensation for the use of construction equipment is common to most DOTs and to some federal government agencies, the results of this research should be of significant value to all of these organizations.

COST COMPONENTS OF RENTAL RATES

There are three categories of cost components used in deriving equipment rental rates: ownership costs, operation costs, and miscellaneous costs.

Ownership Costs

Ownership costs (C_{own}) cover all the expenses involved in the acquisition of a particular piece of equipment. C_{own} are based on the total purchase price, the economic life period, and the estimated salvage value of the equipment at the end of that period. The cost items under C_{own} are depreciation, return on investment, applicable taxes, and other costs such as insurance, storage, and replacement escalation.

Depreciation

Depreciation is a non-cash flow accounting method for determining the loss in value of an investment over a period of time. It is a function of time and use, and it is caused by unrepaired wear, deterioration, obsolescence, and reduced demand (*1*). The method used depends on the purposes of the depreciation. For internal accounting, the straight-line method—which depreciates the cost into equal portions throughout the life of the equipment—is commonly used. For tax purposes, most companies will use an accelerated depreciation method, since it would provide for a larger share of the cost to be written off in the earlier years of equipment ownership. Common accelerated depreciation methods include the declining balance, the sum-of-the-years-digits, the accelerated cost recovery system (ACRS), and the modified accelerated cost recovery system (MACRS). The latter two are schedules of allowable yearly percentages of depreciation published by the U.S. Internal Revenue Service. These two purposes are divergent and result in two different methods of depreciation used on the same capital item. This double accounting procedure is legal and used openly in the construction industry. However, true depreciation can be measured only by determining the market value of the equipment at a particular point in time.

The following are formulas to calculate depreciation amounts:

- Straight-line method (SL):

$$P - SV/N \tag{1}$$

- Declining balance (DB):

$$P \times (Factor/N) \tag{2}$$

- Sum-of-the-year's digits (SOYD):

$$P - SV \times (D/SOYD) \tag{3}$$

where

P = present value;
SV = salvage value;
N = useful life;
$Factor$ = 1 for declining balance, 2 for double declining balance;
D = that year's digit; and
$SOYD$ = sum-of-the-year's digits.

Table 1 and Figure 1 show a comparison of depreciation methods for equipment valued at $200,000, with a useful life of 5 years and a salvage value of $50,000.

Return on Investment

The purchase of equipment is normally considered an investment venture. As such, it is made on the premise that the investment will ultimately yield a rate of return high enough to compensate for the risk of the undertaking. Thus, interest is charged on the capital used for purchasing the equipment, whether the purchase is financed by an institution or paid for with company funds. In the latter case, the allowance of interest charged is made to compensate the company for the rate of return that its funds would obtain if deposited in a banking

TABLE 1 COMPARISON OF DEPRECIATION METHODS ON A 5-YR SCHEDULE

	Straight Line		Double Declining Balance		Sum of the Years' Digits	
Year Digit	Value @ End of Year	Year's Depreciation	Value @ End of Year	Year's Depreciation	Value @ End of Year	Year's Depreciation
1	$200,00	$30,000	$200,000	$80,000	$200,000	$50,000
2	170,000	30,000	120,000	48,000	150,000	40,000
3	140,000	30,000	72,000	22,000[a]	110,000	30,000
4	110,000	30,000	50,000	0	80,000	20,000
5	80,000	30,000	50,000	0	60,000	10,000
6	50,000	------	50,000	-----	50,000	-----
TOTAL Depreciation		$150,000		$150,000		$150,000

[a] Depreciation for the year must be adjusted so that it will not decrease the salvage value of $50,000.

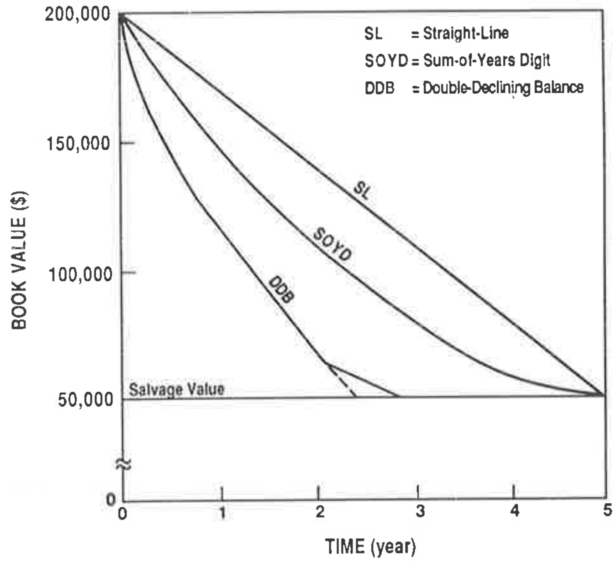

FIGURE 1 Comparison of depreciation methods.

institution or if used for another investment that would yield an acceptable rate of return. Although there are several methods for calculating the actual cost of investment, the most common one is the amortization method, which is used by most financial institutions. The general formula for the amortization method is

$$A = P\,[i(1 + i)^N]/[(1 + i)^N - 1] \quad (4)$$

where

A = periodic amount to be paid to amortize debt,
i = interest rate per time period,
N = number of time periods in amortization schedule, and
P = present cost of equipment.

Taxes

Several forms of taxation enter the caculation of equipment costs. Sales taxes, income taxes, property taxes and any investment tax credits allowable under law should all be considered.

Insurance

Insurance costs include coverage for fire, theft, comprehensive liability, and damage to the equipment.

Storage

Storage expenses include the rent and maintenance of space (storage yard, warehouse, etc.), handling of equipment in and out of storage, and any security—security guard, mechanical security systems—needed.

Replacement Escalation

Historically, construction equipment prices have increased about 5 percent per year for the past 20 years. This escalation in price is because of factors such as inflation, new technology, market conditions, and changes in taxation laws. Although an estimate of the escalation amount can be found, it is almost impossible to predict escalation with any amount of certainty. As of this writing, the FHWA does not allow replacement escalation costs to be paid as part of the costs of rental construction equipment for work subsidized by the U.S. federal government.

Operation Costs

Operation costs (C_{oper}) are those costs associated with use of the equipment. They include the costs of maintenance, repairs, fuel, and lubrication. The type of equipment, the conditions under which it is used, and the location of the project are all factors that will influence the cost of operation.

Maintenance and Repair

Maintenance repair costs (C_{rep}) include scheduled services of a preventive nature and repairs caused by unforeseen mechanical breakdowns. The estimation of maintenance costs can, therefore, be made more accurately than for repair costs. The best way to estimate future maintenance and repair costs is to use past records as a guide whenever available. The expenses under this category are the costs of parts, sales taxes, labor, fringe benefits, shop overhead, any supporting facilities, and any maintenance equipment required. The annual C_{rep} may be expressed as a percentage of the annual cost of depreciation. For example, it has been established that for cranes, the average annual C_{rep} ranges from 40 to 50 percent of the annual straight-line depreciation of the equipment. For draglines and clamshells, the range is 60 to 70 percent (*2*). When accelerated depreciation is used, tables for typical total lifetime costs of operation can be used and C_{rep} may be calculated by the following equation (*2*):

$$C_{rep} = ((D/SOYD) \times TC_{rep}/m \quad (5)$$

where

C_{rep} = hourly cost of maintenance and repair for year D,
D = that year's digit,
$SOYD$ = sum-of-the-year's digits,
TC_{rep} = typical total lifetime for equipment life maintenance and repair cost, and
m = operating hours in that year.

Fuel

Under standard conditions (barometric pressure of 29.9 in. of mercury at a temperature of 60°F) a gasoline engine will consume approximately 0.06 gal of fuel per flywheel-horsepower-hour (fwhp-hr). A diesel engine will consume 0.04 gal per fwhp-hr. Under severe conditions, the consumption of fuel can increase by up to 30 percent (*3*). Engines that seldom operate at the rated output or at a constant output for long periods have a reduced fuel demand because of time and operation factors. The hourly cost of fuel (C_{fuel}) may then be calculated by

$$C_{fuel} = \frac{\text{operating}}{\text{factor}} \times \frac{\text{time}}{\text{factor}} \times \frac{\text{rated}}{\text{fwhp}} \times \text{(gal of fuel/fwhp-hr)} \times \text{cost per gal} \quad (6)$$

Lubrication Oil

The quantity of oil used by an engine depends on the engine size, the capacity of the crankcase, the condition of the piston rings, and the number of hours between oil changes. When past records for a particular type of equipment are not available, the hourly cost of oil (C_{oil}) can be calculated in one of two ways:

$$C_{oil} = [(fwhp \times f \times n)/r] + c/t \quad (7)$$

where

$fwhp$ = rated flywheel horsepower of engine,
f = operating factor,
n = 0.006 lb of oil,
c = gal capacity of crankcase, and
r = 7.4 lb/gal.

A simplified way to calculate the hourly cost of lubrication oil is to consider it as a percentage—between 10 and 30 percent—of fuel cost.

Miscellaneous Costs

Several other cost items can be considered under miscellaneous costs. Among these are mobilization to the jobsite, required safety inspections, and other expenses that may apply depending on the type of equipment and the particular contract under which the equipment is operating.

A summary of all the components involved in the calculation of rental rates is shown in Figure 2.

SURVEY OF CURRENT PRACTICES

A written survey on current construction equipment rental practices was sent to all 50 state departments of transportation and to organizations such as the FHWA and the U.S. Navy.

Departments of Transportation

The survey findings in Table 2 show that 38 states use the *Rental Rate Blue Book* (Blue Book) by Dataquest, Inc. (*4*), as indicated in the standard specifications for their state. Many states arbitrarily adjust the rates on the *Blue Book*. As examples, Nebraska, Wyoming, and Mississippi use 77, 68, and 80 percent, respectively, of the blue book rates. For most of the states the *Blue Book* rates are used in lieu of negotiation. The states of Colorado and Illinois have developed their own rental rates guides. As of this writing, only the state of Massachusetts has adopted the use of the Construction Equipment Ownership and Operating Expense Schedule published by the U.S. Army Corps of Engineers. The state of Florida currently uses 1/176th of the monthly *Blue Book* rental rate as the hourly rate.

FHWA

The FHWA has issued a policy guidance to ensure that predetermined equipment rental rate used by state highway agencies comply with federal cost principles whenever force account

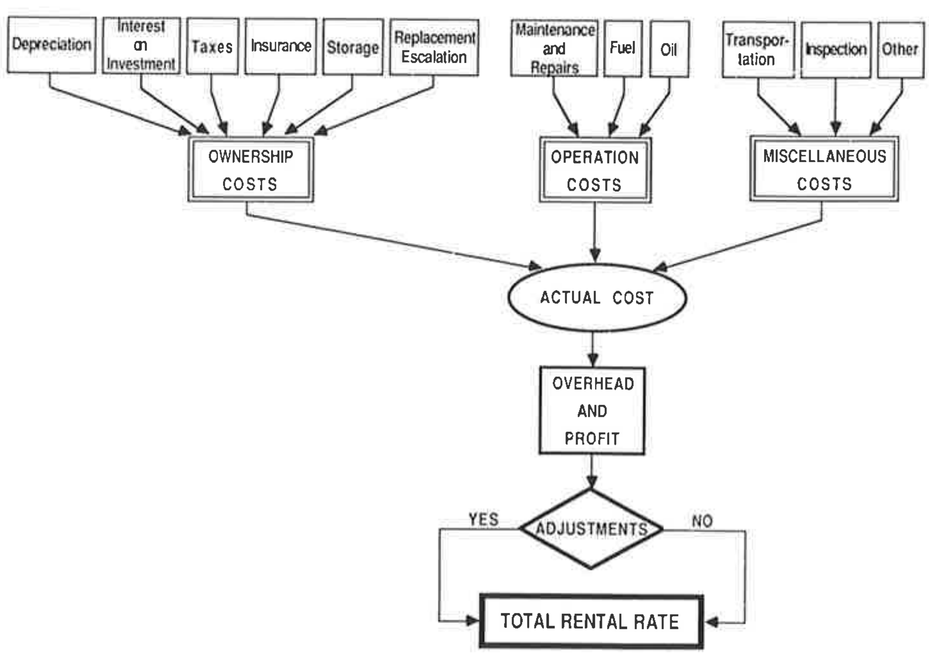

FIGURE 2 Rental cost components.

TABLE 2 TABULATION OF STATE DEPARTMENTS OF TRANSPORTATION RESPONSES

State	Blue Book	COE	AED	DOT	State	Blue Book	COE	AED	DOT
Alabama	X		X		Montana	X			
Alaska	X				Nebraska	X			
Arizona	X				Nevada	X			
Arkansas	X				New Hampshire	X			
California	X			X	New Jersey	X			
Colorado				X	New Mexico	X			
Connecticut	X				New York	X			
Delaware	X				North Carolina	X			
Florida	X				North Dakota	X			
Georgia	X				Ohio	X			
Hawaii	X				Oklahoma				X
Idaho	X				Oregon	X			
Illinois				X	Pennsylvania	X			
Indiana			X		Rhode Island	X			
Iowa	X				SouthCarolina				X
Kansas	X				[a]South Dakota				X
Kentucky	X				Tennessee	X			
Louisiana				X	Texas				
Maine	X				Utah	X			
Maryland	X				Vermont	X			
Massachusetts		X			Virginia	X			
Michigan				X	Washington	X			
Minnesota	X				West Virginia	X			
Mississippi	X				Wisconsin	X			
Missouri	X				Wyoming	X			

AED = Association of Equipment Distributors
COE = Corps of Engineers
DOT = Department's Own method
a = Uses Colorado's method

work is necessary. Federal policy requires that actual costs be used to determine extra work payments. However, since actual equipment costs may not be readily available, the FHWA permits the use of predetermined rental rates guides, as long as they are modified to exclude costs that are ineligible for federal reimbursement. The ineligible costs are those of contingency expenses and of replacement escalation. Contingency factors are not allowed under Circular A-87, Attachment B, Part D-2 (Standards for Selected Items of Cost) of the Office of Management and Budget (OMB). Replacement escalation costs are not allowed under OMB Circular A-87, Attachment B, Section B-11, which states that the computation of depreciation or use allowance is to be based on original acquisition costs.

U.S. Navy

The U.S. Navy follows the guidelines set by the Federal Acquisition Regulations 31.15, which applies to direct government procurements. This regulation identifies the U.S. Army Corps of Engineers as an acceptable source of equipment rental rates but does not require its use. Thus, the U.S. Navy does not have a standard method for establishing equipment rental rates on force account work. Rather, it is left to the individual divisions to negotiate or use a published rental rates guide, as long as the rates on the guide are adjusted to comply with federal regulations.

CURRENT RENTAL RATES SOURCES

Rental Rate Blue Book

The *Rental Rate Blue Book* has been published since 1960 by Dataquest, Inc., a company of the Dun & Bradstreet Corporation. Its introduction states that "The rates in this manual are intended as guidelines paralleling amounts an equipment owner should charge during rental or contractural periods to recover equipment related costs" (*4*, Section A, p. 1). Volume 1 is a comprehensive cost recovery guide for equipment manufactured during the past 5 years, volume 2 is for equipment that is from 6 to 10 years old, and volume 3 is for equipment that is 11 to 20 years old. The rates published in the *Blue*

Book are calculated from cost formulas and factors developed from Dataquest research. Although some general formulas used are shown in the introduction section, it is not clear how Dataquest actually calculates all the cost factors involved. Direct inquiries to Dataquest failed to clarify their methodology. Survey responses indicate that many states feel the rates in the *Blue Book* are too high and not representative of actual costs, and that is why adjustments to decrease these rates have been implemented. The disadvantages of using private publications in general is that the user has no control over the methods and formulas used and that the costs of overhead and profit may be incorporated within individual components, thus compounding these factors and making it difficult to identify actual costs.

U.S. Army Corps of Engineers

The Construction Equipment Ownership and Operating Expense Schedule was developed by the U.S. Army Corps of Engineers in 1979. It establishes predetermined equipment ownership and operating expense hourly rates for use in the preparation of estimates and in the pricing of negotiated procurements requiring independent government estimates. This guide shows in detail how each cost component is calculated. No allowances are made for operating labor, mobilization costs, overhead, or profit (*5*). The advantage of using this guide is that it fully complies with Federal Acquisition Regulations. However, the general consensus of highway agencies is that the rates in this schedule are considered too low, especially when compared to the most commonly used guide, which is the *Blue Book*.

Colorado Department of Highways

The Colorado Department of Highways (CDOH), and the Colorado Contractors Association, through a joint committee, have reached agreement on the procedures to determine rental rates for equipment used in force account work. These procedures are incorporated into the *Construction Equipment Rental Rate Schedule* published by the CDOH (*6*). The current version of this schedule is based on the 1974 publication of the *Contractor's Equipment Ownership Expense Schedule* by the Associated General Contractors organization. Modifications are made to update for costs of fuel, oil, grease, and filters. "This schedule was then computerized and is in use today. The committee agrees that the 1974 data would appear obsolete, however, the repair and ownership percentages are considered applicable to today's equipment values" (G.W. Fritts, Colorado Department of Highways, personal correspondence to Z. Herbsman, 1987). The rates on this schedule do not compensate for operator wages, fringe benefits, or profit. Replacement escalation factors are also excluded, and interest rates are revised annually to reflect market conditions.

Conclusions on Current Practices

Table 3 shows several published rental rates for comparison purposes only. From this table it can be seen that rental rates for the same piece of equipment may vary from 10 to over 400 percent, depending on the rental rates guide used. At present, the majority of state highway agencies (76 percent) use the *Blue Book* of nationwide averages of rental rates as

TABLE 3 COMPARISON OF PUBLISHED 1986 RENTAL RATES[a]

Description	Hourly Rates ($) including operating Costs				
	Blue Book (Jan 1986)		U.S. Army Corps (June 1986)	Colorado (May 1986)	Illinois (Jan 1986)
	Unadjusted Hourly Rate	As adjusted by FDOT			
1. P&H Self-Propelled Hydraulic Crane Model Omega 115, 15 tons, diesel powered	79.10	40.71	21.66	39.55	57.75
2. Caterpillar Wheel Loader, Model 992C, 13.5 c.y., diesel powered	467.30	206.99	152.51	304.85	340.73
3. Caterpillar Motor Grader, Model 16-G, 250 HP, diesel powered	106.95	82.35	49.82	105.00	123.75
4. MKT Pile Driving Hammer, Model DE-70B/50B, 30,000 ft-lbs., diesel powered	87.30	32.72	18.61	48.40	53.90

[a] 1986 rates used for demonstration purposes only

standard reference for compensation of force account work. Many of these states arbitrarily adjust the published rates because of the inherent diversities in geographical working conditions throughout the country, and the distinct policies of highway agencies, which may not be the same as those used by the rental rates guide.

It is recommended that highway agencies that are currently modifying rental rates guides establish their own fair policies and standard guidelines regarding allowable equipment costs in force account work. The rental rates calculated by using the established criteria would ensure that the policies of the user agency are being enforced.

RECOMMENDATIONS FOR CALCULATING CONSTRUCTION EQUIPMENT RENTAL RATES

Outlined below are specific recommendations for the calculation of the necessary costs needed to establish rental rates for construction equipment.

Ownership Costs

It is recommended that the items under C_{own} be calculated as one total number. The major cost items are depreciation and interest on investment. The other C_{own} components (taxes, insurance, storage, and replacement escalation) can be grouped together as an added percentage above that used for calculating the interest on investment. The following formula can be used to calculate the hourly cost of ownership.

$$C_{own} = \{P\,[i(1 + i)^N/(1 + i)^N - 1] - SV/N\}/m \qquad (8)$$

where

P = purchase price of the equipment,
i = percentage rate for interest, taxes, insurance, storage, and replacement escalation,
N = useful life (in years), and
m = operating hours in a year.

Depreciation

- The initial cost of any construction equipment should include the costs of the basic machinery, of necessary accessories to make the equipment operational, and any applicable sales taxes, transportation, inspection, and assembly costs. These costs can be obtained from industry sources, such as the manufacturer's list price, or from guides such as the *Cost Reference Guide for Construction Equipment* (*7*), which publishes list prices.
- Research data indicated that a list price discount rate of between 5 and 15 percent is commonly applied to construction equipment; therefore, a discount percentage should be taken into account when establishing rental rates.
- Straight-line is the recommended method of depreciation for equipment in regular use. For excessive wear and tear, it may be necessary to use an accelerated depreciation schedule.

Salvage value

- Depending on the type of equipment, the average salvage value at the end of the economic life can range from 10 to 35 percent. However, lower and higher values can be encountered, depending on the conditions of the equipment as well as the predominant market conditions. It is best to use up-to-date publications such as the *Green Guide* (*8*).
- Because of the unpredictable state of the economy, it is not possible to estimate with certainty the present value of a future sum, as is the salvage value. It is, therefore, recommended that no provisions be made for inflation and deflation, since these will be better reflected in the current resale values published.

Return on investment

- It is suggested that the interest rate used be the monthly equivalent of the sum of the current yearly cost of money rate as determined by the U.S. Department of Treasury plus 2 percent per year to account for fluctuations in the treasury rate. Depending on the policy of each agency, a varying percent allowance may be added to compensate for the inherent risk of investing in construction equipment. The cost of capital is a major expense and should, therefore, be adjusted periodically to reflect actual economic conditions.
- The amortization method with a monthly schedule is the most accurate method for determining the cost of interest on investment, since it is the method preferred by financial institutions.

Cost of taxes, insurance, storage and replacement escalation

- If the investment tax credit is reinstated by the federal government, it should be considered as an added discount on the original purchase price.
- Income taxes are not considered an expense, but are part of the distribution of total profits of the company (*9*). As such, it is recommended that no allowance be made for these taxes.

  — The cost of sales taxes is part of the original purchase price and should be included as part of the total purchase price of the equipment.
  — Property taxes are also considered indirect costs of the company and as such should not be included under rental rate costs.
  — The cost of insurance can generally range between 1 and 5 percent of the equipment value. This range can vary due to the type of equipment as well as the history of the individual contractor.
  — The cost of storage fluctuates depending on the location of the work area but generally can be considered to be a yearly 1 percent of the equipment value.
  — Replacement escalation costs are not allowed under FHWA regulations (*10*). It is recommended that no allowance for this expense be provided, because this cost can be recovered when the new equipment is depreciated.

Operation Costs

Operation costs can be subdivided into two categories: maintenance and repair costs and fuel and oil costs.

Maintenance and Repair Costs

Maintenance and repair costs are a major component of any rental rate. If past records are not available, it is suggested that this item be calculated with tables of typical lifetime repair costs as percentages of the initial price. These tables can be found in construction equipment textbooks (*2*). Manufacturers and equipment dealers also have maintenance and repair data for specific types of equipment.

Fuel and Oil Costs

If no past records of fuel and oil costs are available, a good approach would be to use validated formulas to calculate these costs. Several formulas are found in current equipment literature, and they can be adjusted for operating and time factors, as well as the costs of fuel and oil. For the sake of simplification, oil consumption usually can be considered to be a percentage (between 10 and 30 percent) of fuel costs.

Miscellaneous Costs

When considering miscellaneous costs, the following are recommended:

- That the costs for mobilization to the job site be reimbursed as a lump sum; and
- That the costs of necessary inspections and any other miscellaneous costs directly related to the equipment in operation be reimbursed to the contractor.

Adjustments

The above recommendations are intended to be used for the calculation of the total actual cost of owning and operating a piece of equipment in a force account operation. To determine

TABLE 4 EQUIPMENT DATA INPUT

EQUIPMENT DATA

Agency:	Equipment Classification:	Date:
Model:	Galion 503L	
Year:	1986	
Description:	Motorgrader	
Horsepower:	74	
Fuel:	Gasoline	
List Price:	$ 40,140	
Discount:	10%	
Transportation:	N/A	
State Sales Tax:	6%	
Equipment Life:	60 month (5 years)	
Work Hours:	145.83 hours/month (1750 hours/year)	
Salvage Value:	25% of Depreciable Amount	
Depreciation:	Straight Line Method	
Finance Method:	Amortization	
Finance Period:	Equipment Life	
Cost of Capital:	10 %	
Life Maintenance:	70% of Depreciable Amount	
Operator Wages:	N/A	
Fuel Price:	$ 1.05/gallon	
Operating Factor:	85% of Full Load	
Time Factor:	50 min/hour = 0.833	
Fuel Consumption:	0.06 gal/hp-hr	
Lubrication Cost:	10% of Fuel Cost	
Insurance:	3% per year	
Storage:	1% per year	
Escalation:	N/A	
Equipment Overhead:	5% of Total Direct Costs	
Profit:	N/A	
Adjustment Factors:	10% more for Severe Conditions 15% less for Monthly Rental	

TABLE 5 RENTAL RATE CALCULATIONS

Agency: Model: Galion 503L Year: 1986

I. OWNERSHIP COST

Net Price = $ 40,140(1-0.10) = $ 36,126

Sales Tax = $ 36,126 x 0.06 = $ 2,167.56

Depreciable Amount = $ 38,293.56

Salvage Value = $ 38,293.56 x 0.25 = $ 9,573.39

Total Interest Rate = 10% cost of capital
3% insurance
1% storage
14% per year
equivalent to 1.1% per month

$$C_{own} = \frac{\$\ 38{,}293.56 \left[\frac{0.011(1+0.011)^{60}}{(1+0.011)^{60}-1}\right] - \frac{\$\ 9{,}573.39}{60}}{145.83}$$

$C_{own} = \$\ 4.91/hr$

II. OPERATION COST (C_{oper})

A) Maintenance and Repairs (C_{rep})

$$C_{rep} = \frac{38{,}293.56 \times 0.70}{60 \text{ months} \times 145.83 \text{ hrs/month}} = \$\ 3.06/\text{hour}$$

B) Fuel (C_{fuel})

C_{fuel} = 0.85 x 0.833 x 74hp x 0.06 gal/hp•hr x $ 1.05/gal = $ 3.30/hour

C) Lubrication (C_{oil})

C_{oil} = 3.30 x 0.10 = $ 0.33/hour

D) C_{oper} = 3.06 + 3.30 + 0.33 = $ 6.69/hour

III. TOTAL DIRECT COST (TDC)

TDC = C_{own}+ C_{oper} = $ 4.91 + $ 6.69 = $ 11.60/hour

IV. ADJUSTMENTS

A) $C_{overhead}$ = 11.60 x 0.05 = $ 0.58/hour

Total Rental Hourly Rate = 11.60 + 058 = $ 12.18/hour

B) Severe Conditions

$ 12.18 x 1.10 = $ 13.39/hour

C) Idle Time

$ 12.18 - $ 6.69 = $ 5.49/hour

D) Long Term Rental

$ 12.18 x (1-0.15) = $ 10.35/hour

the rental rate of the equipment, overhead and profit (if allowable) should be considered. Certain job conditions and agency policies may also warrant adjustments on the calculated rental rate.

- To cover the project overhead directly related to the equipment, a 5 to 15% increase in the hourly rate is considered reasonable. Profit in force account work is usually calculated as a function of the total job cost and, as such, it should not be included in the calculation of rental rates, unless otherwise stipulated by policy of the agency.
- The length of rental should affect the rate paid for construction equipment. Rental rate guides such as the *Blue Book* by Dataquest, Inc., have an hourly rate that is up to 50 percent more than the hourly rate calculated from their monthly rates for the same machinery. The decrease in rates for extended use is because of the continuous productivity derived from such use.
- If the government organization decides to classify work conditions as normal or severe, then a rate increase of 10 to 20 percent is recommended for use of equipment in severe conditions.
- Idle time that is not the fault of the contractor (such as heavy rain) should be compensated at the regular rental rate minus the operating costs. Downtime should not be compensated.

RENTAL RATES GUIDE: SAMPLE CALCULATIONS

The calculations of rental rates for any type of equipment must comply with the policies adopted by the user agency. Tables 4 and 5 show the data and calculations necessary for the determination of a rental rate for a sample piece of equipment. It must be noted that the assumptions and formulas used were chosen arbitrarily and in reality would be dictated by the policies adopted by the particular government agency. Equipment rental rates shown in Table 6 were calculated in the same manner.

SUMMARY AND CONCLUSIONS

When outside sources of rental rates are used to compensate force account operations, the user has little or no influence over how the information is gathered and what cost parameters are used and, therefore, cannot control how the rates are determined. The companies that publish these guides have their own policies and objectives that may or may not comply with federal and/or state guidelines on rental construction equipment cost accountig and reimbursement.

Most states that use percent adjustments of the *Blue Book* have had difficulty correlating their chosen adjustment to factual data. The disadvantages of using arbitrarily adjusted rental rates is that the adjustment is applied across the board to all types of equipment and the result may be arbitrarily inadequate compensation. Given the success of states such as Colorado and Illinois in implementing their own methods for determining equipment rental rates, and given the variable and unpredictable nature of privately published rental rates guides, it is recommended that each highway agency establish its own specific policies and standard guidelines to deal with construction equipment reimbursement in force account work, in a manner that is fair to contractors.

Once criteria for equipment rental rates have been estab-

TABLE 6 SAMPLE PAGE OF FUTURE RENTAL RATES GUIDE

CONSTRUCTION EQUIPMENT RENTAL RATES

Equipment	HP	List Price ($)	Economic Life (hrs)	Ownership Cost ($/hr)	Operation & Maintenance Cost ($/hr)	Miscellaneous Cost ($/hr)	Hourly Rental Rate: Average Conditions ($/hr)[a]	Hourly Rental Rate: Severe Conditions ($/hr)[a]
MOTORGRADERS								
Gasoline Powered								
BASIC 601A (1978)	23	10,500	8,750	1.28	1.93	0.16	3.40	3.70
GALION 503L (1986)	74	40,410	8,750	4.91	6.69	0.58	12.20	13.40
HUBER M-800 (1976	65	23,495	8,750	2.87	4.98	0.39	8.25	9.05
Diesel Powered								
ALLAT SG-100 (1982)	60	43,191	8,750	5.28	5.26	0.53	11.05	12.20
ATHEY AB-6905(1986)	75	38,095	8,750	4.66	5.36	0.50	10.50	11.60
BASIC 701A (1986)	52	23,137	8,750	2.83	3.47	0.31	6.60	7.30
BLADEMOR 727 (1986)	35	18,930	8,750	2.31	2.58	0.25	5.15	5.65
BLADEMOR 747 (1986)	60	38,100	8,750	4.66	4.87	0.48	10.00	11.00
BOWER ROADRUNNER (1982)	36	18,900	8,750	2.31	2.62	0.25	5.20	5.70

[a] Rounded off to nearest $0.05

Note: For long-term rental discounts, idle time and other conditions, see section on adjustment recommendations.

lished, a software and data base system should be developed and specifically tailored to comply with the guidelines and policy requirements of the government organization. The program thus developed should be flexible enough to be updated and accept necessary changes in cost parameter calculations. The costs of developing, implementing, and updating this system would be readily offset by the savings in equipment rental rates paid by the organization, and, on a smaller scale, by the revenues obtained by selling the rental rates guide to contractors and other interested organizations.

There may be instances in which various states have similar policies regarding force account work. In these cases, the best approach would be to determine policies through a joint effort and to develop a rental rates guide to be used in that region.

The rental rate system thus developed could become part of the Standard Specifications of the government agencies and could be used in conjunction with, or could even replace, negotiation.

A final analysis of the research data indicates that rental rates for construction equipment depend on the cost policies applied to the calculation variables. The recommendations and model of calculations presented can be modified by government organizations to more closely resemble actual costs in their areas and, depending on the policies adopted, could result in sizable savings in the cost construction equipment used in force account work.

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Estimating Production of Multiloader-Truck Fleets

SAEED KARSHENAS

A fast and accurate method for forecasting the production that will be achieved by a selected fleet of equipment is fundamental to the successful planning of projects involving large amounts of earthmoving. The theory of queues has been used to determine a reliable forecast of loader-truck fleet production. Application of the queuing method is time-consuming, especially when forecasting multiloader-truck fleet production. This paper presents graphical solutions of the queuing model for multiloader-truck fleets.

Successful planning of projects involving a large amount of earthmoving requires the ability to reliably forecast the production rates that will be achieved by a selected fleet of equipment and optimizing the combination of earthmovers and loaders in the interest of cost reduction. The theory of queues has been used to determine a reliable forecast of loader-truck fleet production (*1*, *2*). Actual observations and cost determinations made on operating projects have verified the accuracy of this theory (*3*). In this study, the theory of queues is applied to the multiloader-truck fleet production problem and nomographs are developed for rapid estimation of loader-truck fleet production.

MULTILOADER-TRUCK OPERATION

Figure 1 represents a multiloader-truck operation. This type of operation involves a number of loading facilities that serve somc hauling units. The typical cycle of a hauling unit consists of loading, traveling to the dump site, dumping, returning to the loading area, and waiting until a loading unit is available. The parameters and assumptions used in formulation of production are listed below.

Mean Arrival Rate

The arrival rate is the number of hauling units arriving at the queue per unit of time. Because arrivals do not occur at regular intervals, a Poisson distribution is assumed to represent this stochastic behavior of arrivals. A constant parameter (λ) is defined as the mean arrival rate of any particular hauling unit. $\lambda = 1/T_a$, where T_a is the mean travel time. In terms of cycle elements previously defined, T_a is the time of all cycle elements, excluding that for loading and waiting in queue.

Mean Service Rate Per Busy Loader

Service or loading times are expressed by an exponential distribution. The constant parameter (μ) is defined as the mean service rate per busy loader. $\mu = 1/T_S$, where T_S is the mean service time of each server. Therefore, the assumptions are as follows:

1. Arrivals of hauling units to the queue are described by Poisson distribution;
2. Loading times of a unit are described by an exponential distribution;
3. Hauling units are served on a first-come, first-served basis; and
4. The system is in a steady-state condition.

PRODUCTION FORECASTING MODEL

For the case of N_l loading units and N_t hauling units, the production forecasting can be formulated as follows:

$$Q = \frac{T \cdot f \cdot q_l}{t_c}$$

where

Q = average quantity of earth moved per unit time,
f = job efficiency,
q_l = rated loader bucket capacity,
t_c = average loader cycle time, and
T = production factor.

If P_i is the probability of i hauling units in the system, T (*3*) would be

$$T = \sum_{n=1}^{N_l} n \cdot P_n + \sum_{n=N_l+1}^{N_t} N_l \cdot P_n$$

P_n may be calculated as follows (*4*):

$$P_n = \begin{cases} \binom{N_t}{n} r^n \cdot P_0 & 0 \le n \le N_l \\ \binom{N_t}{n} \dfrac{n! \cdot r^n}{N_l! \cdot N_l^{n-N_l}} \cdot P_0 & N_l \le n \le N_t \end{cases}$$

where r is $\frac{\lambda}{\mu}$, and P_0 is the probability of an empty system,

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which is calculated as follows (*4*):

$$P_0 = \left\{ \sum_{n=0}^{N_l} \binom{N_l}{n} r^n + \sum_{n=N_l+1}^{N_l} \binom{N_l}{n} \frac{n! \cdot r^n}{N_l! \cdot N_l^{n-N_l}} \right\}^{-1}$$

By putting $N_l = 1$ in the above equations, the loader-truck production equations developed by O'Shea et al. (*2*) can be obtained.

To simplify application of the above formulas to practical problems, graphical solutions for various numbers of loaders may be developed. Figures 2 and 3 show nomographs (*5*) based on the above formulation for one- and two-loader fleets, respectively.

To use the nomographs for estimating production in a project, r must be calculated based on the equipment and haul road characteristics (for examples, see references *6* and *7*). After calculating r, enter the chart on the horizontal scale with the calculated r and move vertically up until intersecting the curve representing the number of trucks used. From the point of intersection, move horizontally to the right vertical T-scale. Connect the point of intersection on the T-scale to loader bucket capacity point of q_l-scale and extend the line to intersect D-scale. The intersection of this line and D-scale would then be joined with the loader cycle time on t_c-scale. The intersection of the latter line with the Q-scale would give the production per hour. The unit of production depends on the unit used for loader bucket capacity. The following examples illustrate the method of using the nomographs.

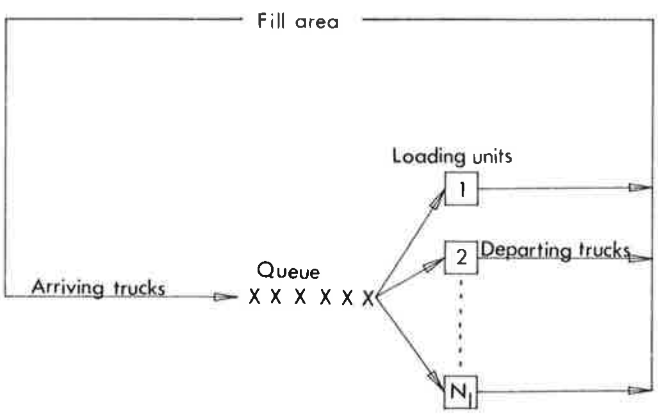

FIGURE 1 A multiserver queueing system.

1. Determine the production per hour of a fleet consisting of a 5-yd^3 (3.8-m^3) loader and ten 30-yd^3 (22.8-m^3) trucks. The average loader cycle time is 0.5 minute, and the average truck travel time is 20 minutes. Assume that the loader bucket fill factor is equal to 1 and job efficiency is 100 percent.

Solution: The ratio of truck arrival rate to loading rate for this project is 0.15. From Figure 2, production per hour for this fleet is about 560 yd^3 (425.5 m^3). The dashed line in Figure 2 shows the solution.

2. What would be the production per hour in example 1 if two loaders and 14 trucks were used?

Solution: From Figure 3, the production per hour is approximately 980 yd^3 (745 m^3). The solution is shown with a dashed line.

SUMMARY AND CONCLUSION

A fast and reliable method for forecasting production of a specified loader-truck fleet is desirable. The theory of queues

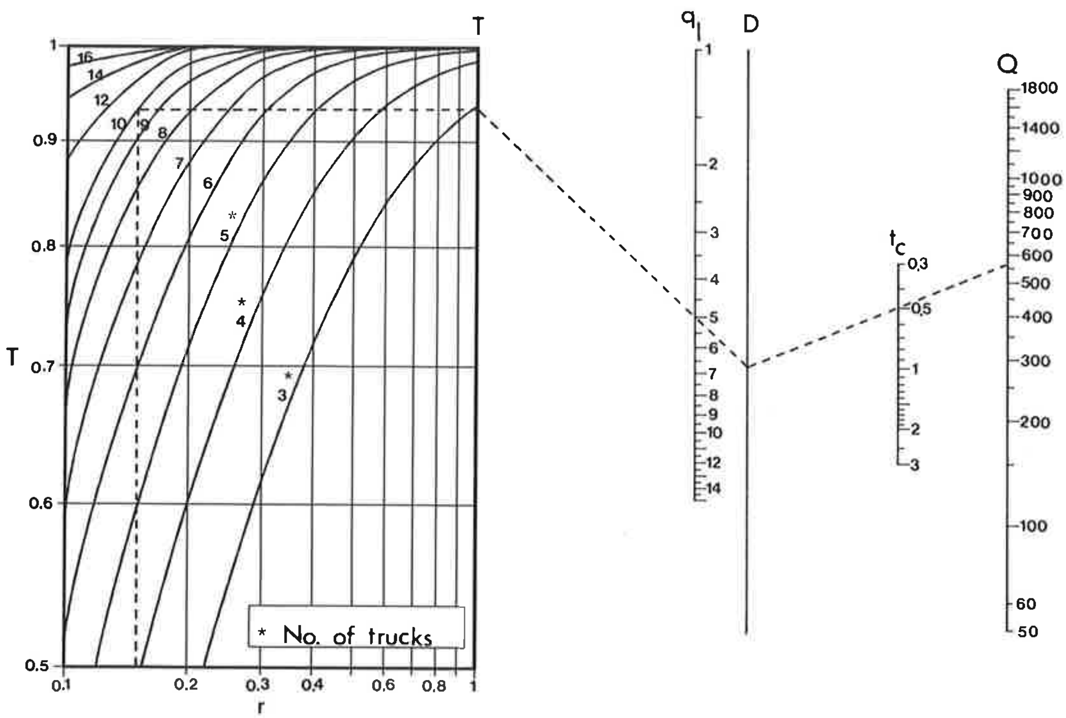

FIGURE 2 Nomograph for a 1-loader fleet.

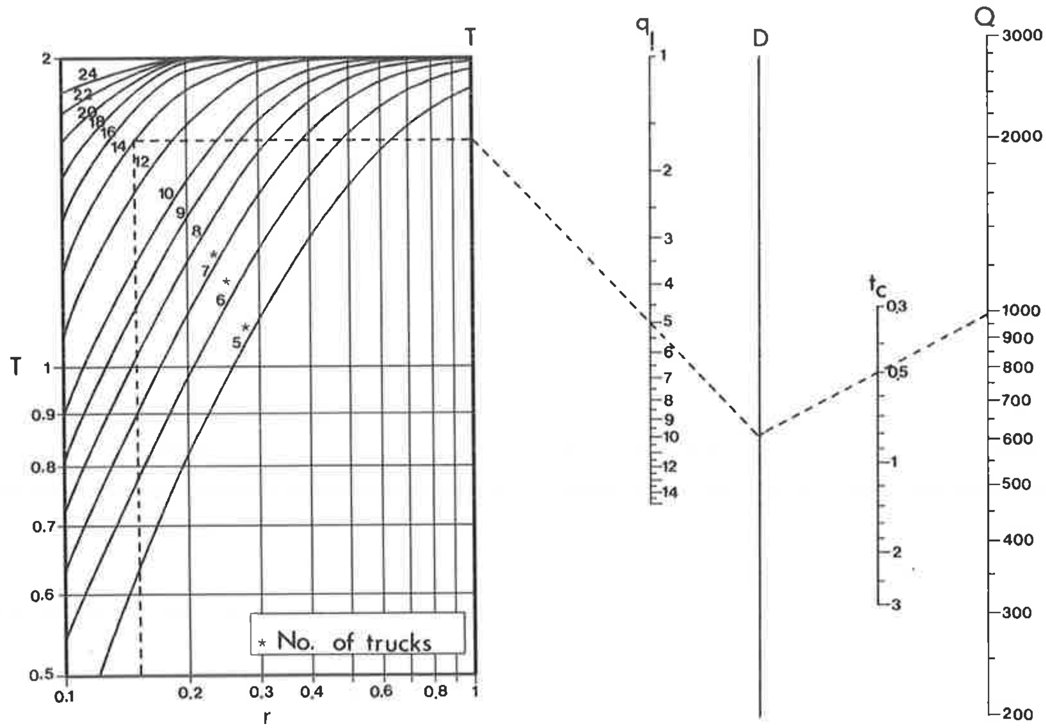

FIGURE 3 Nomograph for a 2-loader fleet.

has been used to determine a reliable estimate of loader-truck production. However, the application of methods based on queuing theory is time-consuming, especially for multiloader-truck fleet production. In this study, the general queueing formulation for loader-truck production forecasting was presented, and graphic solutions of the mathematical model were developed for a fast estimate of production for various fleets.

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Effects of Cyclic Wetting-Drying Weathering on Wear Resistance of Concrete Pavement

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This paper describes the development of a laboratory test procedure to assess the weathering effects of a wet tropical climate on wear resistance of concrete pavements. An accelerated weathering model was fabricated to provide cyclic wetting and drying treatment in a laboratory simulation of the weathering effects of Singapore's climate. The effects of weathering on wear resistance were evaluated by means of a rotating drum test conducted on plain cement mortar specimens with and without weathering treatment. The study showed that cyclic wetting and drying weathering significantly reduced the wear resistance of cement mortar test specimens. Although no correlation relationship is available at this stage between test data of the proposed procedure and field performance of in-service pavements, results obtained in this study suggest that the test procedure could find useful application in evaluating the relative surface wear resistance and durability of various concrete pavement materials.

The provision of a durable traveled surface is an important consideration in highway pavement design. Deterioration of pavement surfaces, such as raveling, spalling, and surface wear, although usually causing negligible loss in structural capacity of pavements, is a major concern to highway engineers. Such surface defects can seriously affect riding quality and may considerably reduce the useful service life of the pavement affected.

The task of providing a durable pavement surface is not an easy one. Highway pavements are exposed to environmental weathering throughout their service periods and are constantly subjected to repetitive wearing actions caused by moving vehicular traffic. Once a surface distress is initiated, by either traffic or environmental factors, both will act together to contribute to the continuing and, in many cases, accelerating deterioration of the pavement surface. A complete study of pavement surface durability would therefore require analysis of the impacts of both traffic loadings and environmental weathering.

On concrete pavements, according to Burwell (*1*), the following two types of wear mechanism are prominent. One is known as surface fatigue wear, which is caused by vertical dynamic wheel loads of vehicles. The other is abrasive wear, which causes damage through rubbing actions between tires and pavement surface and through scratching and gouging actions caused by particles caught between moving wheels and the road surface. This suggests that a meaningful surface wear evaluation of concrete pavement materials must include the effects of both wear mechanisms.

The study of damaging effects of environmental weathering has been a topic of research for many decades (*2*, *3*). This problem is complicated because outdoor exposure tests are time-consuming, and it is difficult to control and standardize test conditions. As a result, laboratory-accelerated tests are often employed to predict the in-service durability of various types of materials. A number of testing devices known as weatherometers are available in the commercial market (*4*). They are typically equipped for test cycles requiring alternate light-dark and wet-dry exposures with temperature and relative humidity controls. Regardless of the type of equipment used, a laboratory-accelerated test must be designed to simulate as closely as possible the in-service environment in which the test material is intended to be used, so as to produce results that correlate well with material performance under outdoor service conditions.

This paper describes a laboratory test procedure for evaluating the relative durability of concrete pavement materials against surface wear. A laboratory-accelerated weathering model was developed to simulate the wetting and drying cycles of a wet tropical climate. The effects of wetting and drying weathering on material surface wear resistance were then determined by means of a rotating drum wear test conducted on specimens with and without weathering treatment.

BACKGROUND

Concrete pavements have been extensively used for bus bay, bus lane, and bus terminal construction in Singapore since the late 1970s. The switch from bituminous to concrete pavements was made after more than 20 years of unsatisfactory performance of bituminous pavements on roads dominated by bus traffic (*5*). The local experience with concrete pavements so far indicates that, although these pavements in general perform satisfactorily structurally, surface deterioration is a common distress problem that could drastically reduce the useful service life of those pavements affected.

The surface deterioration is characterized by loss of cement mortar from the road surface along wheel paths, thereby exposing coarse aggregates and giving the surface a rough and shabby appearance. The ride on such deteriorated surfaces is rough, noisy, and uncomfortable. Field observations indicated that the deterioration of concrete pavements involved initial wear of the hardened cement mortar at the surface. As the cement mortar was worn away, coarse aggregates were ultimately exposed. Depending on the bond between the cement mortar and coarse aggregates, the progressive wear could lead to raveling.

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The present study is confined only to the first phase of the deterioration process, during which the wear resistance of cement mortar governs. This is important because a highway pavement that suffers deterioration of surface cement mortar is undesirable because it would not provide an acceptably smooth ride.

TEST METHODOLOGY

The aim of the test program in this study was to develop a test procedure that could be used by local pavement designers and maintenance engineers in their evaluation of material quality and durability for new construction or resurfacing of concrete pavements. The laboratory test was conducted on 100-mm (3.937-in.) cubical specimens. The procedure developed was composed of two parts. First, specimens were treated with laboratory-simulated wetting and drying cycles. Next, surface wear resistance tests were conducted on treated and untreated specimens to evaluate their relative wear resistance. Basis and details of the two parts are described below.

Weathering Simulation

The wet tropical climate of Singapore, as depicted in Figure 1, is characterized by an abundance of rainfall as well as bright sunshine and a relatively uniform temperature accompanied by high humidity throughout the year. The region generally has rainfall throughout the year but tends to be particularly wet during the monsoon season from November to January. The average annual rainfall is around 2,200 mm (86 in.), with precipitation of more than 50 mm a day occurring about nine times a year (*6*). The pavements in the region are also exposed to sunshine extensively each year. The annual average number of hours each day under bright sunshine is more than 5 hours. The abundance of both rainfall and sunlight results in a relatively large number of wetting and drying cycles on the pavement surface. Precipitation falling on sun-heated pavements and intense sunlight drying up rain-soaked road surfaces are common scenes in Singapore.

On a tropical day, the maximum air temperature is around 31°C and the minimum 24°C. The surface temperature variation on most road pavements is much larger. Concrete pavement surface temperatures, measured by using thermocouple wires, usually range from around 25°C in the early morning to as high as 60°C on wheel paths on a hot afternoon. Thus, a wetting or drying process with a temperature change of 30 to 35°C is not unusual.

A laboratory model designed to provide alternating wetting and drying was used to simulate weathering caused by the frequent alternating rainfall and sunshine exposure of in-service pavements. The accelerated weathering model is essentially a concrete tank with an enclosed space that measured 915 mm (36 in.) in height and 940 mm by 1,420 mm (37 in. by 56 in.) in plane cross-section. Wetting of test specimens was achieved by spraying tap water, at about 28°C, through eight shower heads that were fitted on the interior walls of the weathering tank. The number of shower heads was more than sufficient to keep test specimens wet throughout the wetting phase. Heating during the drying phase was provided by four 500-W ceramic heaters located at the underside of the ceiling of the weathering tank. A surface temperature of 60°C was achieved on 100-mm concrete cube specimens placed on the floor of the tank, after about 100 minutes of exposure to heating in the tank.

This study adopted a 4-hour weathering treatment cycle that consisted of 2 hours of wetting followed by 2 hours of drying. The length of the wetting phase was selected to represent approximately the mean duration of rainfall in Singapore (*6*). The 2-hour drying period was chosen so as to attain the desired maximum temperature of about 60°C during the drying period. Due to high humidity in the weathering tank, the 2-hour drying treatment was, in terms of moisture

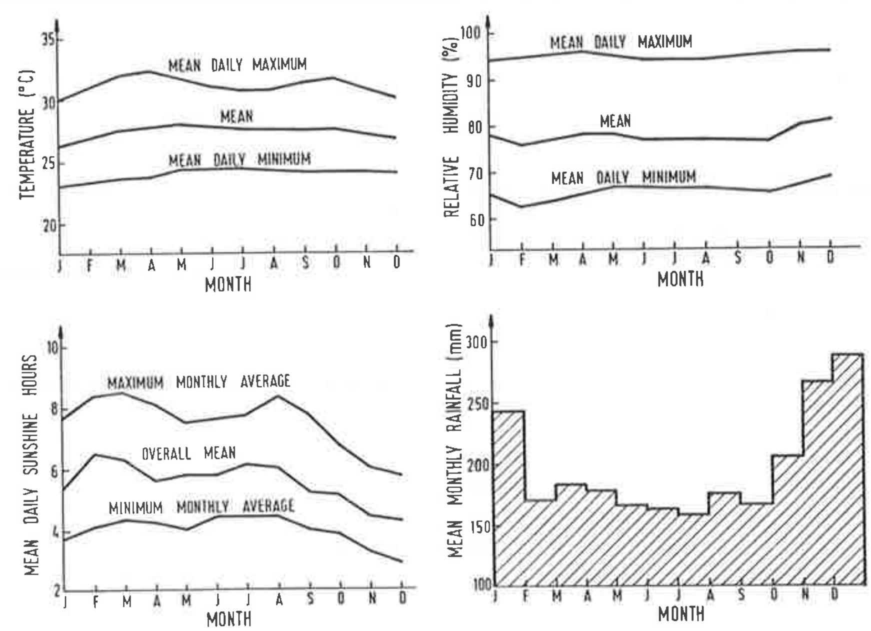

FIGURE 1 Characteristics of Singapore climate based on data from 1967–1986.

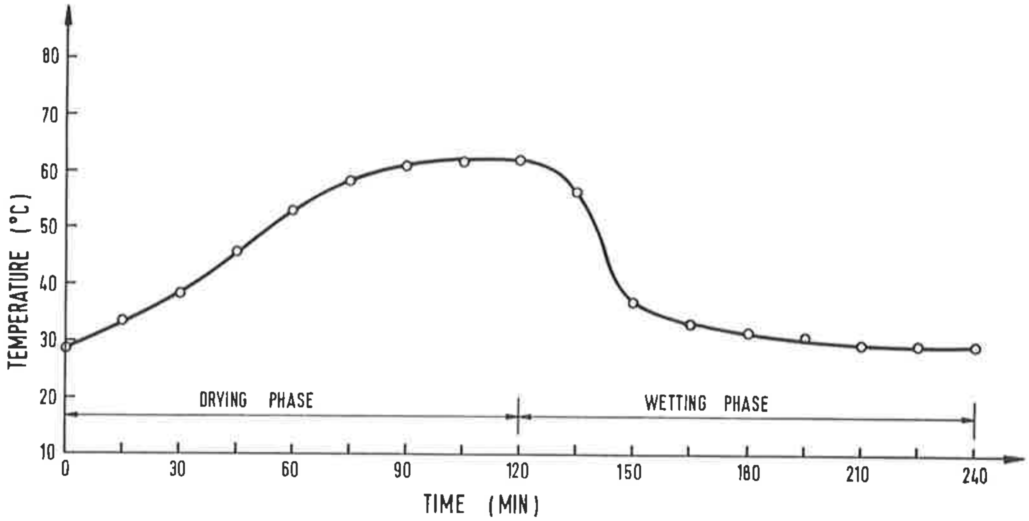

FIGURE 2 Surface temperature variations of cement mortar specimens in weathering model.

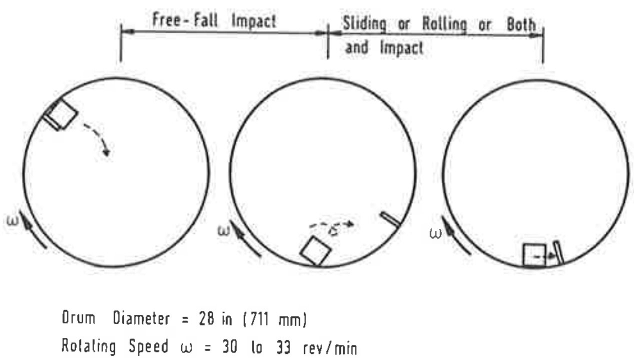

FIGURE 3 Schematic representation of wearing actions on a cubical specimen in a rotating drum test (7).

loss, equivalent to approximately 1 hour of outdoor exposure to bright sunlight at an air temperature of 31°C and a relative humidity of about 75 percent.

The variations of surface temperature of 100-mm (3.937-in.) cubical cement mortar specimens during the wetting and drying cycles are shown in Figure 2. In the drying phase, the specimen surface temperature built up gradually and reached a maximum of about 62°C at the end of the 2-hour period. During the wetting phase, the temperature decreased slightly in the first 15 minutes and then fell fairly quickly to the temperature of water in the next 30 minutes.

A 150-cycle weathering treatment was used in the test program for the purpose of determining if cyclic wetting and drying had any adverse effect on the surface wear resistance of concrete pavement materials. In selection of the number of wetting and drying cycles, the average number of rainy days in a year was used as a guide. An examination of the meteorological data in the last 5 years in Singapore (6) showed that there were, on the average, approximately 150 days with rain each year. Although it may be true to say that a pavement in the field experienced about 150 wetting and drying cycles in a year, it is unlikely that the 150 cycles in the field would be as severe as 150 cycles in the weathering model described above. This difference is because not all rains would fall during a hot afternoon after pavements had been heated up by sunlight, and not all rains would be followed by intense heating from hot afternoon sun. Compared with the weathering model cycles, most field cycles are likely to have longer drying periods and a more gradual rate of temperature changes.

The accelerated application of wetting and drying cycles in the laboratory was designed to intensify the weathering process. The 150 cycles in the laboratory weathering model would probably achieve a weathering effect equivalent to that produced in the field over a period of 2 years or more. Since the surface deterioration distress described in this paper usually appeared 2 to 3 years after a pavement was open to traffic, the use of 150 laboratory weathering cycles appears to be a reasonable choice for the purpose of this study.

Surface Wear Resistance Test

A rotating drum test developed by Fwa and Paramasivam (7) for evaluating impact and abrasion resistance of concrete pavement materials was adopted. The test makes use of the Los Angeles machine specified in ASTM Test C535-81 (8). Specimens of 100-mm (3.937-in.) cubes were used, but no abrasive charge was employed. Figure 3 shows a schematic diagram indicating the wearing mechanism in each rotation of the test. Three wearing actions, namely impact, sliding, and rolling, were involved. In each test run, two 100-mm cubes were placed in the drum for 2,000 revolutions. Worn-off materials were removed, and weights of the two cubes were taken at intervals of 200 revolutions. Results of the test are reported in terms of wear, which is computed as follows:

$$W = \frac{m_o - m}{m_o} \times 100$$

where

W = wear in percent by mass,
m_o = initial total mass of specimens tested, and
m = remaining mass of specimens after wearing treatment.

Studies conducted by Fwa and Paramasivam (7) found the test to have satisfactory repeatability and good sensitivity against variations in the properties of concrete pavement materials tested. Their results indicated that, by conducting three tests for each material type tested, a confidence level of 95 percent can be achieved so that the difference, between sample mean and population mean surface wear, would not exceed an allowable error of 5 percent.

In addition, they observed that the test produced worn specimen surface with good resemblance to a field-deteriorated surface similar to the distress described in this paper. They also found that surface wear of cement mortar test specimens was similar to the initial phase of surface wear of concrete test specimens; they concluded that cement mortar specimens could be effectively used to study the crucial initial phase of surface deterioration, which is of major concern in highway pavement engineering.

Test Specimen Types

Based on the reasonings of earlier discussions, cement mortar specimens were used in the tests. The cement mortar tested corresponds to that of a concrete with the following mix proportions: cement/sand/coarse aggregate ratio = 1:1.5:2.5 by weight. Test specimens were prepared with the following five water/cement ratios: 0.45, 0.50, 0.55, 0.60, and 0.65.

In all the mixes prepared, an accelerator was added at 1 L per 22 kg (0.27 gal per 50 lb) of cement to give the desired strength at the age of 7 days. The 7-day, 100-mm (3.937-in.) cube strength values of water-cured plain cement mortars, in order of increasing water/cement ratio, were 49.7 N/mm^2 (7,210 psi), 46.0 N/mm^2 (6,670 psi), 41.8 N/mm^2 (6,060 psi), 31.6 N/mm^2 (4,580 psi), and 30.1 N/mm^2 (4,365 psi).

Test Specimen Exposure Conditions

To evaluate the significance of wetting and drying weathering effects on test materials, three exposure conditions were specified for each material type. Specimens from a batch of mix were divided into three groups, each subjected to a different exposure condition, followed by a rotating drum surface wear test. The three exposure conditions are described below:

1. Initial condition. Cubical test specimens of 100 mm were cast, then demolded after 24 hours, and cured in a water tank for 5 days. The specimens were removed on the 6th day and left in room conditions at approximately 28°C for 1 day. Surface wear test and compressive cube strength test were conducted on the 7th day after casting. The relative humidity varied from about 70 percent in the afternoon to about 95 percent at night.
2. Weathering treatment condition. Test specimens were cast and water-cured as in step 1 above. The specimens were air-dried for 1 day in room conditions at 28°C before they were subjected to alternating wetting and drying in the laboratory weathering tank for 150 cycles. Each weathering cycle consisted of 2 hours of wetting followed by 2 hours of drying. The specimens went through 6 weathering cycles per day, and it took 25 days to complete the 150 cycles. Specimens were again air-dried for 1 day in room conditions after the weathering treatment and before rotating drum surface wear tests were carried out on the 32nd day after casting.
3. Control condition. Test specimens were cast and water-cured as in step 1 above. After removal from a water tank, the specimens were left in room conditions at 28°C for the same duration as the weathering treatment described in step 2 above. Rotating drum surface wear tests were conducted on the 32nd day after casting.

The control condition provides a basis of comparison on which the effect of weathering can be determined from test results of specimens subjected to weathering treatment. Because of the length of treatment duration, some gain in strength took place during this period. The initial condition was therefore included to offer a more thorough examination of changes in surface wear resistance of test specimens.

ANALYSIS OF TEST RESULTS

The surface wear test data are recorded in Table 1. As an illustration, Figures 4 and 5 show examples of typical progressive wear plots obtained from the surface wear tests. Each curve in the figures represents the average surface wear obtained from three rotating drum tests. Figure 6 summarizes the final results of the test program. Discussed below are findings derived from analyses of the test results.

Initial Versus Control Conditions

Specimens that underwent the control conditions were tested at an age 25 days older than those tested after the initial water-curing. Table 2 records the strength comparison of specimens tested under these two conditions. All five mixes studied had about a 10 percent increase each in their compressive strength.

The surface wear test results in Table 1 show corresponding increases in surface wear resistance of all materials from initial to control condition. The increase in wear resistance of each mix type was found to be statistically significant at a confidence level of 99 percent. As can be computed from the data in Table 1, each material type had approximately 20 percent less in surface wear. This positive correlation of surface wear resistance with strength is in agreement with the findings of other researchers (*9*, *10*) who studied the resistance of concrete against various forms of abrasion.

Control Conditions Versus Weathering Treatment

Since specimens that were exposed to control conditions and those exposed to weathering treatment were cast from the same batch of mix and tested at the same age for surface wear, a comparison of their wear test results offers an indication of the significance of the weathering effect on surface wear resistance of test materials.

An examination of the plots in Figures 4–6 reveals that, compared to specimens in control conditions, specimens exposed to laboratory cyclic wetting and drying treatment suffered increased surface wear. Individual statistical tests conducted separately for each of the five mixes concluded that, at 99 percent confidence level, the effect of weathering

TABLE 1 SUMMARY OF SURFACE WEAR TEST RESULTS

Treatment to Specimen	Water/Cement Ratio				
	0.45	0.50	0.55	0.60	0.65
Initial Condition	56.4 54.1 52.1 (54.2%)	60.9 58.2 64.3 (61.1%)	64.9 64.0 64.4 (64.4%)	71.0 67.8 69.9 (69.6%)	76.2 78.1 78.8 (77.7%)
Control Condition	44.3 47.6 46.4 (46.1%)	48.5 47.9 44.3 (46.9%)	53.2 51.3 51.6 (52.0%)	58.4 54.0 54.4 (55.6%)	62.1 61.9 57.4 (60.5%)
Weathering Condition	54.6 58.2 53.8 (55.5%)	58.8 63.4 57.6 (59.9%)	63.9 66.5 66.6 (65.7%)	70.9 73.3 72.8 (72.3%)	73.6 76.0 74.8 (74.8%)

Notes: 1. Values in table represent percentage wear by mass after 2000 revolutions in rotating drum test.
2. Each value in parentheses represents average result of 3 rotating drum tests.

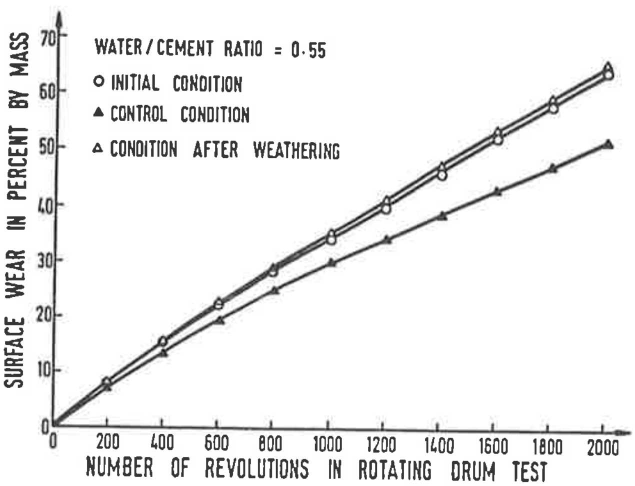

FIGURE 4 Results of surface wear test on cement mortar specimens with water-cement ratio of 0.55.

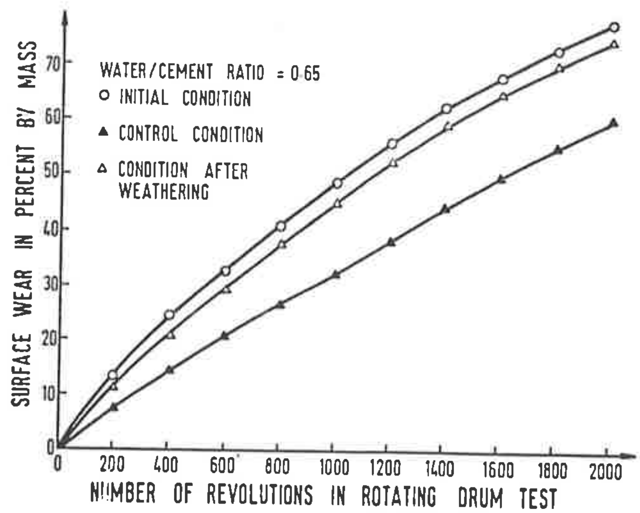

FIGURE 5 Results of surface wear test on cement mortar specimens with water-cement ratio of 0.65.

treatment was significant in reducing the surface wear resistance of test materials.

Initial Conditions Versus Weathering Treatment

It is of practical interest to compare the wear resistance of a concrete pavement at the initial stage and at a later phase of its service life after exposure to weathering. On one hand, there will be improvement in wear resistance because of an increase in concrete strength with age; on the other hand, environmental weathering causes deterioration of concrete and reduces its wear resistance. For materials to be used for highway pavement construction, it is highly desirable that its gain in wear resistance with age should at least be sufficient to offset any loss caused by weathering.

For the mixes tested in this study, Figure 6 shows that the gains in wear resistance of test specimens brought about by the growth of mortar strength were more or less offset by the reductions of wear resistance caused by weathering. Although these findings cannot be indiscriminately extended to cover other concrete or cement mortar mixes before similar tests are conducted, it appears logical to state that designers must not simply assume in their designs any increase in wear resistance with age for concrete pavements exposed to weathering.

Effects of Water/Cement Ratio

Past studies (*7*, *9*, *10*) have found that the abrasion resistance of concrete or cement mortar of a given mix can be improved by reducing its water/cement ratio. In the present study, for

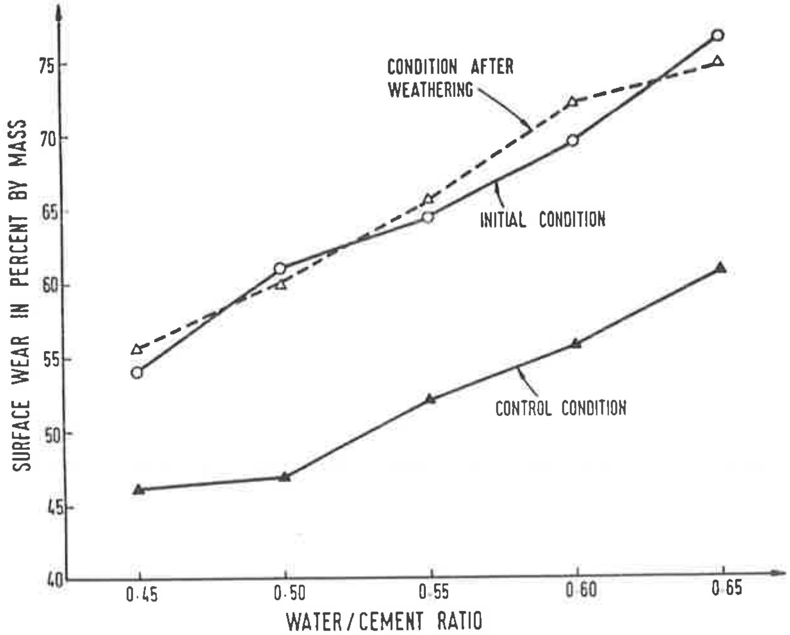

FIGURE 6 Surface wear comparison of test specimens subjected to various exposure conditions.

TABLE 2 COMPRESSIVE STRENGTH OF 100-MM CUBICAL SPECIMENS

Water/Cement Ratio	Cube Strength		
	Initial Condition	Control Condition	Weathering Condition
0.45	49.7 N/mm^2 (7,210 psi)	54.2 N/mm^2 (7,860 psi)	52.1 N/mm^2 (7,555 psi)
0.50	46.0 N/mm^2 (6,670 psi)	49.4 N/mm^2 (7,165 psi)	51.0 N/mm^2 (7,395 psi)
0.55	39.3 N/mm^2 (5,700 psi)	44.7 N/mm^2 (6,480 psi)	42.1 N/mm^2 (6,105 psi)
0.60	31.6 N/mm^2 (4,580 psi)	36.2 N/mm^2 (5,250 psi)	34.3 N/mm^2 (4,975 psi)
0.65	30.1 N/mm^2 (4,365 psi)	35.4 N/mm^2 (5,135 psi)	33.8 N/mm^2 (4,900 psi)

those specimens that were water-cured as in the "initial condition" case, and those that were water- and subsequently air-cured as in the "control condition" case, one would expect the relationship between surface wear and water/cement ratio to hold. It would be interesting to find out if the same is also true for test specimens subjected to weathering.

Referring again to Figure 6, we see that although the initial and control test data demonstrated the trend anticipated, surface wear data of the "condition-after-weathering" case also produced better wear resistance when the water/cement ratio of the mix was reduced. The shapes of the curves of the wear vs. the water/cement ratios for the three test conditions are similar. Based on these test data, it appears appropriate to say that lowering the water/cement ratio can be an effective means of improving the wear resistance of concrete pavements exposed to climatic weathering.

Relationship Between Wear Resistance and Compressive Strength

For a given mix of concrete material, it is convenient to think that its surface wear resistance would improve as its compressive strength increases. Fwa and Paramasivam (7) have shown, however, that such a relationship is likely to be nonlinear in nature, and it varies among various mixes of cement mortar. One must therefore refrain from using concrete or mortar compressive strength as a relative indicator of the wear resistance of various concrete materials. The test results in this study provide further confirmation of this view.

In Figure 7 the surface wear of test specimens exposed to various conditions are plotted against their compressive strength. Looking at the test results of each of the three treatment conditions independently, one can conclude that among specimens there existed a trend of decreasing surface wear with increasing compressive strength. The same however cannot be said when one combines the test data of specimens treated under different exposure conditions. At any compressive strength level, it can be seen that surface wear values could differ by as much as 15 percentage points. There was clearly no unique relationship between compressive strength and surface wear in this case.

It is interesting to note that the compressive cube strength of specimens exposed to weathering and that of specimens under control conditions were similar. There was, however, a large difference in their respective surface wear resistance. It is possible that although the weathering treatment had caused deterioration of the surface layer of test specimens, as reflected by the increased surface wear in these specimens, it was not intensive enough to have adverse effects on the interior of the specimens and their overall compressive strength.

CONCLUSIONS

Based on the findings of the test program, the following conclusions may be drawn:

1. As the strength of concrete increased with age, the limited test results in this study suggest that there was a corresponding gain in wear resistance as cement mortar matured.
2. The laboratory-accelerated wetting and drying weathering treatment caused statistically significant reductions in surface wear resistance of all cement mortar mixes tested. The test was found useful in providing an indication of the detrimental effects that adverse weather conditions have on the wear resistance of concrete pavements.
3. For a highway concrete pavement that is exposed to weathering in its service life, one must not assume that its wear resistance would improve with time as the concrete ages. The present study showed that the reduction in wear resistance caused by weathering could be higher than the gain brought about by maturation of cement mortar.
4. Test results showed that an improvement in wear resistance of the test specimens under weathering conditions could be effected by lowering the water/cement ratio of the cement mortar mix.
5. The wear resistance of a concrete pavement is essentially a function of its surface properties. It cannot be uniquely related to its compressive strength measured from conventional cube tests.
6. Although no correlation can be made at this stage with field performance of in-service pavements, the test results obtained in this study indicate that the test procedure adopted could be useful in evaluating the relative surface wear resistance and durability of various concrete pavement materials.

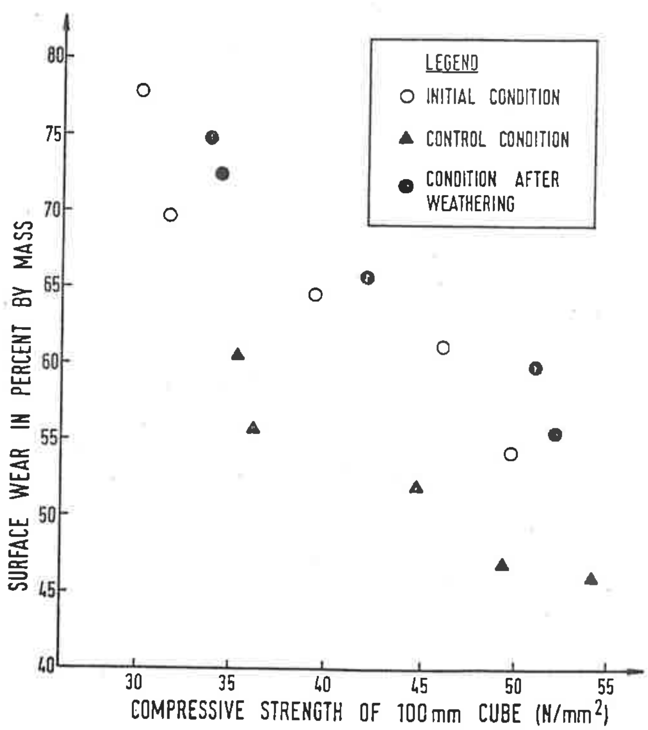

FIGURE 7 Relationship between compressive strength and surface wear resistance of test specimens.

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