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Contents

Foreword

This Record contains information on fast-track concrete, durable longitudinal asphalt joints, asphalt density and compactibility, large-stone asphalt mixes, quality control and quality assurance training, unique bridge construction, construction management, fatigue damage in welded steel structures, and disadvantaged business enterprises programs. The Record should be of interest to state and local materials, construction, maintenance, and pavement and structures design engineers, as well as contractors and material producers.

Grove et al. describe the use of Fast Track and Fast Track II portland cement concrete on a reconstructed street project in which closure of the street had to be held to a minimum. Cortez and Gerlach discuss the design, construction, quality control practice, and research on the durability of roller-compacted concrete pavement constructed in a seasonal frost environment.

Baker et al. developed a technique for producing more durable longitudinal construction joints in bituminous joints. The procedure involves forming the joint between adjoining lanes as two overlapping wedges. E. R. Brown reports on asphalt concrete density—the amount that is needed, the different methods of specifying density, and the methods to measure density. McQueen investigated the degree to which base pavement support influences the compactibility of an asphaltic concrete overlay. As a part of that study, he also compared the FAA Eastern Region in-place air voids compaction standard with the FAA National percent Marshall density compaction standard. Mahboub and Williams discuss the problems associated with the construction of large-stone asphalt mixtures. They provide recommendations for avoiding problems with segregation, poor compaction, low density, and particle crushing.

Hughes and Ahmed describe the development, organization, implementation, and evaluation of the Oklahoma Department of Transportation and Oklahoma State University quality assurance training program. The program was established in response to a need to train department employees concerning recently adopted statistically based quality control and quality assurance specifications.

McCarthy et al. describe unique construction methods on the Robert E. Lee Bridge in Richmond, Virginia. They discuss the use of portable fabric cofferdams for constructing the bridge piers, the concurrent use of eight form travelers, and the use of a strand stability system for the first time in segmental concrete balanced cantilever construction.

Brenner describes some computer applications being developed and applied for the construction planning and traffic studies associated with the design of the Central Artery and Third Harbor Tunnel in Boston. Russell describes a prequalification model for evaluating highway contractors. He includes recommendations for facilitating the model's implementation into existing state DOT prequalification procedures. Beliveau describes the use of CADD positioning technologies for improving the control of construction projects. Kim and Ibbs present an experimental object-oriented data base management system called ODEPSI for managing design and construction project data. Ellis and Herbsman describe an innovative approach for determining the low bidder on highway construction contracts. The method requires each bidder to propose both a time duration for the project and the traditional unit prices for the work items. After analyzing data acquired from 16 case studies, they concluded that the method can be developed for application in both the public and private sectors. Hinze and Carlisle examined the factors involved in the decisions to use nighttime construction schedules. In surveying state transportation agencies, they found that the biggest reason for contractually requiring a nighttime schedule is that unacceptably high traffic congestion would result from daytime construction work. Najafi investigates the extent of computer applications in various aspects of the construction industry.

Fisher and Menzemer review typical types of fatigue damage being found in welded steel bridges; they cite some specific examples, examine common retrofit procedures, and outline proper investigation practices.

The following papers, which were prepared for presentation at recent annual meetings of TRB, concerned current issues related to disadvantaged and women-owned business enterprises (DBEs and WBEs, respectively) operating in the transportation construction industry. Although several of the papers are not based on actual research projects nor are typical of the format of most TRB papers, they have been published because of the timeliness of their topics. The TRB Committee on Disadvantaged Business Enterprises, which is trying to promote the successful integration of DBEs into transportation construction, feels that the papers present viewpoints and information that will be useful for readers involved with DBE programs.

Hancher et al. present a method for predicting the annual work capacity of DBE firms in the Texas highway construction industry. Wormington discusses the perspective and experiences of the Arizona Department of Transportation regarding its DBE program. Gendell et al. discuss the FHWA viewpoint of the DBE Program. Wilson presents the WBE viewpoint in DBE programs. Payne examines the day-to-day problems faced by women who are attempting to gain or maintain their certification in DBE programs. W. R. Brown discusses the Indiana Department of Transportation's procedure for managing projected DBE participation and ensuring the attainment of the state's annual 10 percent DBE goal.

Fast Track and Fast Track II, Cedar Rapids, Iowa

J. D. Grove, K. B. Jones, K. S. Bharil, A. Abdulshafi, and W. Calderwood

Two lanes of a major four-lane arterial street in Cedar Rapids, Iowa, needed reconstruction. Because of the traffic volume and the detour problem, closure of the intersections, even for 1 day, was not feasible. Use of Fast Track concrete paving on the mainline portion of the project permitted achievement of the opening strength of 400 psi in less than 12 hr. Fast Track II, used for the intersections, achieved the opening strength of 350 psi in 6 to 7 hr. Flexural and compression specimens of two sections each in the Fast Track and Fast Track II sections were subjected to pulse velocity tests. Maturity curves were developed by monitoring the temperatures. Correlations were performed between the pulse velocity and flexural strength and between the maturity and flexural strength. The project established the feasibility of using Fast Track II to construct portland cement concrete pavement at night and opening the roadway to traffic the next day.

Experience with the first Fast Track concrete in Buena Vista County on US-71 in 1986 and on several subsequent projects has shown that Fast Track is a viable construction alternative for certain locations. Strengths for opening Fast Track pavement have always been achieved within 24 hr and often within 12 hr. Fast Track concrete is produced from a high cement factor mix incorporating a special Type III Portland cement. The construction requires only conventional equipment and techniques, except for the use of insulating blankets to contain and uniformly distribute the heat in the pavement during the early stages of curing.

Contractors and contracting authorities have begun thinking of portland cement concrete (PCC) pavement as an option for locations with critical traffic control requirements. This was the situation on a reconstruction project in an urban area in Iowa. Neither the city nor the area businesses would support closing intersections for the duration of the paving. A compromise was reached by which the intersections would be closed during the nighttime hours and be open during the day. The reconstruction was scheduled to be PCC pavement, but the intersection would require concrete that could achieve opening strength in 6 to 8 hr.

The conventional Fast Track pavement and a higher cement content concrete (Fast Track II) were subjected to laboratory tests to evaluate their early strength and temperature gain. At a rate of 822 lb of special Type III cement per cubic yard, flexural strengths exceeding 300 psi were achieved at 6 hr. On the basis of this information, the design and specifications were completed for letting.

J. D. Grove, K. B. Jones, and K. S. Bharil, Office of Materials, Highway Division, Iowa Department of Transportation, 800 Lincoln Way, Ames, Iowa 50010. A. Abdulshafi, C.T.L. International, 2860 Fischer Road, Columbus, Ohio 43204. W. Calderwood, Cedar Valley Corporation, P.O. Box 368, Cedar Falls, Iowa 50613.

CONSTRUCTION PROJECT

The project was along a section of Iowa-100 located in the northern part of the city of Cedar Rapids, Iowa. The work involved replacing two westbound lanes of a 1.84-mi, four-lane urban section of road divided by a raised median. The road carries 15,400 veh/day, and 4 percent of these are trucks. The lanes to be replaced were 29 years old and of PCC construction.

The project design called for 10.5-in. plain doweled and jointed PCC pavement using 33,000 yd² of Fast Track mix and 1,700 yd² of Fast Track II mix. Figure 1 shows the project layout. Westbound traffic was to be detoured to a roadway north of the project. Closures of the intersections were restricted. Council Street, Duffy Drive, Park Lane, Rockwell Drive (Station 53+), C Avenue, Northland Avenue, and Twixt Town Road could be closed from 6:00 p.m. to 6:00 a.m. any day. Rockwell entrance (Station 68+) could be closed from 6:00 p.m. to 6:00 a.m. Monday through Thursday and from 6:00 p.m. Friday to 6:00 a.m. Monday. K-Mart entrance (Station 110+) could be closed from 10:30 p.m. to 10:30 a.m. any day. The following pairs of streets were not to be closed at the same time: Council Street and Rockwell Drive (Station 53+), Rockwell entrances (Station 68+) and C Avenue, and Northland Avenue and K-Mart entrance (Station 110+). Within 3 days after each mainline segment was completed past an intersection, the contractor was required to open that segment to traffic.

MATERIALS

The project called for the following materials:

Cement—
  Continental Type III (special—1,300 psi at 12 hr, ASTM C109)
  Lehigh Type III (special—1,300 psi at 12 hr, ASTM C109)
Fly Ash—
  Louisa Class C
Coarse Aggregate—
  Lee Crawford, Cedar Rapids (A57022)
  South Cedar Rapids, Cedar Rapids (A57018)
Fine Aggregate—
  Open Pit, Cedar Rapids (A57528)
Air-Entraining Admixture—
  Daravair, Double Strength, W. R. Grace & Co.
  Protex A.E.S., Prokrete Industries

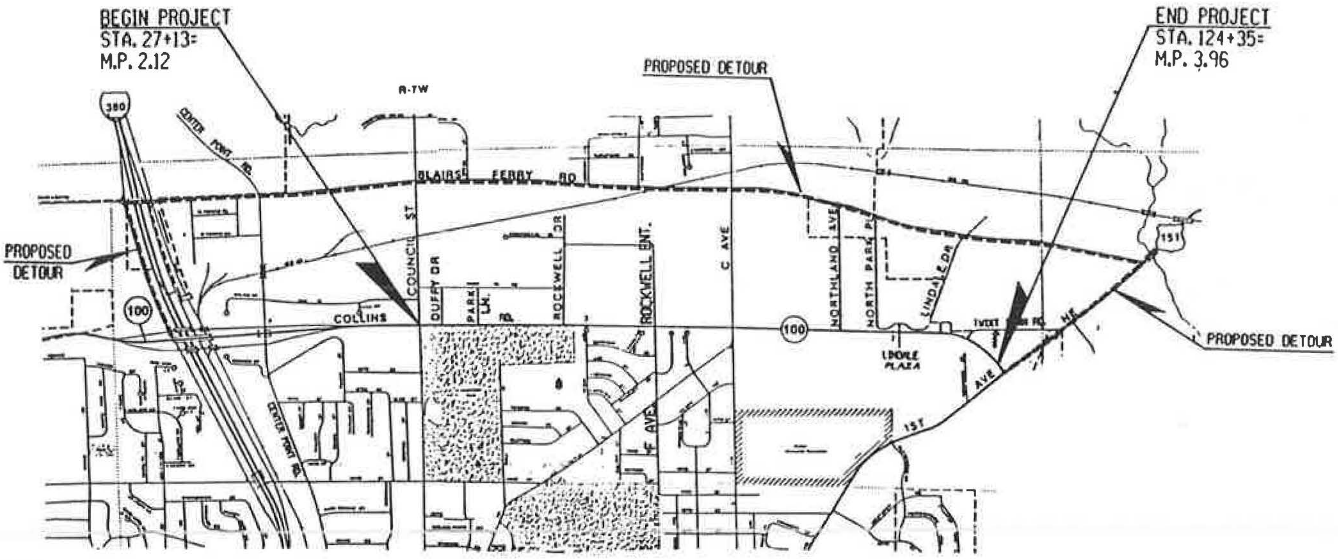

FIGURE 1 Project location.

Water-Reducing Admixtures—
  WRDA-82, W. R. Grace & Co.
  Pro-krete N-3, Prokrete Industries

The conventional Fast Track mix (Class F) consists of 710 lb of special Type III cement, 6 percent entrained air, 50 percent fine, and 50 percent coarse aggregate. Fast Track mix with fly ash contains 10 percent Class C fly ash substituted for Type III cement on a 1:1 weight basis. Fast Track II mix (Class FF) contains 822 lb of special Type III cement. The Fast Track II mix also permits a 10 percent fly ash substitution. The water/cement ratio for Fast Track mixes generally ranges from 0.48 to 0.40. Tables 1–3 show the Fast Track and Fast Track II concrete mix proportions, absolute volumes, and gradations used on the project.

TABLE 2 BASIC ABSOLUTE VOLUME OF MATERIALS

Material	Fast Track (Class F)	Fast Track II (Class FF)
Cement	0.120	0.139
Fly ash	0.016	0.018
Fine aggregate	0.312	0.294
Coarse aggregate	0.312	0.293
Water	0.180	0.196
Air voids	0.060	0.060

CONSTRUCTION PROCESS

Cedar Valley Corporation of Waterloo, Iowa, was the successful bidder for this $1.9 million project. General weather conditions during paving were sunny and warm with few rainy days. The average daily high temperature was 85°F and the average daily low temperature was 61°F. Paving started on June 19, 1989, and the roadway was entirely open to traffic on June 30, 1989. The remaining work was to be finished in 3 to 4 weeks.

Paving began on the west end of the mainline. As the paving train approached the intersection, a header was placed and paving was resumed on the other side of the intersection. A Rex-TBM belt placer and a CMI–SF350 slip form paver were used to place the 26.5-ft pavement and inside curb.

Before paving in the intersections, the pavement was removed and replaced with granular base and 3 in. of asphaltic concrete for a temporary wearing surface. The contractor was able to complete the removal and replacement during the 6:00 p.m. to 6:00 a.m. closure period. The remaining work on the westbound lanes proceeded without interference to intersection traffic.

Once the mainline pavement adjacent to an intersection gained sufficient strength to allow construction traffic, the contractor was permitted to begin the intersection work. The intersection was closed to traffic at 6:00 p.m. to begin the asphalt removal. The contractor removed the 3 in. of asphalt and base and prepared the grade in 3 to 4 hr. This left 2 to 3 hr to place 50 to 79 ft of Fast Track II pavement. The

TABLE 1 MIX PROPORTIONS

Mix	Cement (lb/yd³)	Fly Ash (lb/yd³)	Fine Aggregate (lb/yd³)	Coarse Aggregate (lb/yd³)	Air Entraining Admixture (oz)	Water Reducer Admixture (oz)	Mix Temperature (°F)
F	641	73	1,393	1,359	10	28.6	80
FF	742	80	1,305	1,302	11	24.8	93

TABLE 3 GRADATION PERCENT PASSING

	Aggregate Size					
	Fast Track (Class F)			Fast Track II (Class FF)		
Sieve	Coarse	Fine	Combined	Coarse	Fine	Combined
1-in.	100	–	100	100	–	100
¾-in.	88	–	94	77	–	89
½-in.	54	–	77	42	–	71
⅜-in.	20	100	60	9.6	100	55
#4	1.5	97	49	1.5	96	49
#8	1.1	89	45	0.6	88	44
#16	–	75	38	–	76	38
#30	–	45	23	–	45	23
#50	–	8.7	4.4	–	8.2	4.1
#100	–	0.8	0.4	–	1.2	0.6
#200	1.4	0.4	0.9	0.9	0.6	0.8

intersections were generally open by the 6:00 a.m. requirement. Penalties were assessed for being ½ hr late on opening two intersections and for being 3 hr late on one intersection.

No major problems were encountered in placing the Fast Track II mix, although initially maintaining the target entrained air content was a problem. Type III cement traditionally has required higher dosages of air entrainment than Type I cement. Finishers reported some difficulty in finishing the surface, although finishers on an experimental section of Fast Track II in Dubuque County had reported the mix to be easy to finish. No definite reason for this problem was determined. Some possible reasons are higher mix temperature, warmer weather, longer elapsed time between batching and finishing, or a different gradation or particle shape.

Concrete for the mainline was batched from a portable batch plant two blocks from the project. A local ready-mix plant 4 mi from the project supplied the Fast Track II concrete. Haul times were generally less than 20 min.

Both the intersections and the mainline were cured using a white pigmented curing compound, and burlene thermal blankets were placed over the pavement. At the intersections, the contractor placed the blankets within 1 hr after the section was poured. To avoid marring the surface on the mainline, placement of the blankets was usually delayed for several hours after pouring the concrete.

Joint sawing and sealing at the intersections began 4 to 5 hr after the concrete was placed. Preformed neoprene joint material was used to expedite the sealing process in the intersections. An ASTM D3405-type hot-pour material was placed on the mainline pavement.

PROJECT TESTING

The Fast Track mixes were subjected to strength testing, pulse velocity testing, and maturity testing.

Strength Testing

Both the flexural and compressive strength of the concrete were determined for two placements of Fast Track and two placements of Fast Track II. The locations and the results are presented in Table 4.

Seventy-five beams and cylinders were cast for testing. Vibrators were used for molding cylinders and beams. An external vibrator was used for 4½- × 9-in. horizontal cylinders, and an internal vibrator was used for 6- × 6- × 20-in. beams. Beams and cylinders were sprayed with curing compound and then were placed on the slab under the blankets.

Three beams and three cylinders were tested at each test time. The flexural strength of the concrete was determined using centerpoint loading. The test results are presented in Table 5 and shown graphically in Figures 2–5 both for the flexural and compression tests.

Pulse Velocity Testing

The FHWA staff has used the V-meter on active projects in several states, including Virginia, Pennsylvania, Ohio, Indi-

TABLE 4 PLACEMENT LOCATIONS AND CONCRETE TEST RESULTS

Mix	Station	Slump (in.)	Air (%)	Water/ Cement Ratio
Mainline				
Fast Track (F)	48+00	2.00	5.5	0.415
Fast Track (F)	118+75	1.75	5.0	0.411
Intersection				
Fast Track II (FF)	81+30	2.50	5.2	0.376
Fast Track II (FF)	99+80	2.25	5.2	0.382

TABLE 5 STRENGTH TEST RESULTS

Age	Average Flexural Strength[a] (psi)		Average Compressive Strength (psi)	
Fast Track (Mainline)				
	Station 48+00	Station 118+75	Station 48+00	Station 118+75
6 hr	270	150	1,680	1,500
12 hr	420	420	3,550	3,590
20 hr	460	NA	4,570	NA
24 hr	530	550	4,660	5,080
7 days	720	810	5,840	NA
14 days	790	810	NA	6,440
Fast Track II (Intersection)				
	C Avenue	Northland Avenue	C Avenue	Northland Avenue
5 hr	180	190	1,130	1,570
7 hr	360	380	3,840	3,550
9 hr	500	560	NA	4,250
12 hr	570	640	4,990	4,430
24 hr	690	840	5,260	5,230
7 days	950	940	NA	NA
14 days	1,000	1,040	7,090	7,470

NOTE: NA = Not available.
[a]Center point loading.

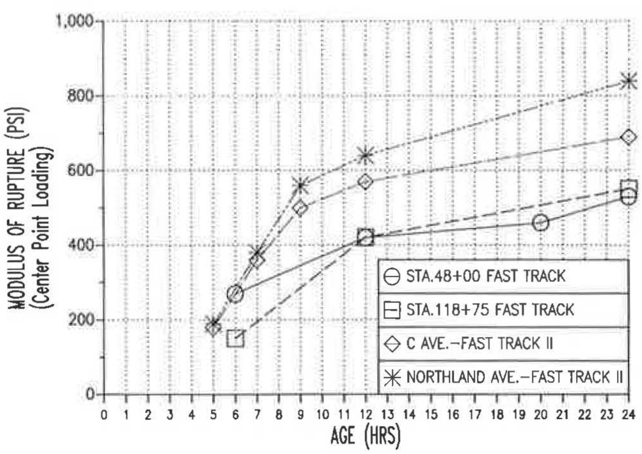

FIGURE 2 Early flexural strengths.

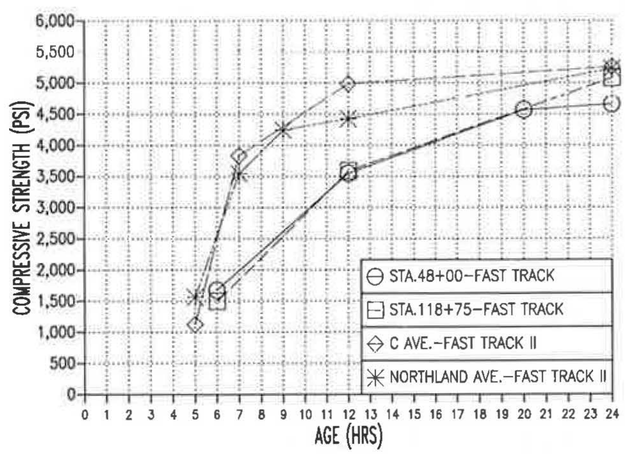

FIGURE 4 Early compressive strengths.

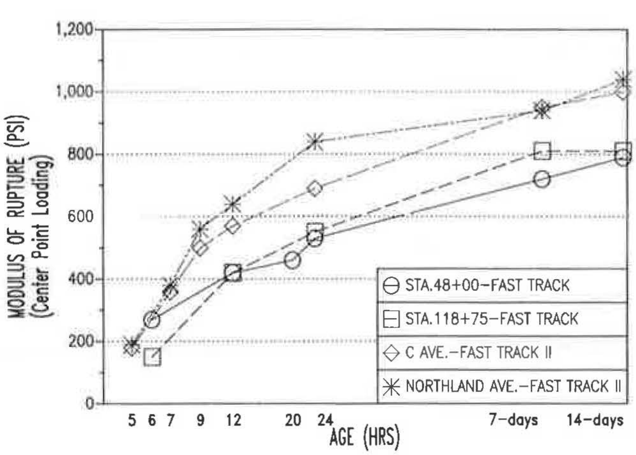

FIGURE 3 Long-term flexural strengths.

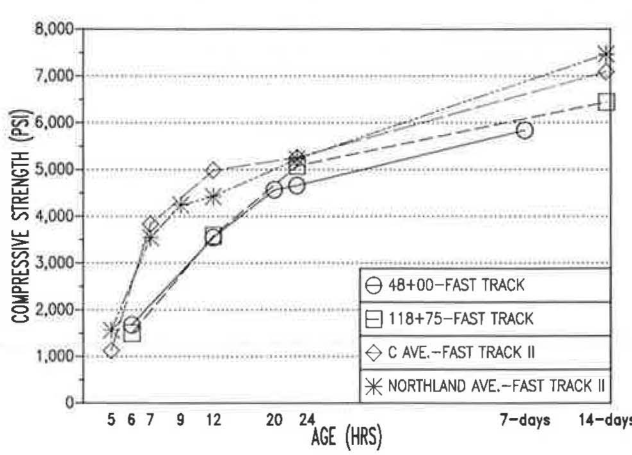

FIGURE 5 Long-term compressive strengths.

ana, Iowa, and Michigan. In general, good correlation was obtained between strength and pulse velocity, as measured by multiple correlation coefficients of ≥0.8. A unique setup for measuring pulse velocity on the pavement was established in the Cedar Rapids–Collins Road Fast Track project. The Iowa Department of Transportation designed and provided a hollow block-out device to form a hole 6 × 6 × 6 in. in the pavement after the texturing operation. The test required two holes 3 ft apart. The surface in contact with the ultrasonic transducers was smoothed out using a steel trowel. The two holes were then covered with insulating blankets. Every time a strength test was performed, ultrasonic pulse velocity measurement of the pavement was also taken. The best-fit line between the flexural strength and pavement pulse velocity is shown in Figure 6.

Maturity Testing

Maturity is defined as the accumulated product of the time and temperature. The precast/prefabricated concrete industry has widely used maturity-strength curves. Several commercial products are available that record temperatures on a continual basis. All use a temperature-measuring probe and a triggering time clock. Information is stored in a microchip board and can be retrieved after the test. The test for maturity is covered by ASTM C1074-87.

In the Cedar Rapids–Collins Road Fast Track project, the test locations in the pavement and field-cured test specimens were monitored by an M-meter. The following locations were monitored with the temperature probe thermocouples:

- At center slab—0.5 in. from top, 0.5 in. from bottom, and middepth;
- One foot from edge of slab—same locations as at center slab;
- In air; and
- At test specimens—4½-in.-diameter cylinder and 6- × 6- × 20-in. beam.

Figures 7 and 8 show the time-temperature data for one Fast Track section and one Fast Track II section, respectively.

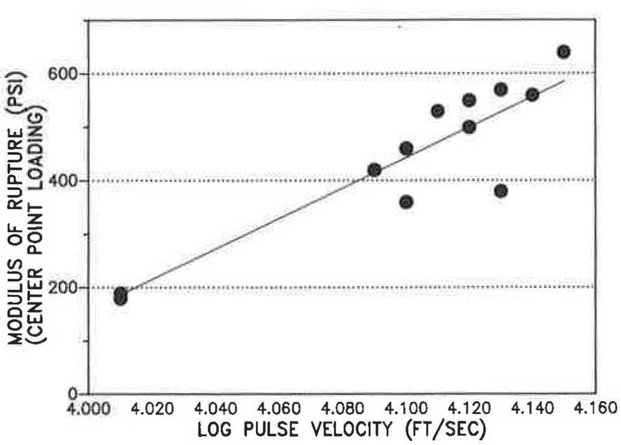

FIGURE 6 Pulse velocity versus flexural strength.

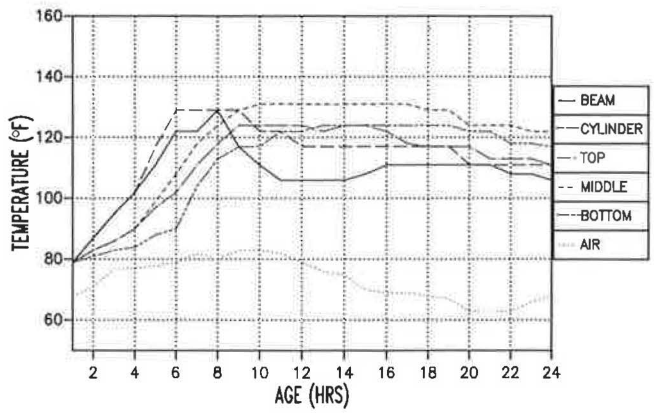

FIGURE 7 Fast track temperatures, Mainline Station 118+75.

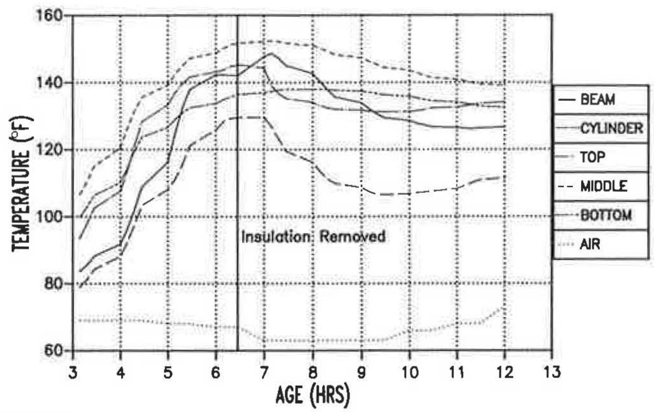

FIGURE 8 Fast Track II temperatures, C Avenue intersection.

The data were used to measure the correlation between maturity and flexural strength (integration of time-temperature plot using the Nurse-Saul equation). These correlations are shown in Figure 9.

RESULTS

The various tests and procedures that were carried out produced several results that have implications for determining when pavement may be opened to traffic.

Strength

The two Fast Track sections exhibited similar strength gains. The flexural tests resulted in nearly identical strengths at 12 hr, 24 hr, and 14 days. The compression tests were nearly identical for the first 20 hr. This consistency is an important verification of the results.

In less than 12 hr, both mainline test sections reached the 400-psi flexural strength required for opening. They reached 500 psi, the opening strength required for conventional paving, in less than 24 hr. The results of these tests, shown in Figures 2 and 3, are similar to other Fast Track projects constructed in Iowa in the past 3 years.

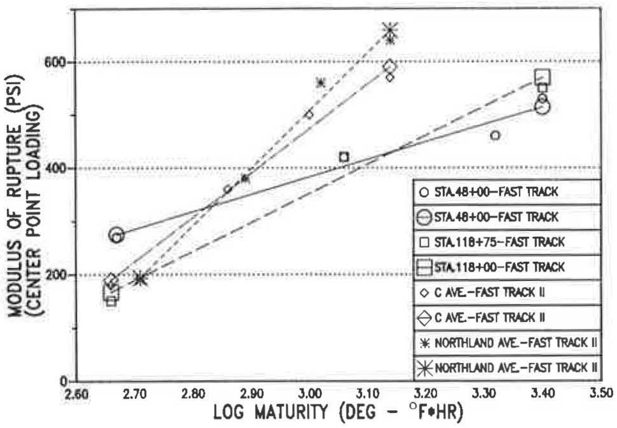

FIGURE 9 Maturity versus flexural strength.

The two Fast Track II test sections exhibited some variation, not only between the two intersections, but also between the flexural and compression test results. The differences do not suggest unusual problems or specific trends. Construction variations would likely account for the differences. Northland Avenue exhibited a continual increase in the rate of flexural strength gain when compared to C Avenue throughout the first 24 hr. At 7 days, however, the tests from the two sections showed virtually the same flexural strength.

The flexural strengths at both intersections were nearly identical for the first 7 hr. By that age, both intersections exceeded the 350-psi flexural opening strength requirement, even though both had strengths of less than 200 psi at 5 hr. The goal of this mix was achieving 350 psi in 6 hr. In both test sections, that strength was reached in less than 7 hr, but it required more than 6 hr. These results closely matched research the Iowa Department of Transportation conducted with the Fast Track II mix in Dubuque County, Iowa, in October 1988. This test took place under very different weather conditions and with different materials than in the Cedar Rapids–Collins Road project. Because all three test sections reached opening strength between 6 and 7 hr, achieving these results in future projects appears likely.

The compression tests gave impressive results. The cylinders gained considerable strength in 5 to 7 hr. The compressive strength went from about 1,500 psi to more than 3,500 psi in that 2-hr period. Although the rate of gain then began to decline, compressive strength still reached almost 4,500 psi at Northland Avenue and about 5,000 psi at C Avenue in 12 hr. The rate of flexural strength gain at these early ages was slower than that for compressive strength. The rapid rate of strength gain began to decrease at approximately 7 hr, according to the compression tests, but the rate of gain in flexural strength did not decrease until 9 hr.

Pulse Velocity

Figure 6 shows the relationship between the pulse velocity and the flexural strength. This integration yielded R^2 values of 0.88. Pulse velocity has potential for this type of application. The difficulty comes in trying to establish vertical surfaces from which to take measurements. Because materials change from project to project, application of the pulse velocity test will probably be limited to larger projects. Those projects involving a large quantity of concrete will justify the initial correlation effort.

Maturity

The temperature-age plots for one of the Fast Track mainline test sections are shown in Figure 7. The maturity of the pavement was only slightly higher than that of the flexural beams at the 12-hr test.

Compression tests were performed on the cylinders. The maturity of the cylinders differed between the two test sections. At Station 48 + 00, the cylinder reached a higher maturity than the slab at 12 hr, except at Station 118 + 75, in which the maturity of the cylinder was lower than that of the slab. The cause is certainly a result of the sun's heating up the small specimen at the first section. This research suggests that the flexural beams give a strength reading that more closely represents the actual concrete in the pavement than do cylinders.

For Fast Track II, the temperature-time plots are shown in Figure 8. The maturity of the concrete in the two Fast Track II test sections was more consistent than in the Fast Track sections. The temperatures in the pavement of the Fast Track II and the test specimens were more consistent in the Fast Track sections. The sun's shining on the specimens during the day caused them to become warmer than the slab on the Fast Track sections. This warming effect was not a factor in the nighttime Fast Track II tests. The flexural beams and cylinders exhibited consistently less maturity than the pavement slab. Apparently, a margin of safety exists between the strength the test specimens indicate and that of the actual pavement. The strength in the pavement is in fact higher than the test results show.

Figure 9 is a plot of the maturity-flexural strength results. The figure shows the best-fit lines for the test data and represents predictive models that have R^2 values >0.8.

Maturity testing for opening time has potential for Fast Track paving. Future maturity evaluations on Fast Track may prove the viability of maturity testing instead of flexural beams to determine opening time.

CONCLUSIONS

The results of the Cedar Rapids project imply the following conclusions:

1. Fast Track and Fast Track II can be placed with conventional paving procedures and equipment.
2. Fast Track II will achieve a 350-psi flexural strength for opening to traffic in less than 7 hr.
3. Fast Track will achieve a 400-psi flexural strength for opening to traffic in less than 12 hr.
4. Fast Track II will require higher than normal amounts of air-entraining agent to obtain desired entrained air. The high cement factor mix may require more effort to float and finish.
5. Construction staging and restrictions required for the project were achievable. The system is applicable for certain future projects.

RECOMMENDATIONS

1. This project should be evaluated for performance after 5 years of service. Cracking, smoothness, and friction properties should be summarized.
2. Study should continue on the occasional problem of finishing Fast Track II concrete.
3. Further evaluation of the maturity concept should be performed on future Fast Track projects.

ACKNOWLEDGMENTS

The evaluation portion of the research was partially funded by the FHWA under Special Project 201, "Accelerated Rigid Paving Techniques." The authors wish to extend appreciation to the District 6 Materials staff and the Cedar Rapids Construction Residency staff for performing the additional testing this project required. The commitment by the FHWA of their mobile concrete laboratory and personnel contributed to the success of the research and was greatly appreciated. The authors also want to thank Cedar Valley Corporation for its cooperation during the project.

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Roller-Compacted Concrete Pavement Construction at Fort Drum, New York

EDEL R. CORTEZ AND JOHN A. GERLACH

During the 1988 and 1989 construction seasons, 335,000 m^2 (83 acres) of roller-compacted concrete (RCC) pavement 0.254 m (10 in.) thick was built for the U.S. Army, 10th Mountain Division (Light Infantry) at Fort Drum, New York. The design, construction, quality control practice, and durability of the RCC under the seasonal frost environmental conditions at this location were studied.

During the design phase of the new U.S. Army Fort Drum post, roller-compacted concrete (RCC) was selected as the best suited and most economical alternative for the construction of hardstand pavements used as parking area for heavy vehicles. The pavements must be capable of withstanding the action of heavy, low-speed, rubber-tired, or tracked vehicles such as troop-carrying trucks and tanks. In addition, the material used for these pavements must endure the low temperatures and multiple freeze-thaw (F-T) cycles that occur in northern New York State.

RCC is a construction technology that combines the features of cement-treated aggregate base, portland cement concrete (PCC), and asphalt pavements to produce a low-cost pavement material. RCC is constructed by placing a zero-slump PCC mixture by means of a heavy asphalt paver and compacting it with several passes of a vibrating roller. Large quantities of concrete can be placed quickly with a minimal amount of labor and equipment. No forms are needed and finishing is not required. These attributes save approximately 30 percent compared to pavements built with conventional methods.

Work crews from Black River Constructors (BRC, a subsidiary of Morrison-Knudsen), under the technical direction of Peltz Constructors, constructed the Fort Drum pavements. Peltz supplied the pavers, the portable mixing plant, and the technical expertise for the placement of the RCC.

RCC has not been successfully air entrained to date. Preliminary tests run on samples from early projects (*1*) revealed susceptibility to F-T cycles, so the U.S. Army Corps of Engineers Cold Regions Research and Engineering Laboratory (CRREL) conducted an extensive research program on the F-T durability of RCC. CRREL is now testing samples from the Fort Drum RCC pavement and is monitoring the field performance through a system of sensors installed within and around the pavement.

E. R. Cortez, U.S. Army Corps of Engineers Cold Regions Research and Engineering Laboratory, Hanover, N.H. 03755-1290. J. A. Gerlach, U.S. Army Engineer District, Construction Management Office, Fort Drum, N.Y. 13602-5200.

PAVEMENT DESIGN

The design procedure for RCC pavements is similar to, and follows the methodology for, designing conventional concrete pavements. The criteria affecting thickness design are flexural strength, fatigue, the supporting strength of the base and subgrade combination, and the type and volume of traffic. The U.S. Army Corps of Engineers has developed thickness design charts for RCC (*2*). RCC pavements typically are allowed to develop shrinkage cracks naturally. They occur at irregular intervals ranging from approximately 12 to 21 m (39 to 69 ft). Often, shrinkage cracks originate where the pavement has been discontinued or cut off to fit utilities. Test data from other RCC projects on load transfer resulting from aggregate interlock revealed an average of 18 percent load transfer with a large standard deviation (*3*). Therefore, thickness design of RCC pavements is based on no load transfer at the joints or cracks. The result is a thicker pavement.

The contractor determined the proportioning of the Fort Drum RCC mixture. Several trial mixtures were prepared to provide a range of flexural strengths (Figure 1). The mix design that was selected produced a 28-day flexural strength of 5.5 mega-Pascals (MPa) (approximately 800 psi), which exceeded the contractual specification of 4.83 MPa (approximately 700 psi). Table 1 presents the selected mixture, and Table 2 displays the aggregate gradation. Coarse and fine aggregate were both manufactured by and obtained from a local limestone quarry. PCC Type II and fly ash Type F from the Canadian manufacturer LaFarge were used. Approximately 25 percent of the cementitious material was fly ash. The water/cement ratio was 0.41.

CONSTRUCTION

Mixing, Transportation, Placement, and Compaction

The RCC paving operation commenced in June 1988 and was concluded in August 1989. A continuous twin-shaft pugmill mixing plant was installed at approximately 5- to 10-min haul time from the paving sites. The plant was capable of producing 535 m^3 (700 yd^3) of concrete a day. Dump trucks of 10-m^3 (13-yd^3) capacity hauled the fresh concrete to the paving site. Two heavy-duty pavers placed the concrete while working in tandem to minimize cold joints. These pavers have a heavy, vibrating screed and dual tamping bars, which were able to obtain 93 percent of the final density at the back of the paver. The design called for a pavement 25.4 ± 0.6 cm (10 ± 0.25 in.) thick.

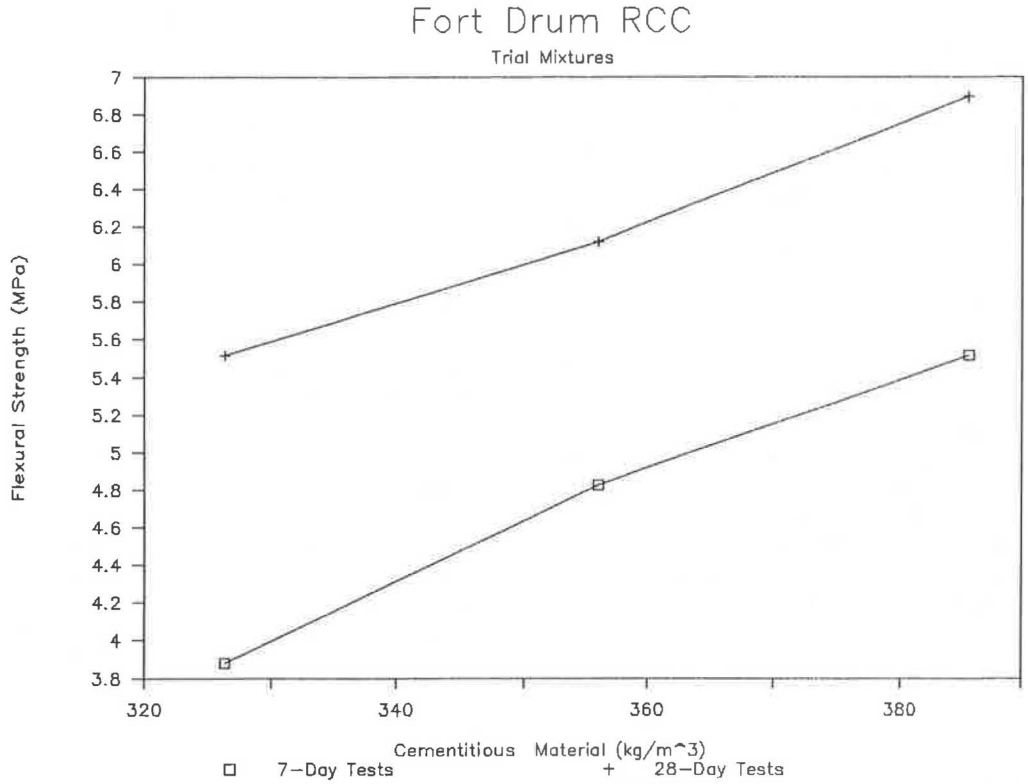


FIGURE 1 Flexural strength of trial mixtures.

TABLE 1 RCC SELECTED MIXTURE

Ingredient	Weight (kg)	Weight (lb)	Blend (%)	Absorption (%)	Volume (m^3)
Cement	245.0	540.2	–	–	0.078
Fly ash	81.3	179.2	–	–	0.035
Crusher run	1,379.0	3,041.0	70	0.7	0.513
NY DOT #1B	586.8	1,293.6	30	1.1	0.220
Water	134.0	295.6	–	–	0.134
Entrapped air	0.0	0.0	–	–	0.020
Total	2,426.1	5,349.6	–	–	1.000

TABLE 2 COMBINED AGGREGATE GRADATION

Sieve Size (mm)	Sieve Size (U.S. Customary System)	Broad Specification Requirements (percentage of material passing)
19.000	¾-in.	100
12.700	½-in.	90–100
9.500	⅜-in.	75–90
6.350	¼-in.	55–75
2.000	#10	32–48
0.430	#40	11–24
0.180	#80	6–15
0.072	#200	3–7

Initially, the RCC was placed in two identical lifts to result in the 25.4-cm design thickness after compaction. Direct shear tests conducted on cores indicated poor bonding between layers. The contractual specifications required 98 percent of the maximum dry density of 2,315 kg/m^3 (144.5 lb/ft^3), as obtained from the five-point modified Proctor test. The optimum density was obtained at 5.8 percent moisture content. The contractor suggested that, by placing the concrete in one lift instead of two lifts, the interlayer bonding problem could be avoided, while still obtaining the required density. The production rate would also be expedited with economic advantage. In order to obtain the 25.4-cm thickness when compacted, the RCC was placed in one lift at a height of 28 cm (11 in.). The extra thickness was added to offset the approximate 10 percent reduction in thickness caused by rolling.

A 10-ton dual-steel drum vibrating roller followed the paver, compacting the concrete within 30 min after the addition of water to the aggregate cement mixture at the plant. Four vibrating passes (one back-and-forth motion is considered two passes) were necessary to achieve the specified density. Initially, rubber tire rollers were used to tighten the surface texture. The rubber tire rolling was soon discontinued because little advantage was gained from it.

Joints

Both horizontal and vertical joints are used in working with RCC. Horizontal joints are those between two layers of RCC. Vertical joints are perpendicular to the top surface of the pavement. At Fort Drum, only vertical joints were used, because the concrete was placed in a single lift.

Any RCC joint may be either a fresh joint or a cold joint. A joint finished within the initial setting period of the concrete is considered a fresh joint, otherwise it is a cold joint. The initial setting period varies with the placing temperature, cement type, and other factors. For most cases, this period is between 45 to 60 min from the time the water and cement are mixed.

To create a fresh joint and to minimize cold joints, the specifications required that adjacent lanes be placed and compacted together within 60 min. This condition was met by working the pavers in tandem and by limiting the distance between pavers. Cold joints were constructed by removing approximately 15 cm (6 in.) of the outer edge of the hardened paving lane by sawing. This distance was determined by noting the distance from the edge where the density reached the required 98 percent.

Shrinkage cracks were allowed to develop naturally in most of this project. The contractor was required to rout and seal all cracks with a bitumen sealer. Generally, these cracks were irregular in shape and spacing, so the routing might not intercept the entire crack. In an effort to minimize this problem, at a late stage of the project, the contractor decided to saw cut control joints at 18-m (59-ft) spacing to a depth of one-third of the pavement thickness. This practice was shown to be effective in controlling the shrinkage cracks at most of the locations where it was done.

Finishing and Curing

The RCC construction process does not include a finishing operation. The surface left behind the roller is the final one. The surface texture is similar to that of an intermediate asphalt course. The curing consisted of gentle water spray applied directly to the exposed concrete surface. The flow of water is controlled to keep the surface of the pavement with a wet appearance for 7 days. Failure to do so may result in shallow microcracks and spalling. Curing compounds are not used with RCC because of the low water/cement (*w/c*) ratio and the openness of the pavement surface.

QUALITY CONTROL

Several tests and techniques were used to monitor the quality of the Fort Drum RCC construction.

Strength

Beams and cores were taken during construction to verify the density and evaluate the compressive strength of the concrete. Once the required density was achieved, no difficulty arose in attaining the specified compressive and flexural strength for any location.

Density

Density is the most important control parameter in RCC construction. The density is measured in the freshly compacted concrete by means of a nuclear density and moisture meter in the direct transmission mode. The density is reported as a percentage of the maximum dry density as obtained from the five-point modified Proctor test. In this test, zero-slump concrete is treated as a soil. The mix proportions are determined by conventional methods using a *w/c* ratio of 0.38. The *w/c* ratio is then allowed to vary up and down from 0.38. A set of five samples with *w/c* ratios of 0.32, 0.35, 0.38, 0.41, and 0.44 are compacted as prescribed by the standard modified Proctor test commonly used for soils. The unit weight of each sample after compaction is determined, and a portion of it is then oven-dried to determine its moisture content as a percentage of the dry weight. Moisture content on the horizontal axis and calculated dry unit weight on the vertical axis are plotted. The peak of the curve so formed is identified to obtain the optimum moisture content and the 100 percent maximum dry density. The corresponding *w/c* ratio is also recorded, but the ratio is increased slightly at the mix plant to compensate for evaporation during transportation and placement.

The nuclear gauge is initially calibrated with a block of concrete of measured unit weight to determine any necessary correction constant for the actual field measurements.

Experience from prior RCC projects has indicated that the density in the vicinity of vertical joints tends to be lower than the density at other points. This may lead to spalling along the joints. At Fort Drum, density measurements were obtained along the joints and the paving lanes at 61-m (200-ft) intervals by means of a single-probe nuclear density meter in the direct-transmission mode. The probe was inserted within the RCC pavement along one side of the joint, with the apparatus straddling the joint, thereby determining the density across the joint. The values obtained were checked against the density required for acceptance. Core samples were taken at the joints to verify the nuclear gauge readings. The core diameter, height, weight, and moisture content were determined in the laboratory to calculate the corresponding dry unit weight. The dry unit weight determined in the laboratory agreed with the density given by the nuclear gauge within 5 percent for most cases.

Smoothness and Texture

The smoothness of the surface was checked using a 3-m (10-ft) straightedge on lines 1.53 m (5 ft) apart, parallel with, and at right angles to, the centerline of the paved area. Undulations in the surface of less than 1 cm (3/8 in.) were initially noticed at regular intervals of 2.44 to 3 m (8 to 10 ft). Water would pond and freeze in these undulations, posing a potential safety problem. These undulations were caused by the paver screed settling when the paver hopper was allowed to be empty while waiting for a new supply of concrete. The contractor corrected this problem by maintaining a more continuous feed of concrete to the paver hopper and by adjusting the sensors that control the settlement of the paver screed.

F–T DURABILITY RESEARCH

A full-scale RCC laboratory F–T experiment conducted at CRREL (*3*) showed that, after 300 F–T cycles, the material properties did not reveal any important decay. The compressive strength measured after 300 F–T cycles was slightly higher than the compressive strength at the beginning of the cycling. The dynamic modulus of elasticity after 300 F–T cycles was between 66 and 91 percent of its value at the beginning of the test. The temperature limits and the induced moisture conditions during this experiment were rather severe.

Field Monitoring of the Fort Drum RCC Pavements

In 1988, CRREL installed one set of environmental sensors in each of two representative locations at the Fort Drum RCC project. Each set of sensors measures the following parameters:

- Air relative humidity,
- Precipitation,
- Barometric pressure,
- Wind speed,
- Wind direction,
- Direct solar radiation,
- Reflected solar radiation,
- Concrete moisture,
- Groundwater table,
- Air temperature,
- Pavement surface temperature,
- In-concrete temperature at several depths,
- In-base/subbase temperature at several depths, and
- Subgrade temperature at several depths.

Athough presentation of detailed data from that study is beyond the scope of this paper, a summary of the temperatures during the 1988-to-1989 winter is presented in Figures 2a–2i and in Figure 3.

Figure 2a shows the hourly air temperatures 1.37 m (4.5 ft) over the pavement surface. Sixty-five F–T cycles were recorded from November 15, 1988, to April 24, 1989. For the moisture in the atmosphere, one F–T cycle occurs when the temperature falls below 0°C and then returns to above 0°C.

Figure 2b shows the temperatures at the concrete upper surface. The sensor was embedded in the concrete at only 2 mm from the pavement surface. Twenty-four F–T cycles were recorded during the 1988-to-1989 winter. For the moisture inside the concrete, one F–T cycle occurs when the temperature falls below −5°C and then returns to above 0°C.

Figures 2c–2e show the temperatures and number of F–T cycles at several depths inside the concrete during the study period.

Figure 2f shows data from a temperature sensor embedded in the concrete at only 4 mm from the RCC-base interface. Only four F–T cycles were recorded during the 1988-to-1989 winter.

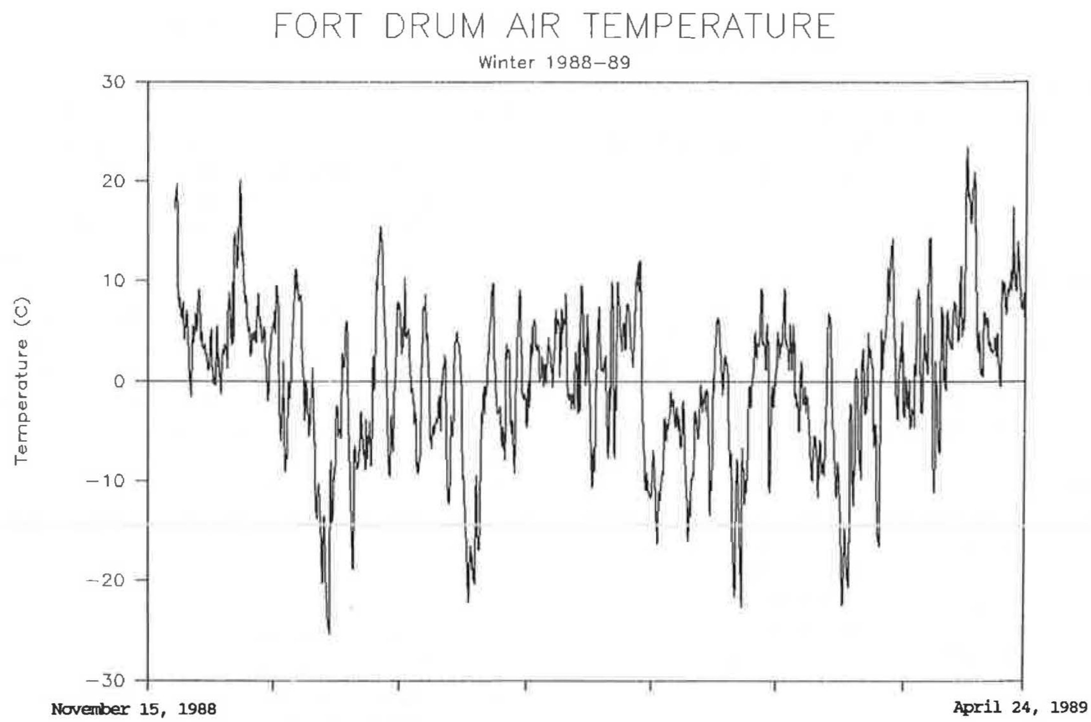
Figure 2g shows data from a sensor embedded in the granular base course. For the moisture inside a granular soil, an F–T cycle occurs when the temperature falls below −0.5°C and then returns to above 0°C. Nine F–T cycles were recorded at this point during the test period.

Figures 2h and 2i show data from temperature sensors located within the subgrade material. For moisture inside a silty soil, one F–T cycle occurs when the temperature falls below −2°C (or lower in many cases) and then returns to 0°C. Because no recorded temperature reached −2°C, no F–T cycle occurred.

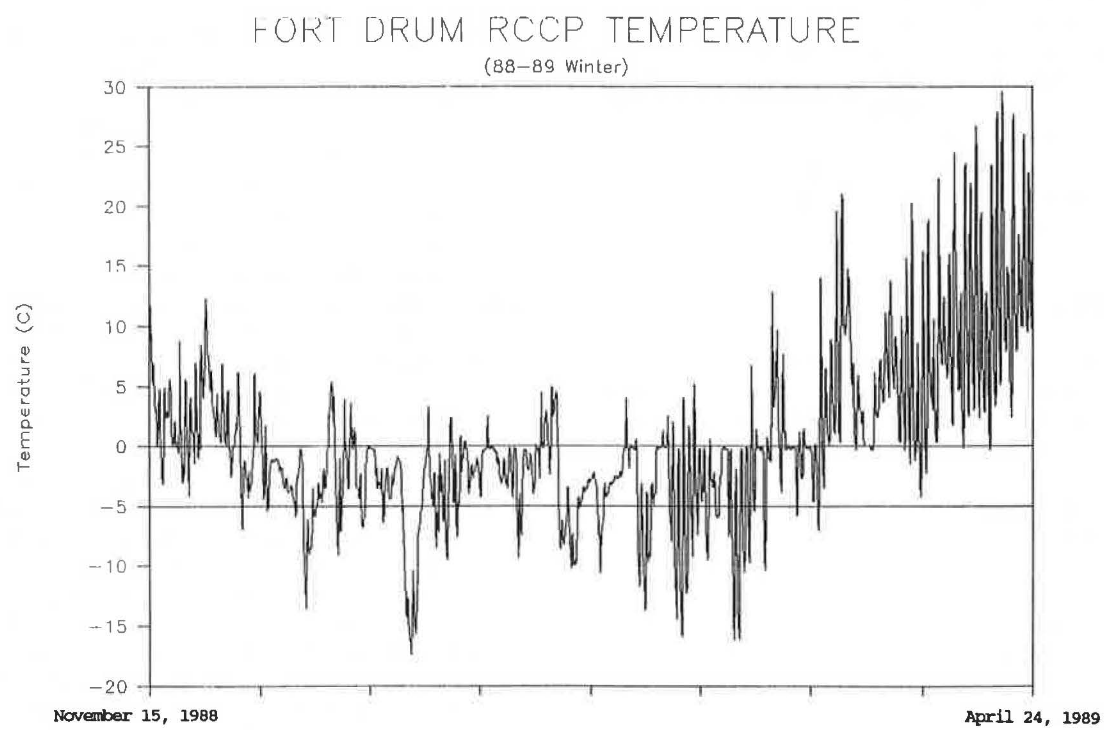
Figure 3 shows a profile of the number of F–T cycles that occurred at several depths through the RCC slab, the base course, and the subgrade. Notice that 65 F–T cycles were recorded in the air but only 24 cycles at 2 mm below the pavement upper surface.

An air F–T cycle and an in-pavement F–T cycle differ conceptually. Measurement of an air F–T cycle uses temperature data from sensors located at 1.37 m above ground level (standard weather thermometer level). It is based on the freezing point of pure water at atmospheric pressure. One F–T cycle occurs when the temperature falls below 0°C and then returns to above 0°C.

The in-pavement F–T cycle concept uses data from temperature sensors embedded in the pavement material at several depths. It is based on the temperature at which freezing of capillary water starts. This temperature was found to be approximately −5°C (*4*). Moisture in concrete is an alkaline solution rather than pure water. If a sealed container having a small air release valve is filled with water to 91.7 percent of its volume and then subjected to a sustained subfreezing temperature, the water will occupy 100 percent of its volume; the air will be expelled. If the same container is filled with more than 91.7 percent of its volume and then subjected to a sustained subfreezing temperature, the excess water will be expelled. If the container is a concrete capillary void critically saturated (more than 91.7 percent of its volume is filled with moisture) and the release valve is a gel pore, the excess moisture will be expelled through the pores when the water freezes, causing high hydraulic pressure. If this pressure exceeds the tensile strength of the cement paste, breakdown occurs. Therefore, in the in-pavement F–T cycle concept, a cycle



(a)

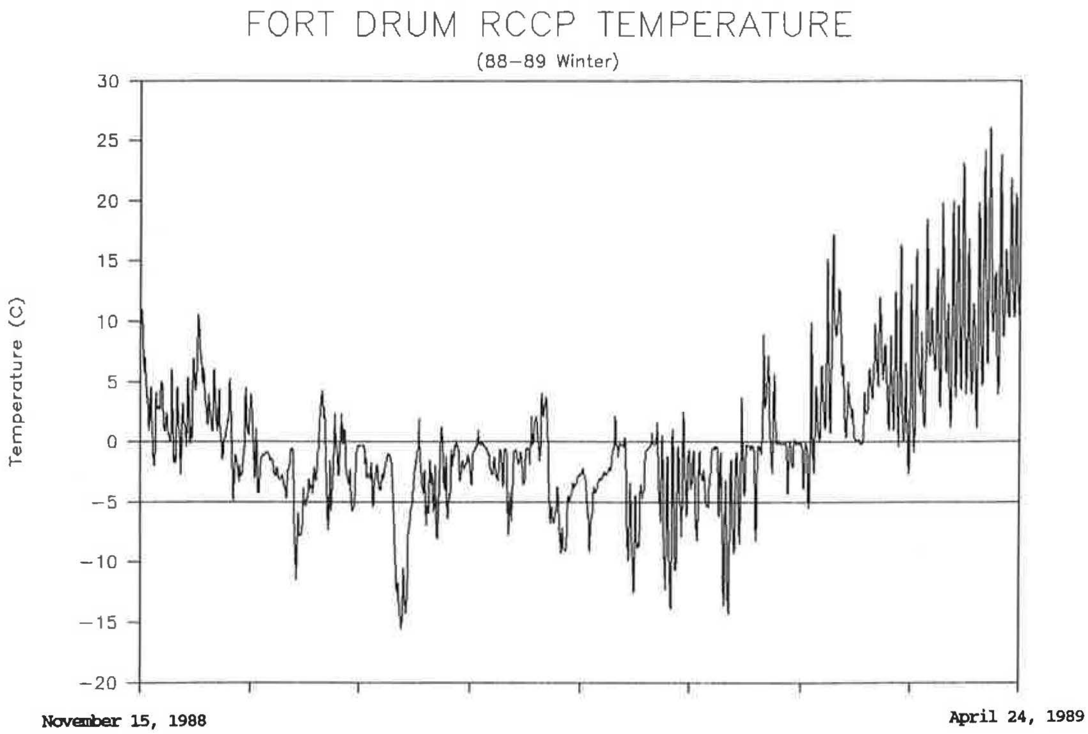


(b)

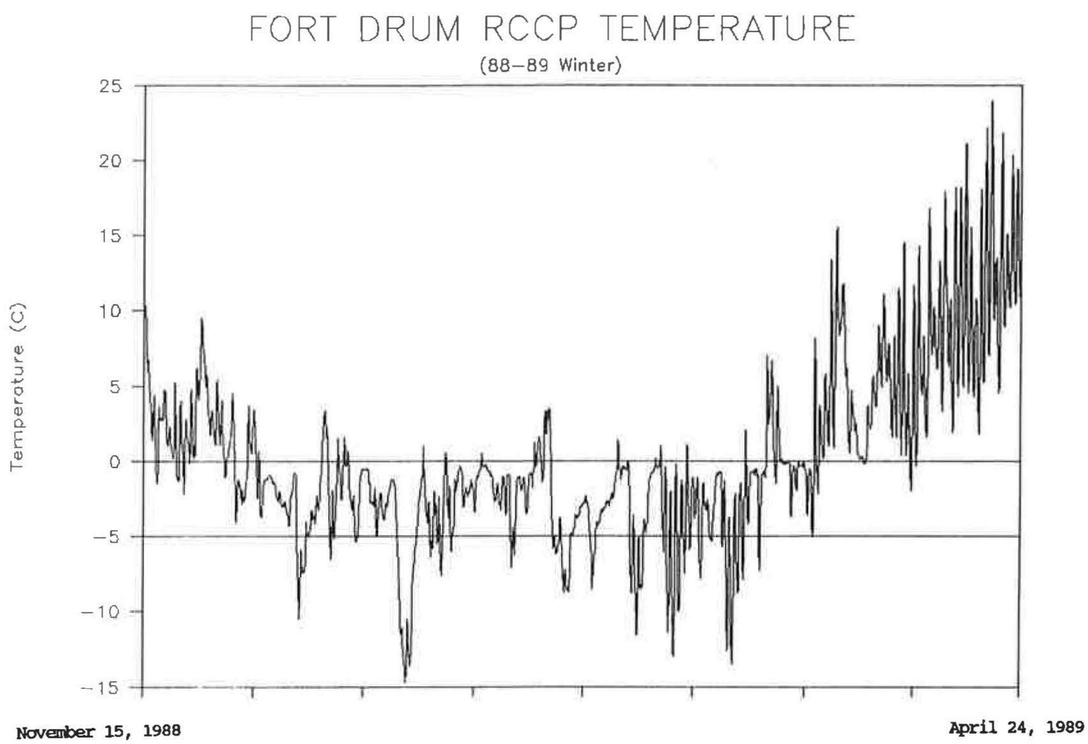
FIGURE 2 Temperatures (a) at 1.37 meters (4.5 ft) over the pavement surface; (b) at the pavement surface (2 mm into RCC); (c) at 0.05 m (2 in.) below the pavement surface; (d) at 0.076 m (3 in.) below the pavement surface; (e) at 0.15 m (6 in.) below the pavement surface; (f) at 0.25 m (10 in.) below the pavement surface; (g) in the granular base 0.05 m (2 in.) below the RCC-base interface; (h) at 0.86 m (34 in.) below the pavement surface, within the glacial till subgrade; and (i) at 1.47 m (58 in.) below the pavement surface, within the glacial till subgrade.

FIGURE 2 ***(continued on next page)***

FIGURE 2 ***(continued)***



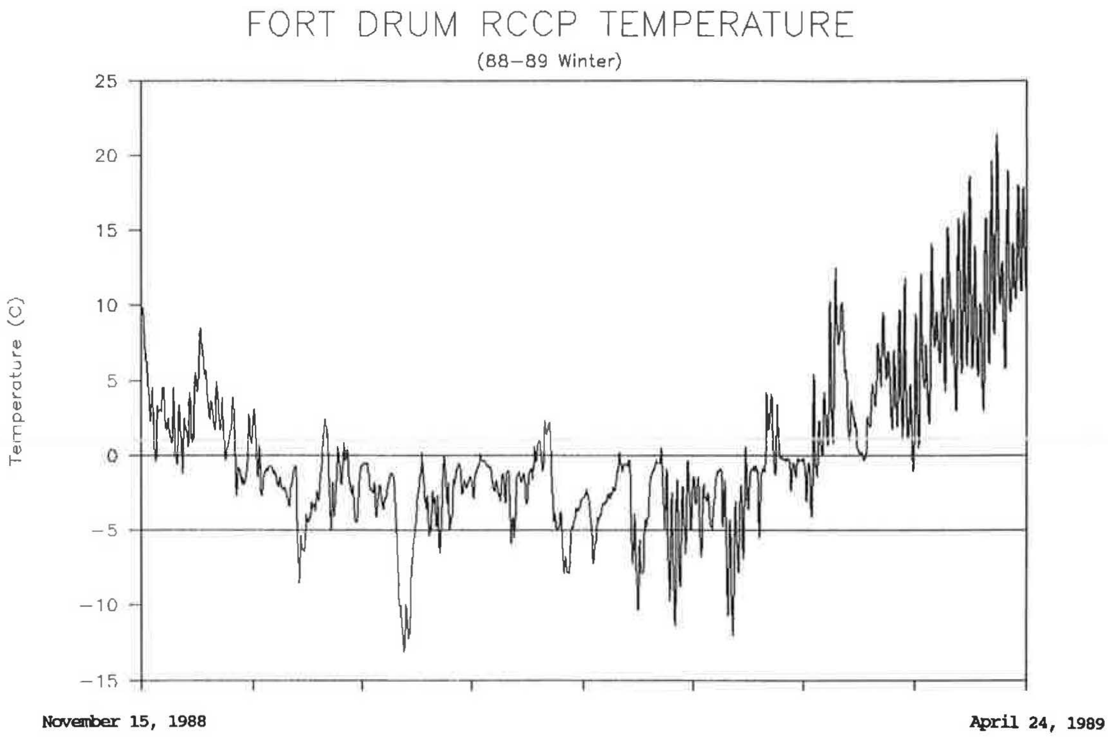
(c)



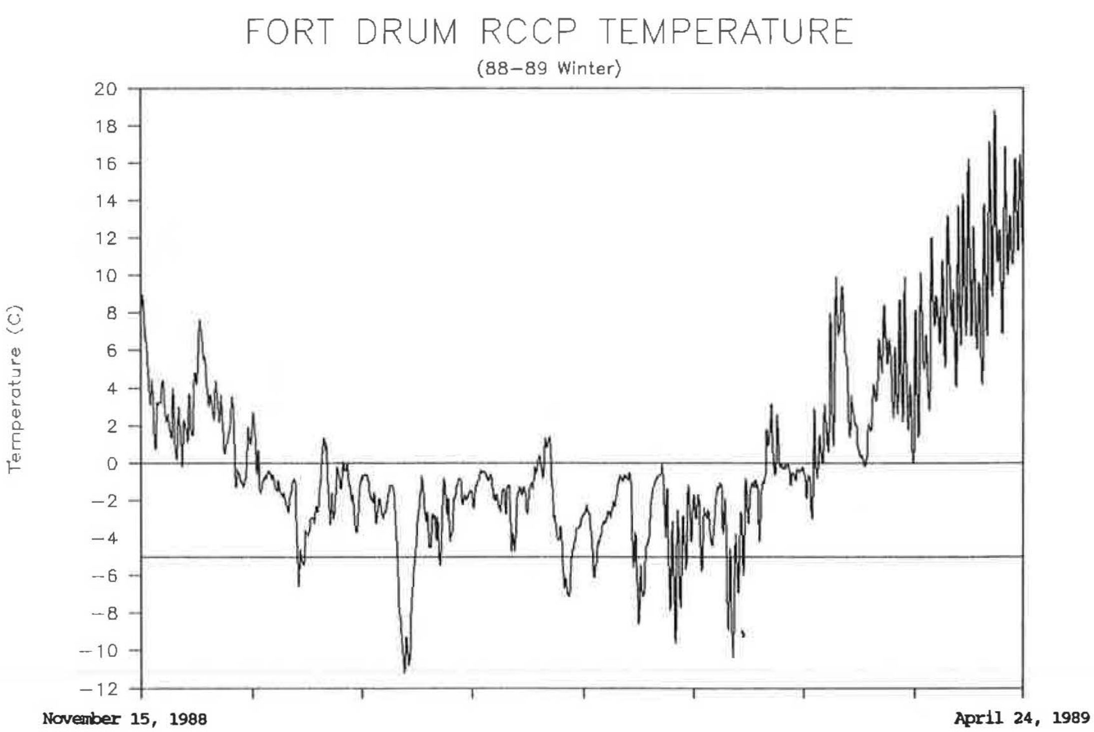
(d)

FIGURE 2 ***(continued on next page)***

FIGURE 2 ***(continued)***



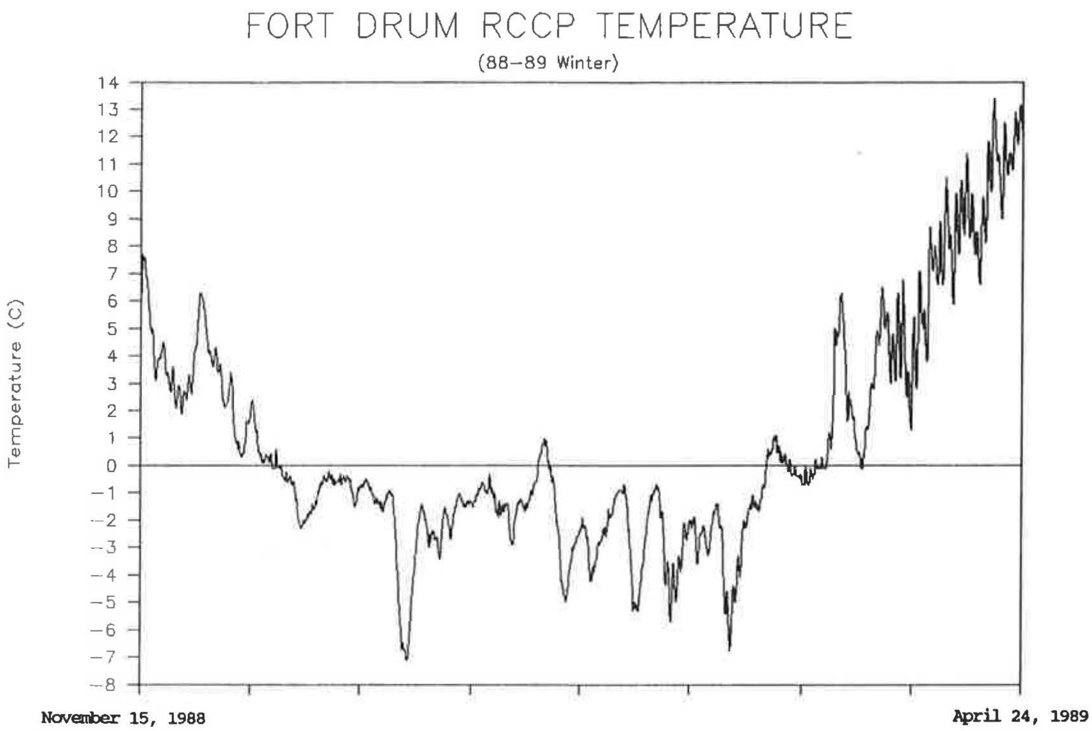
(e)



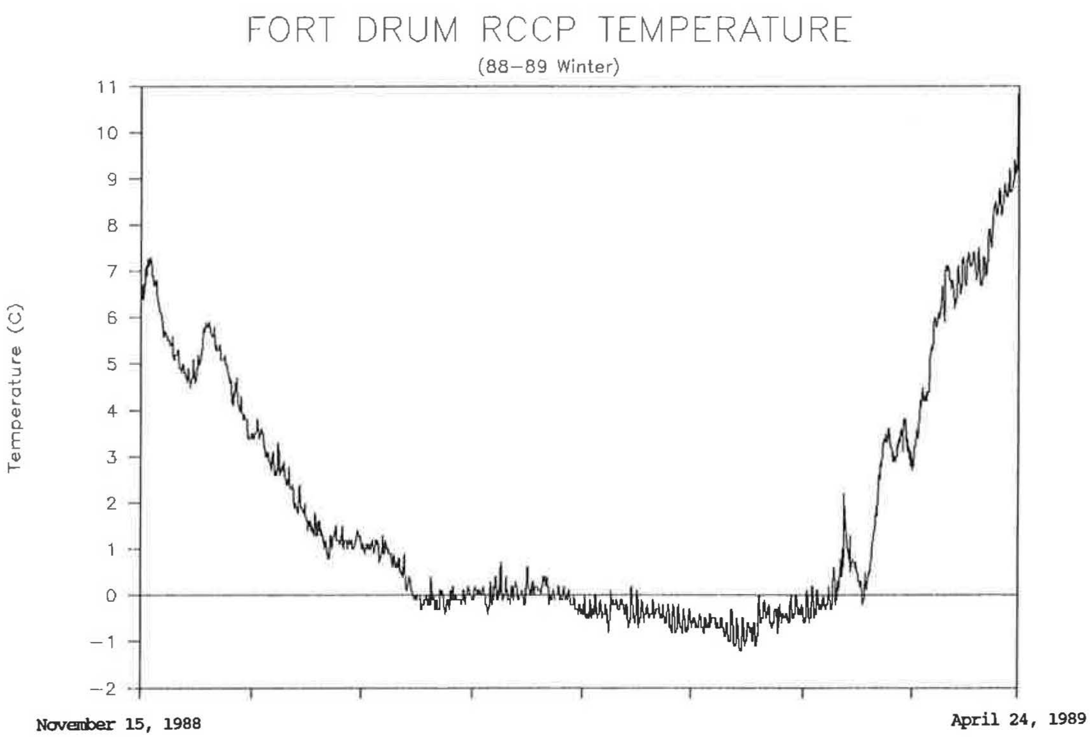
(f)

FIGURE 2 ***(continued on next page)***

FIGURE 2 ***(continued)***



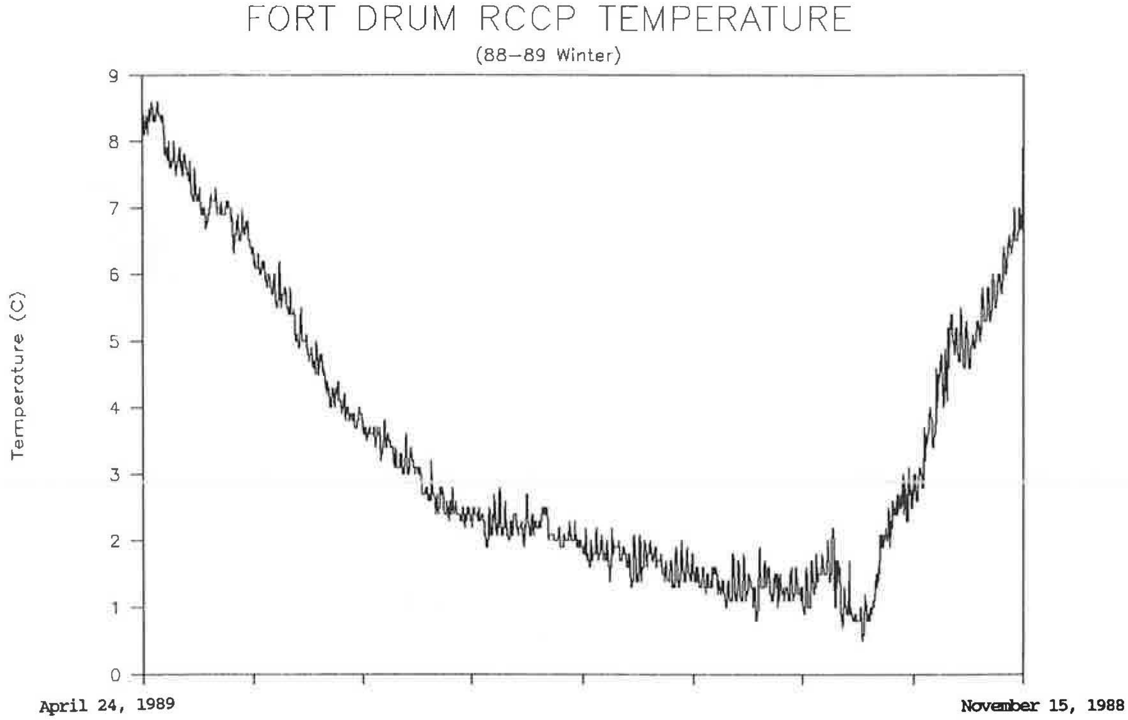
(g)



(h)

FIGURE 2 ***(continued on next page)***

FIGURE 2 *(continued from previous page)*



(i)

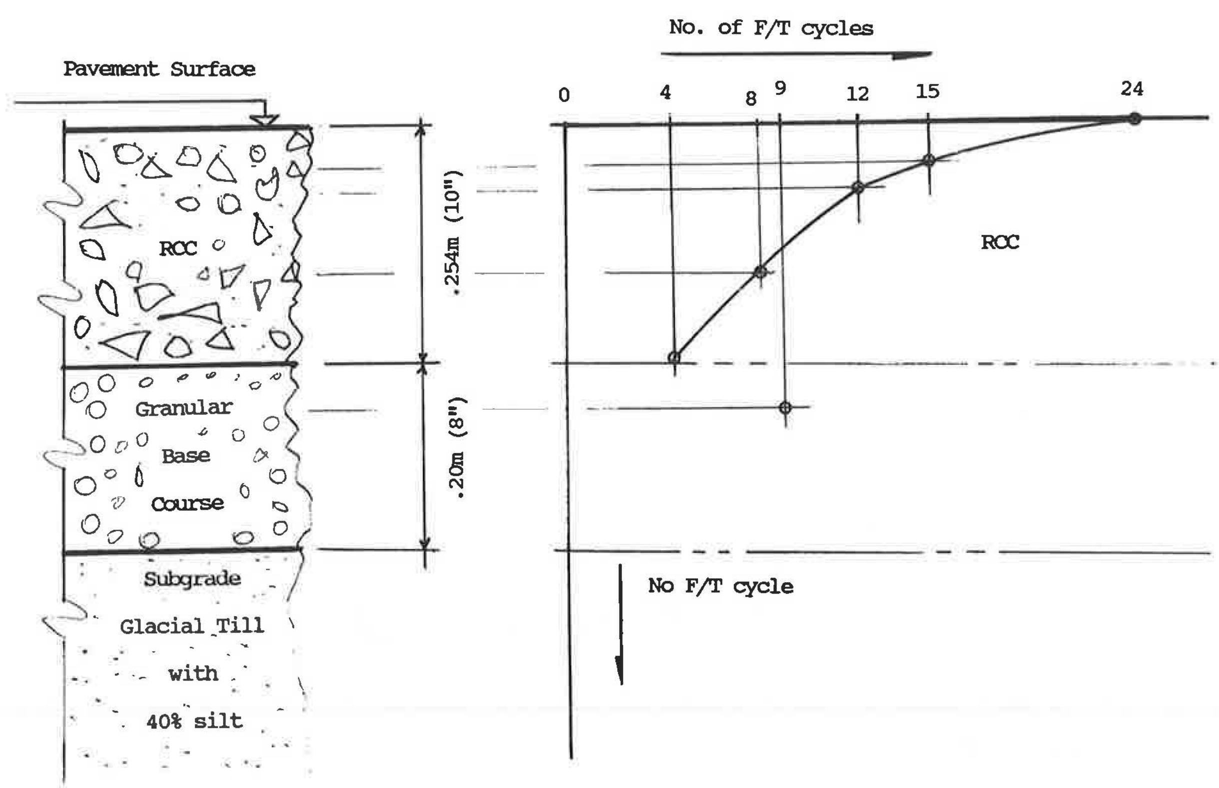


FIGURE 3 Fort Drum RCC pavement freeze-thaw cycle profile, 1988-to-1989 winter.

occurs when the temperature falls below $-5°C$ and then goes above 0°C.

Typically, as occurred here, the density of an RCC pavement decreases gradually from top to bottom. In Figure 3, the number of F–T cycles decreases in the same manner. Thus, although the lower density at the lower levels reduces the F–T durability of the material, the number of F–T cycles is also smaller at those depths.

Future Research

Using the RCC technology in large projects commonly permits savings of about 30 percent compared to alternative paving methods. This advantage explains the growth in acceptance of RCC construction. The F–T durability concern has diminished in view of the results from the Fort Drum field study. In 1990, the U.S. Army Corps of Engineers, Pennsylvania State University, the Pennsylvania Department of Transportation, and several private firms initiated a joint research project intended to introduce the RCC technology to high-speed, heavy-load highways.

CONCLUSION

RCC is an economical construction method appropriate for low-traffic, heavy-load pavements. Its overall engineering properties are similar to those of a conventional PCC (*5*). A quality control and quality assurance program must be implemented to ensure that strength, density, smoothness, and texture requirements are met. The spacing and location of saw-cut shrinkage and dilation joints must be carefully designed considering the location of utilities and any other discontinuity. In most cases, density is the key RCC quality control parameter. Density in the vicinity of construction joints tends to be lower than elsewhere, which frequently calls for additional compaction. If proper density is not achieved, spalling may soon occur. The number of cold joints may be minimized by keeping the hauling, paving, and compaction operations within the initial setting time. Proper curing practice is essential to avoid excessive shrinkage cracks and shallow microcracks, which lead to premature surface deterioration.

The conceptual distinction between the number of F–T cycles of the air moisture and the number of F–T cycles of the moisture inside the concrete is essential to evaluate the F–T durability of pavement materials properly. Figure 3 shows that the number of F–T cycles that moisture in the concrete pavement experiences is substantially smaller than that of the moisture in the air.

Research at CRREL has proven that the F–T durability of properly constructed RCC is good (*3*). CRREL is laboratory testing core and beam samples from the Fort Drum RCC pavement to verify their F–T durability. The test results were not available as of this writing, but the performance of the Fort Drum RCC pavement at the end of two winters and 1 year of service is satisfactory. No major distress has been observed.

The acceptance of RCC as a paving technology is growing rapidly, motivated by savings on the order of 30 percent and faster construction. In 1989, General Motors built 544,000 m^2 (approximately 135 acres) of RCC pavement at its new Saturn automobile plant in Tennessee. In 1990, the Massachusetts Port Authority (MASSPORT) plans to expand one of its large RCC pavements in Boston. Government and industry are working together to bring the advantages of RCC to our national highway system, making possible enormous savings in construction and maintenance costs.

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Longitudinal Wedge Joint Study

ROBERT F. BAKER, JACK R. CROTEAU, JOHN J. QUINN, AND EDGAR J. HELLRIEGEL

A 5-year study was undertaken to develop a technique for producing more durable longitudinal construction joints in bituminous pavements. The construction procedure evaluated involves forming the joint between adjoining lanes as two overlapping wedges. The wedge joint is formed by a steel plate attached to the paver screed. The effectiveness of the wedge joint was measured by the extent to which this procedure was able to eliminate a density gradient across the joint. Nuclear density testing was undertaken to determine the uniformity of the density across the joint and, hence, the nature of the density gradient. Density measurements were taken across the wedge joint and compared with the standard longitudinal center joint. These measurements indicated that the wedge joint had a more uniform density across the joint and a higher average density than the standard joint. The wedge joint eliminates the density gradient and, hence, lowers the potential for joint deterioration. By eliminating the vertical edge, the wedge joint eliminates the vertical dropoff and offers a safer condition for motorists making lane changes in construction areas.

The objective of this 5-year study was to develop the wedge joint technique for producing more durable longitudinal construction joints in bituminous pavements. The construction procedure involves forming the joint between adjoining lanes as two overlapping wedges. This wedge joint is formed by a steel plate attached to the paver screed, which produces a 3:1 sloped face at the edge of the first bituminous mat placed.

In placing bituminous concrete, paving the full width of the pavement in a single pass is often impossible. This problem particularly arises in the resurfacing and rehabilitation work that is now the primary focus of the New Jersey Department of Transportation capital program. As a practical necessity, then, most bituminous pavements contain longitudinal construction joints.

As is well known, these construction joints can be the weak link in the chain that eventually causes an otherwise sound pavement to deteriorate. The typical stages of distress of longitudinal joints include an initial separation, the ingress of water and incompressibles, and cracking and raveling.

No truly effective technique exists for repairing distressed longitudinal joints. Figure 1 shows a photograph of typical longitudinal joint distress on a major highway (I-295) in the vicinity of Trenton. As noted in the photograph, the department has undertaken some rather costly repair measures in an attempt to correct the severe joint distress. This repair strategy is to saw out the pavement on either side of the joint, excavate the bituminous material, and install a replacement inlay. Whether these expensive remedial measures will provide a long-term solution remains to be determined.

Bureau of Transportation Structures Research, Division of Research and Demonstration, New Jersey Department of Transportation, 1035 Parkway Ave., CN 600, Trenton, N.J. 08625.

One procedure for minimizing joint distress is to require that the two adjoining mats both be hot enough during construction to permit adequate compaction. In an attempt to avoid cold longitudinal construction joints, New Jersey's specifications place a 1,500-ft limit on the length of the bituminous mat that may be placed before bringing the paver back to place the subsequent lane. This method clearly has not eliminated cold joints and subsequent joint deterioration. Indeed, New Jersey's high traffic volumes often preclude enforcement of this limitation on resurfacing work.

Safety is also a consideration in constructing pavement joints. In the lane-at-a-time paving typically used on resurfacing projects, a height differential between the newly placed mat and the adjoining pavement is inevitable. This vertical stepoff can pose a hazard to traffic traveling through the construction area, especially if lane changes are required. A recent (unpublished) national survey of highway engineers including longitudinal joint construction indicated that 30 to 40 percent of the respondents were of the opinion that the stepoff was somewhat or extremely hazardous to compact cars and motorcycles.

States have used a variety of measures to deal with this perceived safety problem, including prohibiting stepoffs, limiting their height, using special signing, and limiting the time within which the adjacent lane must be paved. New Jersey's practice is to minimize the use of lane-to-lane changes in the construction work zone and to require that adjoining lanes be paved the same day. Depending on project conditions, however, exceptions to this policy do occur (e.g., on maintenance resurfacing projects).

Beginning in 1982, the New Jersey Department of Transportation began experimenting with improved longitudinal joint construction techniques. On the basis of a literature review, the most promising technique was the use of a wedge joint. The Arizona Department of Transportation was one of the first agencies to form joints using overlapping wedges. In the Arizona work, the joint was formed by a sloping shoe attached to the paving machine. This shoe produced a wedge that tapered from 2 in. to zero over a 1-ft length. After compacting the face of the wedge with a pneumatic-tired roller, the adjoining lane was paved to form the finished joint.

In adapting this wedge joint for trials in New Jersey, two modifications were made. The first was to use a steeper sloping face (6:1 versus 3:1) to reduce the potential for raveling. The second was to supplement the use of the wedge joint with infrared heating. Supplemental heating of longitudinal joints has been used with at least some success with conventional vertical butt joints, for example, by Foster et al. (*1*). The theory was that such supplemental heating could be used with particular effectiveness with the wedge joint by softening

FIGURE 1 Typical longitudinal joint distress and repair.

the wedge immediately before placement of the second (overlapping) mat, thereby providing a denser, more homogeneous joint.

RESEARCH METHODOLOGY

The primary measure used to gauge the effectiveness of the wedge technique was the extent to which this procedure was able to eliminate a density gradient across the joint. The significance of joint density gradients was pointed out by Foster et al. (*1*). In 1964, Foster (*1*) summarized the results of density tests on cores taken from variously constructed longitudinal joints. One of the primary findings of that study was that in poor-performing (cold) joints, a low-density zone occurs at the joint in the lane first paved (i.e., the unconfined joint edge) and a high-density zone in the adjoining lane (the confined edge). Foster (*1*) concluded that eliminating this differential is the basic problem in constructing a durable longitudinal joint. He theorized that this differential density is a major factor permitting the initial opening of the joint and the subsequent inevitable process of distress. However, he compared only various types of conventional butt joints.

By virtue of geometry alone, the wedge joint technique was expected to provide a more monolithic, better-performing joint. By eliminating the vertical shear plane in the conventional butt joint, the finished joint should be more resistant to opening as a result of traffic or temperature changes. In order to test this assumption, a program of nuclear gauge testing was undertaken to determine the uniformity of density across the joint and, hence, the nature of any density gradient. As shown in Figure 2, at a given location, those density measurements were taken directly over the joint and 1 ft on either side.

This program of density testing was performed on five resurfacing projects. On three of those projects, a control section consisting of the conventional butt joint was incorporated to provide a specific basis of comparison with the wedge joint. No formal testing was performed to determine the relative safety advantage of the wedge joint as compared to the conventional vertical joint. That the wedge joint does, in fact, possess such an advantage seemed intuitively obvious.

JOINT CONSTRUCTION TECHNIQUES AND EQUIPMENT

Figure 3 shows the sequence of operations involved in constructing a typical New Jersey butt-type joint. After placing the first bituminous mat, the subsequent lane overlaps the first lane by 2 to 4 in. (Figure 3a). The overlapped material is pushed back with a lute, creating a bump (Figure 3b). Rolling in the first lane overlaps the subsequent lane by about 6 in. (Figure 3c). In the subsequent lane, the roller pinches the material into the joint (Figure 3d).

Wedge Joint Construction

As shown in Figure 4, the longitudinal wedge joint consists of two overlapping wedges. The 3:1 inclined face of the joint is formed in the first bituminous mat placed by a sloping steel plate (Figure 5), which is attached to the inside corner of the paver screed extension. A typical wedge plate installation is shown in Figure 6. The plate is mounted about ⅜ to ½ in. above the existing pavement. The specific dimensions of the wedge joint plate and the attachment details vary with paver models.

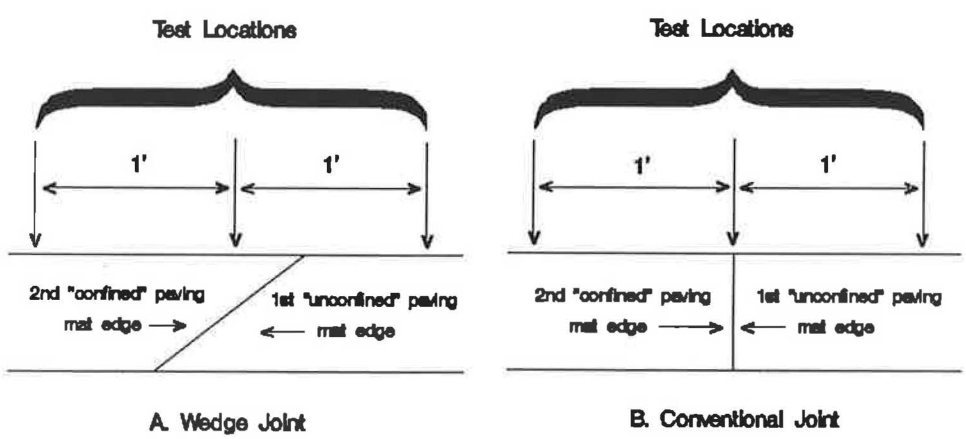

FIGURE 2 Density test layout.

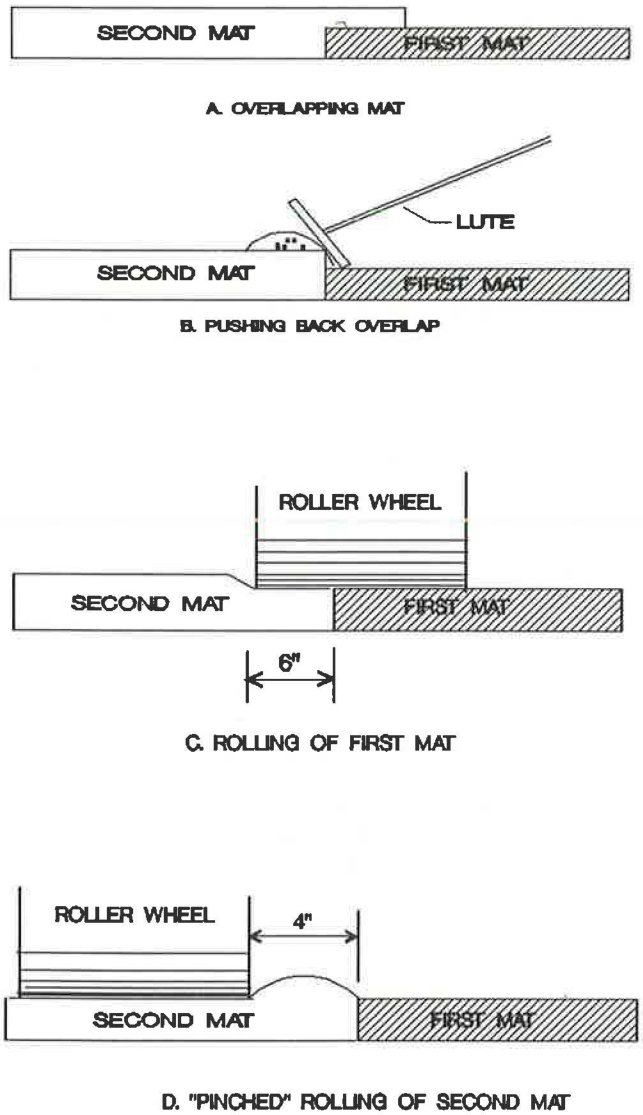

FIGURE 3 Construction of typical butt joint.

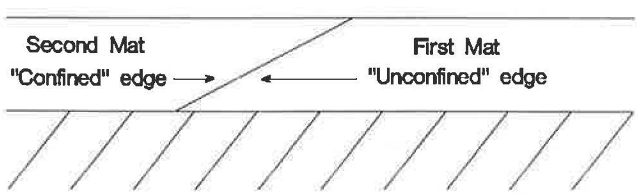

FIGURE 4 Cross section of wedge joint.

After the initial mat is placed using the wedge-forming plate, the mat is rolled to the top of the unconfined edge. In this operation, the roller should not extend more than 2 in. past the top of the unconfined edge (Figure 7). The inclined face of the wedge edge should not be compacted. Leaving the unconfined edge in an uncompacted state permits a more homogeneous bond with the second mat, thereby providing a denser finished joint. The appearances of the unconfined wedge edge and the rolling operation are shown in Figures 8 and 9, respectively.

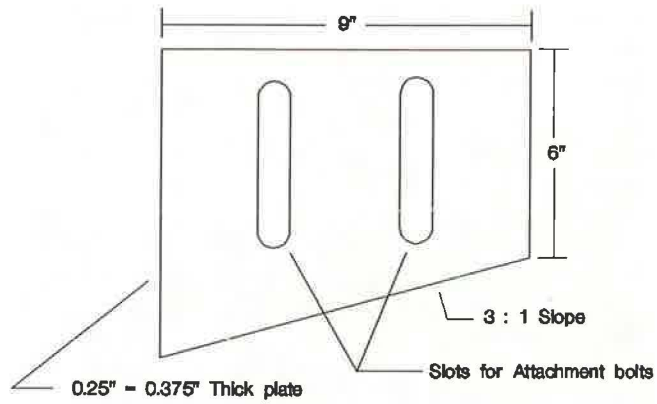

FIGURE 5 Typical wedge plate design.

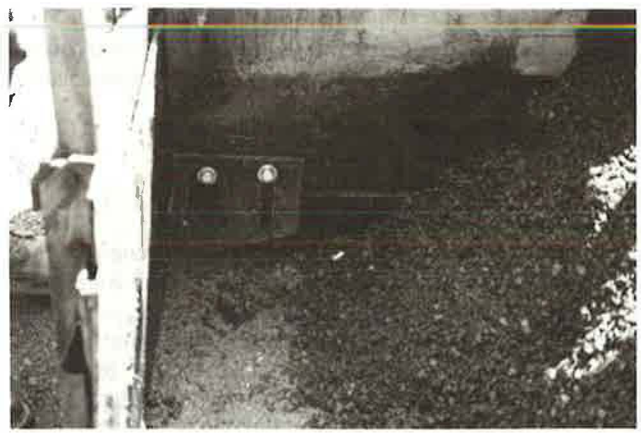

FIGURE 6 Typical attachment of wedge plate to paver.

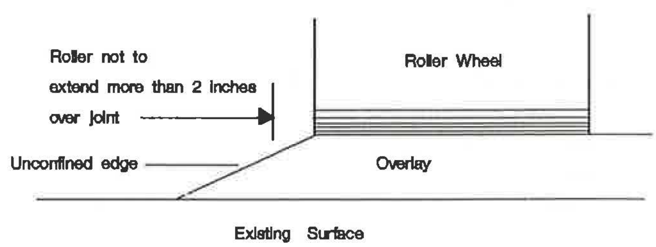

FIGURE 7 Compaction procedure at unconfined wedge edge.

When the second paving mat is placed, an infrared heater attached to the side of the paver heats the unconfined joint edge. This preheating and softening of the joint edge and the adjoining mat is designed to provide a more monolithic joint and to increase the achievable density across the joint.

The second paving mat overlaps the first by 2 to 3 in. (Figure 10a). This overlapped material is pushed back with a lute 3 to 4 ft from the edge of the second mat (Figure 10b). No special rolling is necessary for the completed wedge joint.

Infrared Heater

The infrared heater is designed to heat the surface of the unconfined wedge edge to a depth of about 1½ in. The heater

FIGURE 8 Initial unconfined wedge edge.

FIGURE 9 Rolling the mat at the wedge edge.

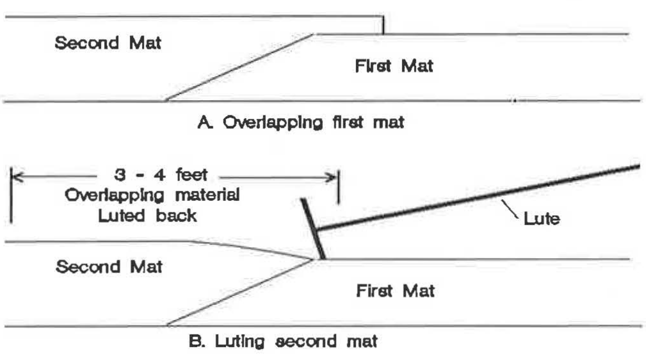

FIGURE 10 Placing the second mat.

is mounted on the paver in front of the screed with a height of about 2 in. above the previously placed mat. The typical heater and mounting are shown in Figure 11. The heater consists of four ribbon burners mounted in a rectangular stainless steel box (15 in. wide × 76 in. long × 4 in. deep). The heater is suspended from the paver with a pair of support pipes. The heater is fueled by an LP vapor withdrawal cylinder. The manufacturer furnishes the heater ready to mount

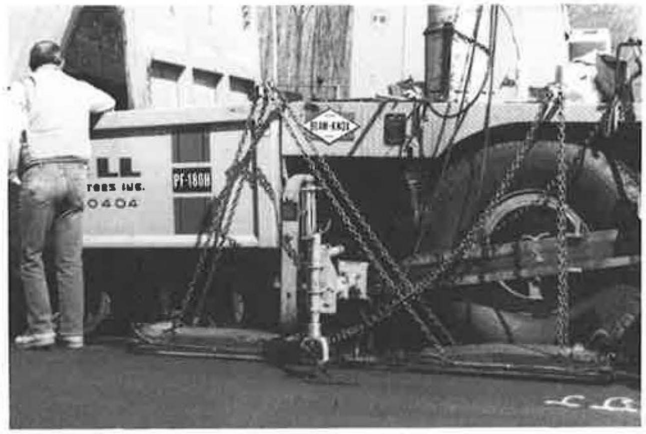

FIGURE 11 Typical heater mounting.

with all necessary hoses, regulators, and electrical connections.

RESULTS AND ANALYSIS

The testing program consisted of making density measurements across the joints on each of three projects in which both the wedge joint and conventional joint were used (the controlled experiments) and testing on three projects in which only the wedge was used (the uncontrolled experiments). On each project, from 5 to 17 test locations were selected for density measurements. The nuclear testing was done with a Troxler Model 3411 B gauge.

Controlled Experiments

Routes US-40 and US-322, Section 2D, Atlantic County

In July 1984, the wedge joint technique was used in constructing all of the longitudinal joints in the 1½-in.-thick top course on this resurfacing project, except for a 300-ft control section. In the control section, the joint between the inside and outside eastbound lanes was constructed using the conventional technique.

Average density data for the wedge and butt joints are shown in Figure 12 and presented in Table 1. Examination of the plotted data indicates that the wedge joint technique was successful in achieving the goal of a generally higher, more uniform density. A statistical analysis of the differences in means confirms that density across the wedge joint does not differ significantly on this project. The butt joint density data display the typical density gradient pattern reported in the literature. The density measurements in the first paving mat and those directly over the joint are significantly lower than those for the second paving mat.

Route NJ-10, Sections 2G and 3G, Morris Plains

In July 1986, the wedge joint technique was used to construct the longitudinal joint in the 2-in.-thick top course on this

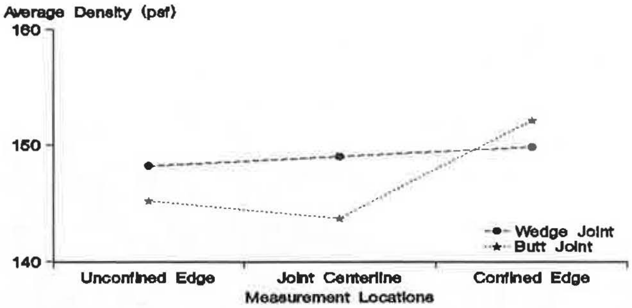

FIGURE 12 Comparative joint density measurements, Routes 40 and 322, Section 2D.

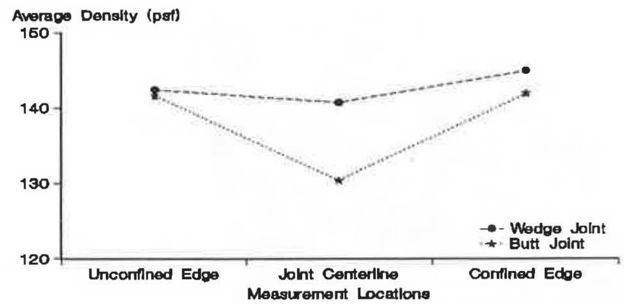

FIGURE 13 Comparative joint density measurements, Route 10, Sections 2G and 3G.

resurfacing project, except for an approximately 500-ft test section in which the conventional butt joint was installed for comparison purposes. The control section was located in the eastbound roadway between the railroad bridge and the Route NJ-53 bridge.

The average density results for the wedge and butt joints are shown in Figure 13 and presented in Table 2. The plotted density data for this project indicate that the wedge joint provided greater, more uniform density. The average density at the confined joint edge was statistically higher than at the other two transverse test locations. However, whether this statistically significant difference (amounting to approximately 4 lb/ft^3) is an important difference in terms of joint performance is doubtful. Examination of Figure 13 clearly indicates a marked density gradient in the conventional butt joint, with the density directly over the joint being signifi-

TABLE 1 NUCLEAR DENSITY MEASUREMENTS, ROUTES 40 AND 322, SECTION 2D

Wedge Joint

Location	Unconfined	Over Joint	Confined
1	152.1	148.8	150.5
2	150.0	147.2	147.1
3	150.1	147.2	148.0
4	149.2	150.1	149.0
5	148.3	150.6	151.2
6	145.7	150.2	153.1
7	148.5	148.4	150.2
8	146.7	148.8	149.0
9	146.0	147.5	149.9
10	145.3	151.0	149.8
Average	148.2	149.0	149.8

Butt Joint(Standard)

Location	Unconfined	Over Joint	Confined
1	148.3	141.1	151.3
2	150.6	146.2	152.5
3	149.6	148.7	150.9
4	147.6	145.3	149.3
5	144.8	146.7	152.6
6	138.5	145.5	153.4
7	144.4	142.0	151.7
8	146.5	141.9	149.1
9	138.1	136.7	155.2
10	143.1	142.6	154.8
Average	145.2	143.7	152.1

TABLE 2 NUCLEAR DENSITY MEASUREMENTS, ROUTE 10, SECTIONS 2G AND 3G

Wedge Joint

Location	Unconfined	Over Joint	Confined
1	136.6	139.4	141.8
2	144.6	138.8	135.9
3	133.1	137.6	144.5
4	135.6	138.6	145.4
5	141.6	139.9	141.1
6	144.7	141.1	141.2
7	146.7	142.7	146.1
8	142.4	139.3	149.2
9	143.4	142.4	145.1
10	144.5	142.6	144.5
11	145.4	134.9	149.8
12	144.1	145.6	146.9
13	143.4	137.2	145.0
14	144.2	142.6	145.8
15	143.4	142.5	149.9
16	143.4	144.2	146.0
17	144.5	142.6	145.8
Average	142.4	140.7	144.9

Butt Joint(Standard)

Location	Unconfined	Over Joint	Confined
1	138.8	125.8	140.1
2	141.6	128.2	142.6
3	143.4	130.0	141.1
4	142.7	128.3	141.2
5	143.1	137.2	144.9
6	143.5	130.0	141.8
7	137.6	122.2	141.6
8	140.6	133.4	139.1
9	143.1	137.4	143.2
10	140.0	129.0	141.5
11	141.6	121.6	141.5
12	140.3	129.9	140.5
13	142.0	137.0	143.2
14	142.0	137.4	143.8
15	143.5	127.9	142.6
Average	141.6	130.4	141.9

cantly lower than that attained in either of the adjoining paving mats.

Route I-78, Sections 6F and 7F, Warren County

In May 1986, the wedge joint technique was used in constructing the joints in the top course and two lifts of binder from Station 369+00 (Bloomsbury Road) to Station 432+00 (Asbury Road) except for a 300-ft butt joint control section. The control section was located between Stations 387+00 and 390+00 in the outside and center westbound lanes. The top course was 2 in. thick, and each of the two binder courses was 3 in. thick and variable.

The average density results for the wedge and butt joints are shown in Figure 14 and presented in Table 3. The plotted data indicates the wedge joint results on this project were similar to those observed on Route NJ-10. The wedge joint displayed approximately equal densities in the first paving mat and over the centerline of the joint but somewhat higher densities in the second (confined) paving mat. Here again, the butt joint displayed a pronounced density gradient. The density measurements were the lowest directly over the joint.

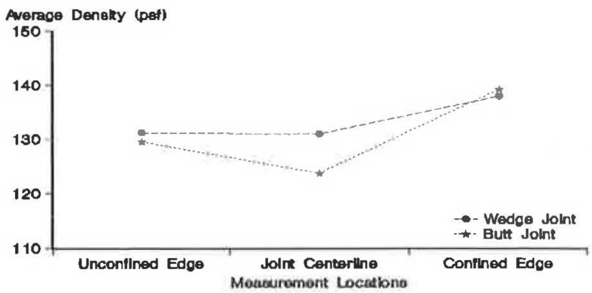

FIGURE 14 Comparative joint density measurements, I-78, Sections 6F and 7F.

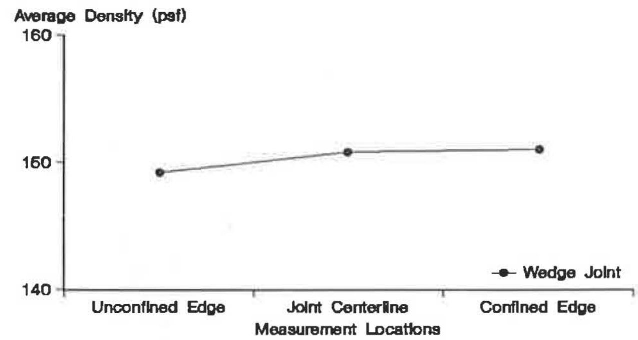

FIGURE 15 Joint density measurements, I-80, Section 4BB.

Uncontrolled Experiments

Route I-80, Section 4BB, Paterson

In July 1985, the wedge joint technique was used in constructing all of the joints in the 2-in. top course in this resurfacing project. No butt joint control section was provided. The average density results are shown in Figure 15 and presented in Table 4. On this project, the wedge technique succeeded in eliminating the joint density gradient. A statistical analysis indicates that no significant difference in density across the finished wedge joint.

Route US-206, Sections 6B, 5A, 4B, and 3B, Red Lion

In May 1984, the wedge joint was used in constructing all the longitudinal joints in the 2-in. top course of this resurfacing project. Supplemental (infrared) heating was not used in constructing the wedge joints on this project. The average density results are shown in Figure 16 and presented in Table 5. As shown in Figure 16, the finished wedge joint displayed no statistically significant density gradient across the joint.

TABLE 3 NUCLEAR DENSITY MEASUREMENTS, I-78, SECTIONS 6F AND 7F

Wedge Joint

Location	Unconfined	Over Joint	Confined
1	130.3	134.7	140.6
2	130.7	133.9	138.7
3	132.3	129.9	140.3
4	134.3	130.3	137.1
5	136.5	136.7	139.9
6	132.8	130.7	137.2
7	127.5	128.9	134.5
8	130.5	124.5	136.6
9	130.2	132.8	137.2
10	127.3	128.9	137.5
Average	131.2	131.0	138.0

Butt Joint(Standard)

Location	Unconfined	Over Joint	Confined
1	131.4	129.1	136.2
2	131.4	119.7	139.3
3	120.4	124.4	142.2
4	126.4	121.0	140.3
5	133.9	123.1	141.3
6	131.3	123.5	139.7
7	132.2	125.7	136.4
Average	129.6	123.8	139.3

TABLE 4 NUCLEAR DENSITY MEASUREMENTS, I-80, SECTION 4BB

Wedge Joint

Location	Unconfined	Over Joint	Confined
1	144.0	148.9	148.8
2	149.2	142.7	149.8
3	150.0	149.7	151.2
4	147.7	154.7	155.4
5	151.1	149.5	139.2
6	147.9	154.1	156.4
7	154.6	155.8	156.1
Average	149.2	150.8	151.0

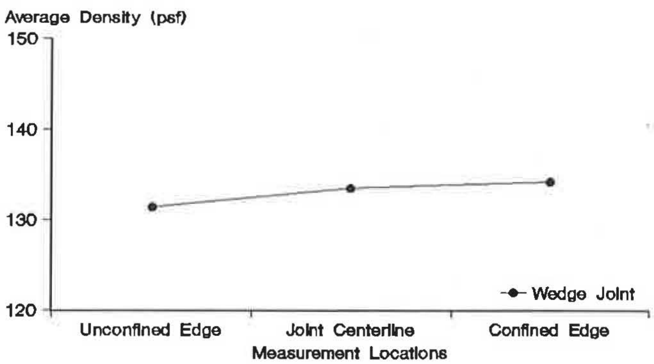

FIGURE 16 Joint density measurements, Route 206, Sections 3B, 4B, 5A, and 6B.

Costs

The costs associated with the use of the wedge joint technique have been estimated on the basis of information supplied by New Jersey contractors, resident engineers, and an infrared heater supplier. The purchase price of the infrared heater reportedly ranges from $1,000 to $4,000, depending on the size of the heater and number of burners (BTU rating). The total cost of acquiring and installing the wedge joint plate, the heater mounting, and propane tank is approximately $650. The daily operating cost of the heater is insignificant. For example, the Route NJ-3, Section 2J, project required only about $10 of propane (30 lb) per day for each 1,000 tons of bituminous concrete placed in a 2-in.-thick overlay.

CONCLUSIONS

The results of the extensive program of field density testing undertaken in this study indicate that the wedge joint technique produces higher, more uniform density than the conventional butt joint technique. These observed improvements in density, combined with the elimination of the vertical shear plane in the conventional butt joint, suggest that the wedge joint procedure will provide a finished joint that is more resistant to opening under the effects of traffic and weathering. Furthermore, after 5 years of testing and analysis, the wedge joint construction on five projects shows none of the deteri-

TABLE 5 NUCLEAR DENSITY MEASUREMENTS, ROUTE 206, SECTIONS 3B, 4B, 5A, AND 6B

Wedge Joint

Location	Unconfined	Over Joint	Confined
1	127.3	133.7	133.1
2	134.1	133.3	135.2
3	133.1	125.8	134.3
4	130.3	132.7	134.6
5	127.8	131.6	130.9
6	134.3	137.5	133.8
7	133.9	135.4	134.2
8	132.1	133.7	134.3
9	129.8	137.6	137.2
Average	131.4	133.5	134.2

oration or raveling that had been observed in the standard joint.

In addition to improved performance, the wedge joint offers two other advantages. First, by eliminating the vertical edge, the wedge joint is safer for motorists making lane changes in the resurfacing construction area. Second, use of the wedge joint with supplemental heating eliminates the need to pull back the paver to avoid cold joints, thereby providing the contractor with greater flexibility and production capability. This elimination of the pullback requirement also has the potential for improving the riding quality of the finished pavement by reducing the number of transverse joints in the surface course.

Although this study of the effectiveness of the wedge joint procedure was limited to resurfacing projects, certain of the observed results are equally applicable—and the benefits equally desirable—on new construction. The wedge joint with supplemental heating is therefore recommended for use in constructing surface mixes (top and bottom layer) both on new bituminous pavement construction and resurfacings.

One point still unsettled, however, is whether supplemental heating should be required for bituminous mixtures other than surfacing mixes. Construction personnel have expressed the opinion that it may not be necessary, because base course joints have historically not been a factor in pavement distress. They agree that the safety feature of the wedge joint—elimination of the edge dropoff—is important for all mixes on resurfacing work. Under these circumstances, the conclusion is that the wedge joint should also be used on base course construction on both new and overlay projects, except that the use of supplemental heating should be a contractor option. The merits of this approach should be verified by research as a part of the normal implementation process.

RECOMMENDATIONS

1. Specifications. It is recommended that the Department adopt the wedge joint technique as the standard method for constructing longitudinal joints in all bituminous paving.
2. Further Research. It is recommended that the merits of eliminating the use of supplemental heating in base course wedge joint construction be evaluated by research as part of the implementation process.

REFERENCE

1. C. R. Foster, S. B. Hudson, and R. S. Nelson. Constructing Longitudinal Joints in Hot-Mix Asphalt Pavements. *Highway Research Record 51*, HRB, National Research Council, Washington, D.C., 1964.

The opinions, findings, and conclusions expressed in this paper are those of the authors and not necessarily those of the New Jersey Department of Transportation or the FHWA. This report does not constitute a standard, specification, or regulation.

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Density of Asphalt Concrete—How Much Is Needed?

E. R. Brown

Density is one of the most important parameters in construction of asphalt mixtures. A mixture that is properly designed and compacted will contain the optimum amount of air voids. Because density of an asphalt mixture varies throughout its life, the voids must be low enough initially to prevent permeability of air and water and high enough after a few years of traffic to prevent rutting caused by plastic flow. There are three primary methods of specifying density: (a) percent of control strip, (b) percent of laboratory density, and (c) percent of theoretical maximum density. If used correctly, all three methods can result in satisfactory compaction. The initial in-place air voids must be below approximately 8 percent and are determined by comparing bulk density and theoretical maximum density (TMD). The final in-place air voids, which must be above approximately 3 percent, are estimated by comparing the bulk density of laboratory-compacted samples and the TMD. The two methods that have been used to measure bulk density of asphalt mixture are physical measurements of cores and use of the nuclear gauge. The nuclear gauge is fast and nondestructive but is not as accurate as the core method.

The amount of voids in the asphalt mixture is probably the factor that most affects performance throughout the life of an asphalt pavement. Voids are primarily controlled by asphalt content, compactive effort during construction, and additional compaction under traffic. The density requirements and the methods of measuring density vary considerably from state to state. Some states construct a control test strip, measure the density on the strip, and use that density as the target density for the project. Other states compact samples in the laboratory during mix design and construction and use that density as the target density. Finally, other states measure the theoretical maximum density (TMD) (ASTM D2041) and use some percentage of that density as the target density. All of these techniques have been used successfully to build well-performing pavements, but they have also been misused, resulting in poor performance. Which method should be used? How much density should be specified and obtained during construction to ensure good performance?

A problem with density is the method of measurement. The two primary methods that have been used include measurement of bulk density of cores taken from the in-place pavement, and use of a nuclear gauge to measure the in-place density. The nuclear gauge method is not considered as accurate as measuring the density of cores. Many states use the nuclear gauge for developing rolling patterns but specify that cores be taken and measured for acceptance or rejection of the in-place mix. However, several states use the nuclear gauge for acceptance testing of the asphalt mixture.

The existing methods of specifying density of asphalt mixtures are compared in the following paragraphs. The relationship of each method to construction and performance is discussed, along with the various methods of measuring density during construction.

National Center for Asphalt Technology, Auburn University, 211 Ramsey Hall, Auburn, Ala. 36849.

DESIRED ASPHALT DENSITY

The voids in an asphalt mixture are directly related to density; thus, density must be closely controlled to ensure that the voids stay within an acceptable range. Previous work has shown that the initial in-place voids should be no more than approximately 8 percent and should not fall below approximately 3 percent during the life of the pavement. High voids lead to permeability of water and air, resulting in water damage, oxidation, raveling, and cracking. Low voids lead to rutting and shoving of the asphalt mixture.

In a study for the state of Arkansas, Ford (*1*) showed that asphalt mixtures should be designed and constructed so that the in-place air voids stay above 2.5 percent. He demonstrated that, as long as the voids are above that amount, the expected rut depth will be no greater than 10/32 in. (see Figure 1). Ford's work was based on tests conducted on asphalt samples that were obtained from in-place pavements. The rut depth reported was from actual measurements on these pavements.

Brown and Cross (*2*), in a study of rutting of asphalt pavements, showed that significant rutting was likely to occur once the in-place voids reached approximately 3 percent (see Figure 2). When a suitable aggregate was used and the voids stayed above 3 percent, rutting was normally not a problem. Some of the projects evaluated showed significant rutting when the in-place voids were well above 3 percent. It was speculated that rutting began after the voids had decreased to an unacceptable level. Once rutting began, the integrity of the mix was lost and the voids increased. For these mixes, it was generally found that recompacting the mixtures in the laboratory with standard compactive effort produced low voids, which helped explain why the rutting occurred.

In a study of asphalt mixtures in Canada, Huber and Heiman (*3*) considered a number of causes of rutting. It was determined that one of the primary causes was low voids (below 3 percent) in the asphalt mixtures.

Zube (*4*) showed that asphalt mixtures become permeable to water at approximately 8 percent air voids (see Figure 3). As long as the voids were below 8 percent in the 10 projects studied, permeability was not a problem. However, the permeability increased quickly as the void level increased above 8 percent.

In a study of segregated mixes, Brown et al. (*5*) showed that the asphalt mixes were impermeable to water as long as the air void content was below approximately 8 percent (see

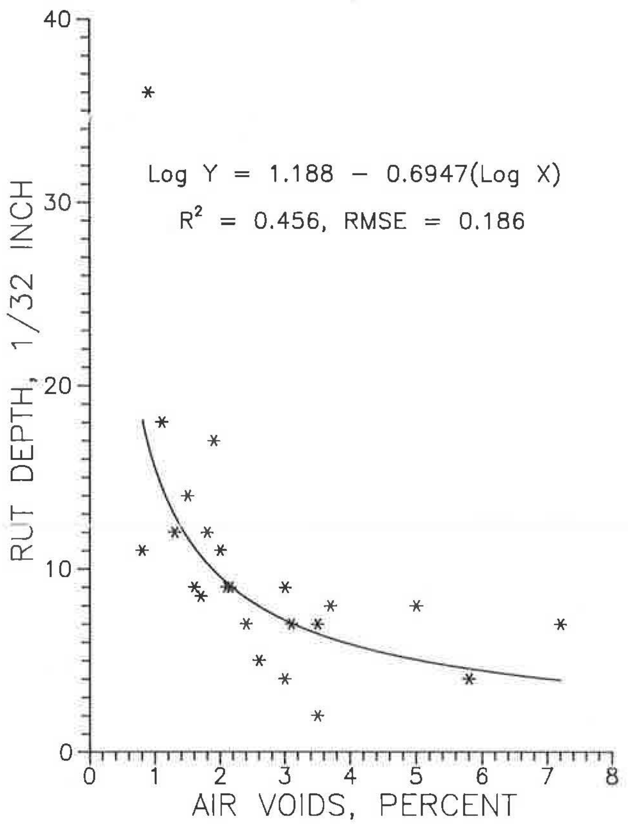

FIGURE 1 Relationship between air voids and rut depth in Arkansas (*1*).

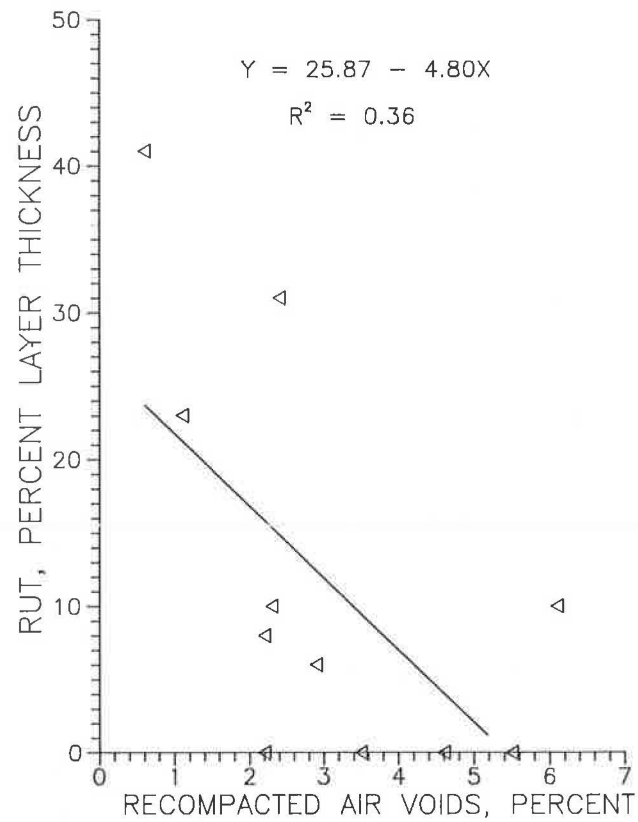

FIGURE 2 Relationship between air voids and rut depth in rutting study by National Center for Asphalt Technology (*2*).

Figure 4). The permeability increased rapidly as the void content increased above 8 percent.

Santucci et al. (*6*) showed that the retained penetration of asphalt cement is affected by the air voids in the pavement (see Figure 5). The loss in asphalt penetration was greatly increased for air voids significantly greater than 8 percent. They concluded that asphalt mixes must be constructed with low air voids (below 8 percent) to prevent rapid oxidation that leads to cracking and raveling of the asphalt mixture.

From these studies, it is apparent that asphalt mixes must be constructed with an initial air void content below approximately 8 percent and that the final air void content after traffic should remain above approximately 3 percent. The initial air void content is determined by comparing the in-place bulk density and the TMD for the mix being evaluated. The final in-place air voids are estimated on the basis of the mix design and field quality control testing. The voids obtained during the mix design and laboratory compaction of samples during construction are used to estimate the in-place voids after traffic. A Marshall hammer was selected to provide voids in laboratory-compacted samples equal to the measured voids after traffic (*7*).

SPECIFICATION OF ASPHALT DENSITY

Percentage of Laboratory Density

One method that has been used to specify density is to require that the in-place material be compacted to some percentage of the laboratory density. The standard laboratory density is specified as 50 to 75 blows with a Marshall hammer. In recent years most states have required 75 blows for high-volume roads. Some specifications require at least 95 percent of laboratory density, whereas others require at least 98 percent. Some specifications do not allow mixes to be compacted to a density greater than 100 percent of laboratory density. When mixes designed to have 4 percent voids are compacted to a density greater than 100 percent, premature rutting is likely to occur.

Several items are important for this method of specification to work effectively. Samples of the mix produced during construction have to be compacted in the laboratory to establish a reference density and to determine the air voids in the mix at that density. If the air voids are not satisfactory in the laboratory-compacted samples during construction, the mix must be adjusted so that acceptable air voids are obtained. Most often, the adjustment simply involves a modification in the asphalt content. The density produced during the mix design should not be used as the reference density because the laboratory properties will be somewhat different than test results on plant-produced materials. Aggregates sometimes break down during mix production, creating an increase in dust and thus altering the properties of the compacted asphalt mixture.

The density produced with a manual hammer has been shown to correlate with density in the field after traffic (*7*). Hence, any other type of compaction (mechanical or otherwise) must be calibrated to produce a density equal to that obtained with the hand hammer or, better yet, a density equal

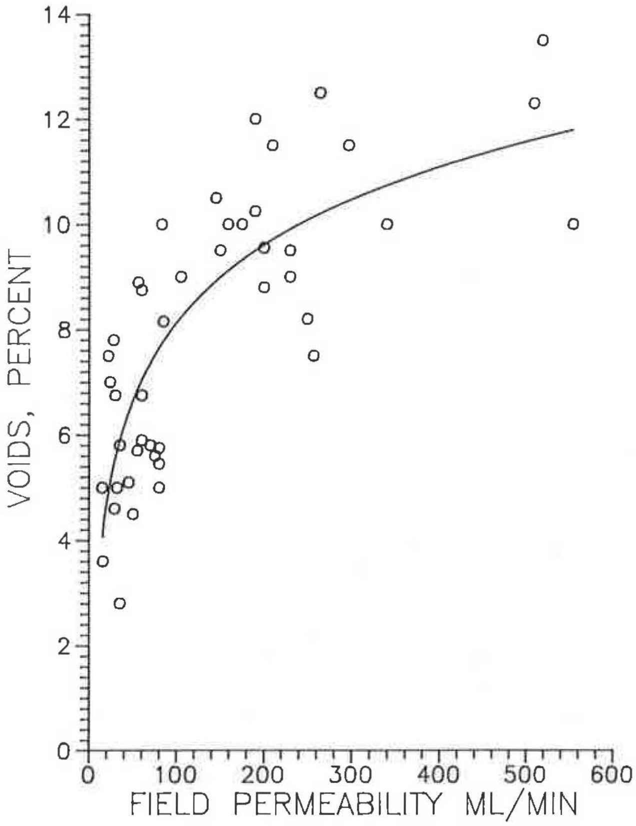

FIGURE 3 Relationship between air voids and permeability in California study (*4*).

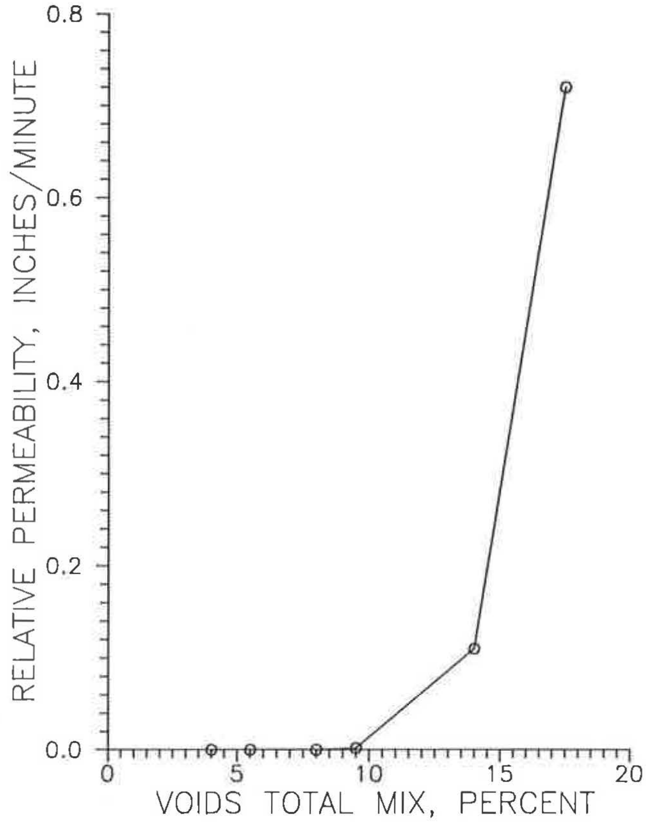

FIGURE 4 Relationship between air voids and permeability in Georgia study (*5*).

to that obtained in the field after traffic. The procedures specified in ASTM D1559 and AASHTO T245 for the Marshall test require that the manual hammer be used or that the method used be calibrated with the manual hammer. Density data from eight construction projects are presented in Table 1. These data show that the in-place density (80th percentile) after traffic is 2.2 lb/ft^3 higher than that obtained in the mix design. There are two likely reasons for this higher density after traffic. First, the mix probably changed somewhat during production to increase the laboratory density. Second, the laboratory compaction effort was probably insufficient and thus should be increased to be more representative of traffic. As shown in the table, the density of the mixes recompacted with the manual hammer compares closely to the in-place density. This finding emphasizes both the need to compact samples in the laboratory during construction to verify voids in the mixture and the need to use correct laboratory compactive effort.

Suppose a mix is designed to provide 4 percent voids and is specified to be compacted to at least 95 percent of laboratory density. This specification will result in up to 9 percent voids immediately after compaction and should result in approximately 4 percent voids after several years of traffic. The initial voids (9 percent) may be a little high with this specification; however, the final voids (4 percent) should be acceptable. The high initial voids may result in increased oxidation, causing more cracking and raveling if not subjected to significant traffic to provide further compaction. If this mix is subjected to a high volume of traffic, a small rut (5 percent of layer thickness, or ⅒ in. for a 2-in. layer) will result after additional channelized compaction under traffic increases the density from 95 to 100 percent of laboratory density.

If a mix is designed to have 4 percent air voids and is compacted to a density greater than 100 percent, immediate failure caused by rutting is likely. If the laboratory compactive effort is satisfactory, it is not practical for the mix to be compacted to a density greater than 100 percent. Hence, any project that continually approaches or exceeds 100 percent of laboratory density is likely the result of low laboratory density, not excessive compaction in the field.

This method of specifying compaction will result in good performance of properly designed mixes if (a) laboratory samples are compacted during construction to establish reference density, (b) correct laboratory compaction techniques are used, and (c) a minimum compaction requirement is set to ensure that in-place air voids after compaction do not exceed approximately 8 percent.

Percentage of Theoretical Maximum Density

A second method often used to specify compaction requires that the asphalt mixture be compacted to some minimum percentage of the TMD. This procedure is a direct method of specifying maximum in-place air voids and an indirect method of controlling compaction. It involves taking a sample of the asphalt mixture during construction and conducting tests to measure the TMD (ASTM D2041). The bulk density of the asphalt mixture is measured after compaction and compared to the TMD, providing a direct measurement of in-place voids.

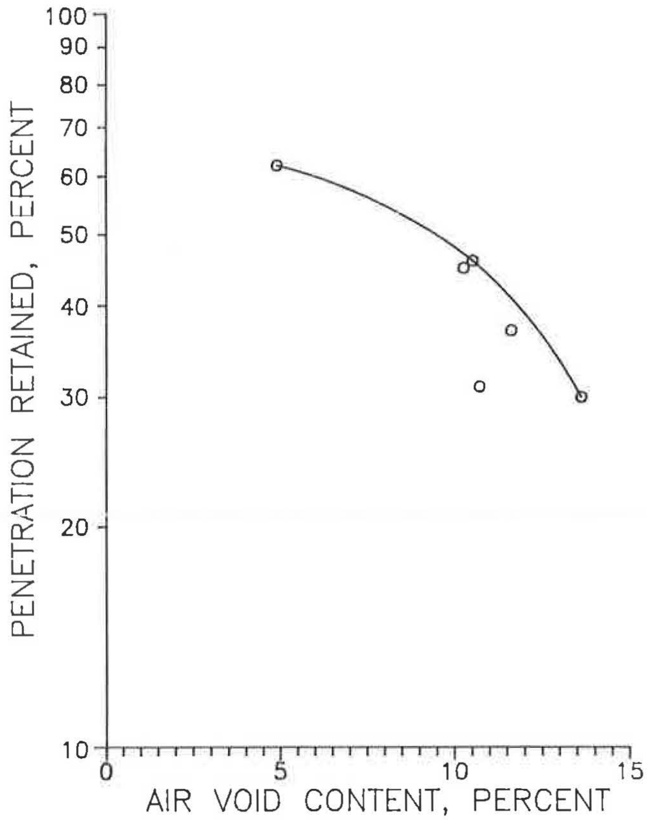

FIGURE 5 Relationship between retained penetration and air voids (*6*).

For instance, a mixture compacted to 93 percent of the TMD will have 7 percent air voids.

This type of compaction specification requires that the TMD, which is the reference density, be measured routinely during construction. The TMD measured during mix design should not be used as a reference for the mix being produced at an asphalt plant. As stated before, the materials change when heated and mixed at an asphalt plant, hence the TMD must be measured on these plant-produced materials.

Some states do not compact samples of asphalt mixture in the laboratory during construction. Many bituminous engineers do not believe that laboratory compaction of samples is necessary because the relative density is now the TMD and the time normally spent on compacting and testing laboratory samples can be used to conduct other tests. To control the construction process adequately, samples must be taken during construction and compacted in the laboratory. The voids in the laboratory-compacted samples must be measured and evaluated to determine the final expected in-place voids. It is not worthwhile to compact an asphalt mixture to 7 or 8 percent air voids initially if the voids are going to be reduced to 1 or 2 percent after one summer of traffic. The only way to estimate the final in-place voids (one of the most critical properties of an asphalt mixture) is to compact samples in the laboratory using the specified technique (manual or equivalent) and to measure the voids. If the voids are not acceptable, the mix (usually asphalt content) must be modified to produce acceptable voids.

This type of density specification is often misused. On many projects, so much emphasis has been placed on the initial in-place voids after compaction that the asphalt content has been arbitrarily increased to reduce the initial in-place voids to an acceptable range. This increase in asphalt content is often done when paving in cold weather or at other times when compaction is difficult. The increase in asphalt content will lower the air voids in laboratory-compacted mixes to an undesirable level and will likely result in rutting when subjected to a significant amount of traffic. If voids are high during construction, more compactive effort, improved roller patterns, or a modified mix design should be used to increase density. An increase in asphalt reduces the TMD and typically increases the actual density, which can significantly decrease the voids in the mix after being exposed to traffic.

This method of specifying density does encourage higher asphalt content and higher filler content; however, it can be correctly used if properly monitored. Laboratory compaction

TABLE 1 COMPARISON OF JOB MIX FORMULA DENSITY, IN-PLACE DENSITY, AND RECOMPACTED DENSITY

Project	JMF Density (pcf)	In-Place Density (80 percentile) (pcf)	Recompacted Density (75-blow Hand Hammer) (pcf)
1	143.1	149.9	151.1
2	143.7	145.6	147.4
3	145.5	143.9	143.3
4	144.4	147.1	147.3
5	145.8	147.7	148.9
6	146.6	146.0	148.7
7	146.6	148.9	151.0
8	147.3	151.4	151.0
Average	145.4	147.6	148.3

tests must be conducted during construction to ensure that the voids are maintained within an acceptable range. The TMD must be measured on the actual material being placed to ensure an accurate measurement. Additional asphalt must never be added for the sole purpose of reducing in-place voids. If the in-place voids are too high and the mixture has been properly designed, more compactive effort must be exerted to decrease them. In particular, more asphalt should not be added to decrease voids when paving in cold weather. Again, more compactive effort must be applied to the asphalt mix.

Percentage of Control Strip Density

A third method used to specify density is to compare the bulk density of the in-place asphalt mixture to that of a previously constructed control strip. The control strip is constructed using standard compaction techniques. Most specifications require that it be compacted to some minimum percentage of the standard laboratory density or the TMD. If the specifications do not require a minimum density, the inspector must closely evaluate the contractor's compaction equipment and rolling procedures to ensure that reasonable compactive effort is being applied to the asphalt mix. Any significant changes in the mix during construction should require that a new test strip be constructed and evaluated.

This method of density control is probably the least desirable of the three methods discussed. Although it does allow the compactibility of a mixture to be evaluated, it is difficult for an inspector to know when a contractor has applied a reasonable compactive effort to the control strip. Many items affect density, and a change in any of them may alter the results obtained from a control strip. Some of the items that affect density include gradation (especially for content of particles passing No. 200 sieve), asphalt content, moisture content, mix temperature, air temperature, layer thickness, roller weight, roller pattern, and roller speed.

As stated earlier, a minimum density is normally required for the control strip. This requirement ensures that the contractor does apply some minimal effort during compaction. The specification requires a minimum density in the control strip and then a minimum percentage of the control strip density in the remaining work. This specification could be made simpler by requiring the compacted mix to meet some percentage of the laboratory density or TMD. For example, assume a specification requires that a control strip have a density of at least 94 percent of the TMD and that all asphalt mix placed after the control strip have a density at least 98 percent of that of the control strip. This specification could be simplified by requiring that the mixture be compacted in place to a minimum density of 92 percent of the TMD. These two examples of specifying density result in similar compaction requirements.

The control strip method of specifying density can achieve satisfactory results. However, the specifications should require that the initial in-place voids in the asphalt mixture do not exceed approximately 8 percent and that the final in-place voids do not decrease below approximately 3 percent. This requires that samples be compacted in the laboratory during construction to estimate the final in-place voids and that the initial in-place air voids be measured during the construction process. As long as sufficient testing is performed to ensure that the initial and final in-place voids are acceptable, then this procedure can be used satisfactorily to specify compaction requirements.

MEASUREMENT OF ASPHALT DENSITY

Core Method

The core method is the referee procedure for density measurement and is the standard to which other methods are compared. It requires a significant amount of time because the pavement has to cool before cores can be taken and the cores must be air-dried to obtain dry weight. In most cases, the density results are obtained the day after construction.

After cutting the core from the pavement, the material outside the layer in which density is being measured must be removed. In some cases, paper or other material is placed on the existing surface before overlaying to reduce the bond between layers. The core can then be separated easily so that the density of the asphalt layer being placed can be measured. The location must be carefully marked so that the core can be taken over the paper. There are some problems in using paper to break the bond between two layers. Because there is a lack of bond in this location, this method may result in lower density over the paper. This approach also identifies the location at which cores will be taken and, hence, may result in some additional rolling in these locations by the contractor. This method of taking cores is not reliable and is not widely used today.

The method most often used to obtain core samples is to randomly locate samples, cut the core full depth, and saw or otherwise separate the layers being tested from the remaining material. This method is usually the most accurate way to evaluate the overall density of the pavement and is the least disruptive to the paving operation.

A problem that sometimes occurs in measuring the bulk density of a core is failure to allow the core sufficient time to dry before obtaining the dry weight. The core should be allowed to air-dry before the density is measured. Drying in an oven at an elevated temperature may result in distortion of the core and, hence, result in an error in density measurement. Measuring density of a core that is not completely dry will result in an erroneously high density value.

Burati and Elzoghbi (*8*) showed that the variability of density test results was less when measured with cores than when measured with a nuclear device (described in the following section). They examined three nuclear gauges on two construction projects and found that there was a statistically significant difference in the average density when measured with cores and nuclear gauges.

Nuclear Gauges

Nuclear gauges have been used for a number of years to measure the bulk density of asphalt mixtures. This technique has the advantage of being rapid and nondestructive.

Most density measurements on asphalt mix have been done in the backscatter mode. In this method, the gauge is set on top of the pavement and a reading is taken that represents the average density for the top several inches of material. For instance, the average density may be representative of the top 6 in. of material, but the layer being evaluated may only be 2 in. thick. Part of the error is removed by calibrating the nuclear gauge to provide the same density as that provided by cores. However, errors still exist because of variations in thickness and density in the underlying layers.

In recent years, a nuclear gauge has been developed to measure the density of thin lifts. This gauge should provide greater accuracy in density measurements when compared to the previous gauge, but sufficient tests to show overall accuracy have not been developed.

The best use of nuclear gauges is in development of rolling patterns and quick determination of approximate density. Because of the possibility of error, nuclear gauges should never be used alone for acceptance testing. Some cores should be taken routinely to verify the accuracy of the gauge and to ensure that an acceptable density is obtained.

Many projects have been constructed in which the nuclear gauge was the only method used to measure density. This practice is not recommended because, even if the gauge is calibrated daily, problems can develop that result in inaccurate readings.

CONCLUSION AND SUMMARY

The amount of voids is the most important property of an asphalt mixture. Because voids vary throughout the life of the pavement, the initial and final voids must be controlled. Initial in-place voids are determined by comparing the bulk density to the TMD. They should not exceed approximately 8 percent. Final voids are controlled by compacting samples (using a manual hammer or similar method) in the laboratory during the construction process. The voids in these samples will be representative of the final in-place voids if correct compactive effort is used. The final in-place voids should not be below approximately 3 percent. Typically, the mix design is performed to provide 4 percent voids in the mix.

As long as a specification is written to ensure that maximum voids do not exceed 8 percent and minimum voids do not fall below 3 percent, then density can be specified as a percentage of laboratory density, control strip density, or TMD. All three methods of specifying density will provide acceptable results if properly used, but the TMD method has been grossly misused.

The method of measuring density must be controlled because voids are directly related to density. The nuclear gauge is quick and nondestructive but is not as accurate as cores. Some cores should always be taken during the construction process to verify that acceptable initial in-place density is obtained.

ACKNOWLEDGMENT

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Investigation of the Interrelationship Between Base Pavement Stiffness and Asphalt Overlay Compaction

ROY D. MCQUEEN

The degree that base pavement support influences the compactibility of an asphaltic concrete (AC) overlay was determined. As a secondary objective, the FAA Eastern Region in-place air voids compaction standard was compared with the FAA National percent Marshall density compaction standard. Field data were collected on three paving projects in FAA's Eastern Region. Nondestructive testing (NDT) was used to quantify the stiffness of base pavements before construction of overlay. After overlay construction, the unit weights of the asphalt overlays were determined at the same locations at which NDTs were performed, and in-place air voids and percent Marshall densities were computed. Statistical techniques were used to investigate correlations between stiffness and AC density (i.e., unit weight, in-place air voids, and percent Marshall density). Although a mild correlation between stiffness and density was found at one project, no general trends were detected for the other projects or from regression analyses performed on combined data bases. This finding may suggest that base pavement stiffness is not a primary variable in affecting overlay compaction on airport pavements; however, the effect of stiffness may have been masked by other external variables, such as temperature, rolling, mix properties, or quality control. Finally, apparent inconsistencies in acceptable quality level and payment were observed between FAA Eastern Region and FAA National density acceptance plans. A follow-up study was initiated by FAA to evaluate the basis both of Eastern Region and National acceptance plans for AC. Recently completed, the new study made definitive recommendations for both acceptance plans with regard to (a) acceptance requirements; (b) quality control requirements; (c) acceptance limits for air voids, stability, flow, and density; and (d) a payment adjustment plan for density.

Because of the high tire pressures and gross wheel loads of modern aircraft, airport pavement construction requirements are necessarily more demanding than those for road construction. This is especially true of asphaltic concrete (AC), both in terms of material and compaction requirements. Highway specifications vary from state to state with respect to AC compaction, but the FAA P-401 specification (*1*) requires AC to be compacted to a target density of 98 percent of 75-blow Marshall density for air carrier airports.

A statistically based acceptance procedure for AC was incorporated into the FAA P-401 specification, and payment adjustment factors were specified for various levels of noncompliance with the density standard. The use of a prescribed acceptance procedure tended to make the specification more enforceable, especially in terms of assessing reduced payments for material not complying with the specification.

Although these acceptance procedures may have heightened contractor awareness of the need for meeting the compaction standard, they also resulted in concern that achieving the standard may not always be possible. Contractors and engineers were particularly concerned that the density requirements would be overly restrictive for overlays on existing pavements that were either extremely weak or variable, which was reportedly the case at many smaller general aviation or commuter airports.

In his study on field compaction of bituminous mixes (*2*), McLaughlin identified 14 parameters that affect pavement compactibility. Of these, high stability, low material temperature, improper equipment, testing error, and weak support systems were found to be at fault most often.

Although contractors generally disagree on the effect of process control on lack of compaction, their most common view is that lack of compaction is usually caused by weak or yielding pavement layers. Proof rolling has been attempted before placement of asphalt overlay to identify areas that could be exempted from the compaction standards, but the results have been inconclusive because of the subjective nature of this process. With many thousands of dollars at stake, as well as the future serviceability of a rehabilitated pavement system, it is generally agreed that a more objective quantitative evaluation process is required.

Although the effect of base pavement support on asphalt compactibility does have intuitive appeal, there is no general agreement that it is a primary variable in influencing the compaction of an AC overlay. FAA has argued that, in developing the acceptance limits for P-401 density, a wide variety of support systems were examined (*3,4*). According to FAA, this resulted in the choice of an acceptable quality level (AQL) for asphalt density, which addressed not only the needs of the FAA, but also the condition of the base pavement and the capabilities of the industry.

With nondestructive testing (NDT) generally recognized as a valid analytical tool for pavement evaluation, it was believed that NDT could be used to objectively determine whether base pavement stiffness affects asphalt compaction and, if so, to what extent. If the stiffness of the support system does affect compaction, then it was hoped that a limiting pavement stiffness could be established. Below this limiting stiffness, a lesser density standard, or a methods specification, could then be applied.

Roy D. McQueen & Associates, Ltd., 3112 Fox Den Lane, Oakton, Va. 22124.

OBJECTIVES

As explained, the basic objective of the research was to determine the degree, if any, that base pavement support influences the compactibility of an AC overlay. If pavement support was found to affect overlay compaction, and, if that effect could be quantified, then a limiting support condition would be determined below which consideration could be given to modifying the current P-401 density standard.

In order to achieve these objectives, NDT was used to quantify preoverlay pavement support conditions. Because primary FAA guidance on NDT involves use of the dynamic stiffness modulus (DSM) (*5,6*), pavement support was defined as the DSM at a particular test location.

As required by FAA, asphalt compaction is defined as the percent of daily Marshall density of field cores. For this study, to allow for economical acquisition of a large data base without operational disruption of or damage to the new pavement surface, the field unit weight of the overlay was determined by nuclear density testing. Any required calibration of the nuclear density device was accomplished through correlation with core density samples.

To accomplish the basic objectives of the study, specific procedures were determined as follows:

1. NDT was performed at three airports in FAA's Eastern Region to obtain data on pavement support conditions before construction of programmed overlays. The NDT program was designed to obtain data for DSM computation and for possible future layered elastic analyses. Approximately 100 tests were performed at each location, within 1 or 2 days of the scheduled overlay.
2. Daily production Marshall and maximum theoretical density (by the Rice method) test results (performed by others) were used to develop a reference data base for compaction computations. Normally, the Eastern Region specification requires four Marshall tests (with specimens compacted at 250°F) and two Rice tests daily.
3. After completion of the overlay, nuclear density testing was performed at the same locations as the NDT tests to determine the unit weights and relative density of the overlays. Several cores were also taken at NDT locations at each airport to establish a core density data base and for correlation of the nuclear density device. Because FAA's Eastern Region uses in-place air voids as its compaction standard (*7*), the unit weights were also used to determine both the percent Marshall density and the in-place air voids at each location.
4. Linear and nonlinear regression analyses were performed to determine whether a correlation between DSM and asphalt density (i.e., percent Marshall density or in-place air voids) exists and could be quantified.
5. The existence of a limiting base pavement stiffness (i.e., DSM) was investigated, below which acceptance procedures other than those currently in use could more aptly apply.
6. Construction acceptance test data were collected to evaluate mix properties or other variables that may have influenced compaction.
7. Although not originally envisioned as part of the research, the collection of data both on percent Marshall density and in-place air voids enabled a comparison of the two compaction standards.

PROJECT DESCRIPTIONS

For field data acquisition, three candidate airports were selected in FAA's Eastern Region. In selecting sites, an attempt was made to obtain data at airports having relatively weak and stiff support conditions, with differing base pavement thicknesses and subgrade soils. Also, because most compactibility concerns had to do with overlay construction, initial candidate sites were limited to those that had overlay construction.

After consultation with FAA, the following projects were identified for field data collection:

- Teterboro Municipal Airport—Runway 1–19 overlay;
- Leesburg Municipal Airport—parallel taxiway overlay; and
- Ocean City Municipal Airport—apron overlay and parallel taxiway extension.

DATA COLLECTION

At each project, field data collection consisted of the following:

1. Assembling contract documents (i.e., plans and specifications),
2. Laying out nondestructive and density test locations using reproducible control points,
3. Performing NDT before overlay construction,
4. Assembling acceptance and quality control test results (performed by others) as required by the FAA Eastern Region P-401 specification,
5. Performing density tests at previously referenced NDT locations after overlay construction, and
6. Obtaining core density test data (performed by others) at selected nuclear test locations for calibration of nuclear test devices.

Nondestructive Testing

Because FAA guidance on NDT references the DSM using a vibratory test device (*5,6*), this control test was used for the correlation analysis. In normal practice, the primary purpose of NDT is to determine the dynamic properties of pavement systems for design. However, for this study, NDT was used to determine the stiffness of various pavement structures before receiving an overlay.

The NDT equipment (dynamic loading system) used for the testing program was designed to generate a dynamic load on the pavement surface and to measure the resultant vertical response of the pavement system, including subgrade, base courses, and surface layers. The equipment includes a microcomputer, which allows rapid data processing during testing. The reliability and repeatability of the dynamic loading system and NDT procedures in general have been demonstrated in several studies (*8–10*).

The equipment generates a dynamic load over a broad frequency range and has the following performance features:

Vibratory force range	500 to 10,000 lb
Impulse force range	5,000 to 25,000 lb
Frequency range	3 to 100 Hz

The DSM (or load sweep) test procedure (*5,6*) is conducted in the vibratory mode at 15 Hz at two force levels, with the DSM defined as the slope of the resulting load-deflection curve.

For asphalt pavements, the DSM at test temperature is adjusted to the DSM at standard 70°F for pavements with asphalt 3 in. thick or more (e.g., Leesburg and Teterboro), using the procedures detailed in the references. For regression analyses (described in a later section), both temperature-adjusted and temperature-unadjusted DSMs were used. To minimize the effects of NDT variance, each test was performed twice at each test location with the results averaged for the regression analysis.

Nuclear Density Testing

Nuclear density tests were performed at NDT locations within 1 to 2 weeks after overlay construction. Test points were carefully located to be as close as possible to the NDT locations.

At each site, nuclear density tests were performed with a Troxler 3411–B nuclear density device used in the backscatter mode. According to manufacturer's recommendations (*11*), backscatter tests were performed at the same locations both before and after overlay construction to factor out the effects of the base pavement unit weight in measuring the unit weight of the relatively thin overlays. The density tests with the Troxler 3411–B were performed by averaging four 15-sec tests, with the gauge rotated 90 degrees after each 15-sec test, holding the probe in the same location.

At Teterboro, an independent set of nuclear tests was performed with the Troxler 4640 Thin Lift nuclear gauge, owned and operated by the Troxler Corporation. Thin Lift measurements were taken at the same locations as the 3411–B measurements. Although the Thin Lift gauge was also available at Ocean City and Leesburg, only a limited amount of data was collected because of time constraints.

Regression Analysis

Because the FAA standard for asphalt density testing is from core unit weight measured in accordance with ASTM 2726, several nuclear density tests were performed near cores taken for normal project acceptance testing. A comparison of nuclear and core unit weights was made, and regression analyses were conducted on the data from each project and on a combined data base.

From tables presented by Young (*12*), the significance of the correlation was examined by comparing the probability (p) of obtaining a given correlation coefficient (R) for a given data set according to accepted rules. A commonly used rule of thumb in interpreting values of R is to regard the correlation as significant if there is less than 1 chance in 20 ($p = 0.05$) that the value will occur by chance.

For Teterboro, a significant correlation ($p < 0.001$) between the Thin Lift results and cores was obtained. Essentially, the analysis indicates that at Teterboro, the Thin Lift results could be used without correction. Although the data bases were smaller, poorer correlations were obtained with the Thin Lift gauge at Ocean City and Leesburg.

At Leesburg, a significant correlation ($p < 0.001$) was obtained for the 3411–B gauge without the thickness correction suggested by the manufacturer. The regression equation obtained at Leesburg was similar to that reported by Burati (*13*) at Morristown.

Correlations of lesser significance were obtained at Ocean City, possibly because of the smaller data base.

On the basis of the analysis of core versus nuclear densities, the following data were used at each project:

- Teterboro—Thin Lift gauge without correction.
- Leesburg—3411–B gauge without thickness correction using the following equation:

$$\text{core density} = 50.63 + 0.634 * \text{nuclear density}.$$

- Ocean City—3411–B gauge without thickness correction using the following equation:

$$\text{core density} = 40.89 + 0.736 * \text{nuclear density}.$$

Marshall Test Data

Because the FAA Eastern Region and National P–401 specifications require daily Marshall testing, Marshall acceptance test data (acquired by others during production) were collected for each project. All samples for Marshall testing were selected by random sampling on a lot basis according to FAA Eastern Region standards (*14*), with a lot defined as 1 day's production. The data were used to compute percent Marshall density and in-place air voids at each nuclear test location for correlation with NDT DSM data. Average Marshall test data for each airport are presented in Table 1.

DATA ANALYSIS

Regression Analysis

Linear and parabolic regression analyses were performed on the data. For each case, the DSM was considered the independent variable (x) and percent Marshall density, in-place air voids, and mat unit weight were considered dependent variables (y). In other words, for each airport an attempt was made to develop regression equations for

- Percent Marshall density as a function of DSM,
- In-place air voids as a function of DSM, and
- Mat unit weight as a function of DSM.

Because of unit weight variances between projects, regression analyses on the combined data base were only performed for DSM versus percent Marshall density and in-place air voids.

In all cases, a better fit was obtained from linear analysis than from nonlinear analysis; therefore, only the linear regression equations were reported. Because statistically significant correlations were found only at Teterboro, these results are presented in Table 2.

TABLE 1 MARSHALL TEST DATA SUMMARY

Project	Gmb	Gmm	%AC	Stability (lbs)	Flow (.01 in)	Voids	Unit Wt (pcf)
Teterboro	2.528	2.612	5.2	2303	10.9	3.2	157.7
Leesburg	2.605	2.677	5.3	2826	12.8	2.7	162.5
Ocean City	2.441	2.501	5.45	2215	11.1	2.4	152.3

Gmb = Bulk Specific Gravity of Marshall Specimen

Gmm = Maximum Theoretical Density of Mixture

%AC = Average Asphalt Content During Production

TABLE 2 RESULTS OF REGRESSION ANALYSES AT TETERBORO PROJECT

$$Y = AX + B$$

X	Y	n	Slope (a)	Intercept (b)	R	P
A. Nuclear Gauge Data - DSM Not Temperature Corrected						
DSM	% Marshall	104	0.000011	0.955	0.252	0.01
DSM	% Air Voids	104	-0.00001	0.080	-0.308	<0.01
DSM	Unit Wt	104	0.00242	150.04	0.326	<0.01
B. Nuclear Gauge Data - Temperature Corrected DSM						
DSM	% Marshall	104	0.000009	0.954	0.291	0.01
DSM	% Air Voids	104	-0.00001	0.081	-0.354	<0.01
DSM	Unit Wt	104	0.00203	150.0	0.364	<0.01
C. Core Data - DSM Not Temperature Corrected						
DSM	% Marshall	18	0.000012	0.951	0.555	0.02
DSM	% Air Voids	18	-0.00001	0.085	0.598	0.01
DSM	Unit Wt	18	0.00246	149.49	0.598	0.01
D. Core Data - Temperature Corrected DSM						
DSM	% Marshall	18	0.000009	0.952	0.565	0.01
DSM	% Air Voids	18	-0.00001	0.084	-0.612	<0.01
DSM	Unit Wt	18	0.00195	149.59	0.612	<0.01

In evaluating the results, the following observations are noted:

1. For the Teterboro project, significant (i.e., $p < 0.05$) correlations were found between DSM and all *y* parameters. However, because the average percent Marshall density for the project, at 96.5 percent, is less than the FAA target average of 98.0 percent, the predictive equations may not be appropriate. This result was found for regression analyses conducted on data bases using both the temperature-corrected and temperature-uncorrected DSM results.
2. For the Leesburg and Ocean City projects, no correlations were found between DSM and any of the *y* parameters.
3. Regression analyses on the combined data base found no correlation between DSM results and either percent Marshall density or in-place air voids.
4. In all cases, similar results were found from regression analyses of both nuclear- and core-generated density data.

Class Groupings

In a second attempt to evaluate the data, the DSM results for each project and the combined data base were sorted in ascending order, with corresponding percent Marshall density, in-place air voids, and unit weights. Class intervals of each hundred measure of DSM (i.e., 200, 300, 400, etc.) were chosen, and DSM (without temperature correction), percent Marshall density, in-place air voids, and unit weights were statistically processed to yield the mean, standard deviation, and coefficient of variance of each parameter within each interval.

No trends were readily apparent either from comparison of averages or coefficients of variance of the data. In other words, the data suggest that, under the conditions of this study, a stiffer support system (as characterized by the DSM), did not result in a higher degree of compaction. Further, variability in base pavement stiffness does not necessarily result in variability in compactibility.

Discussion of Results

A broad range of DSM values (approximately 200 to 2,000 kips/in.) was involved in this study, representing an approximate 10-fold increase in pavement support conditions. Thus, the range of the independent variable should have been sufficient to detect any significant trends in the dependent variables.

However, analysis of the regression and class grouping results found no consistent correlation between base pavement stiffness and compactibility. Although statistically significant correlations were obtained at Teterboro for all parameters, the improvement in correlation coefficients for regression analyses on data using the temperature-corrected DSM results may suggest that the correlations are less robust than suggested by the correlation coefficients.

With this background, analysis of the data collected during this study can be broadly interpreted in two ways:

1. Although base pavement support conditions may influence compaction of an asphalt overlay, the effect of the base pavement may be masked by numerous other variables (*2*). This can mean either that the other variables (e.g., equipment, quality control, and temperature) overwhelmed the effect of base pavement stiffness or that base pavement stiffness is not a significant variable in influencing compaction. In other words, if proper mix design and construction procedures are followed, base pavement stiffness (or the lack thereof) may only then be observed to influence the compactibility of an asphalt mat.
2. Base pavement stiffness on airports designed according to FAA standards has little or no effect on the compactibility of an asphalt overlay.

In developing this study, it was thought that concentrating the data collection at individual airports with the same contractors, mix, and equipment would represent a real-world situation yet reduce the effect of outside variables. However, the scope of the study did not allow detailed observation of construction or analysis of such variables as mix properties, quality control, and compaction temperatures. The purpose was to collect appropriate data as objectively as possible for statistical analysis.

COMPACTION STANDARDS

At the outset of this study, it was recognized that there were differences in density testing requirements between the FAA National (*1*) and Eastern Region (*7*) P–401 specifications. Although the study required evaluating correlations on the basis of both percent Marshall density (National requirement) and in-place air voids (Eastern Region requirement), it was thought that the two requirements were essentially different measurements of the same standard. However, in evaluating the density data collected at the airports, it appeared that the two specification requirements, or application of the two statistical acceptance plans, may be resulting in different density standards.

The National P–401 specification defines compaction in terms of a percentage of the 75-blow Marshall density. A target average density of 98 percent was established with substantial compliance (i.e., full payment) defined as 90 percent of the material in a lot having a density greater than 96.7 percent. The lower tolerance limit of 96.7 percent was established by working back from the target density at an assumed standard deviation of 1 percent. The 1 percent standard deviation for percent Marshall density was confirmed through test results from nuclear devices and cores. Marshall voids are specified to fall between 3.0 and 5.0 percent.

On the other hand, the Eastern Region P–401 specification defines compaction in terms of in-place air voids. Although no target density is specified, substantial compliance is defined as 90 percent of the material falling within lower and upper tolerance limits of 1.0 and 7.0 percent, respectively. For Marshall voids, 90 percent of the material is specified to fall within lower and upper limits of 1.0 and 5.0 percent, respectively.

In equating the two requirements, the percent Marshall target density and the midrange Marshall air voids for the National specification can be compared with the midrange of in-place and Marshall air voids for the Eastern Region specification. This would result in an average in-place air void content of approximately 6.0 percent with the National requirements and 4.0 percent with the Eastern Region specification. With a midrange Marshall air voids content of 3.0 percent required by the Eastern Region, approximately 99 percent Marshall density would be needed to achieve the midrange requirement of 4.0 percent. Further, in applying the upper limits of both criteria to obtain full acceptance, the National specification will allow a maximum average in-place air voids content of 7.0 percent (5.0 percent Marshall laboratory voids plus 2.0 percent from 98 percent compaction). Applying the Eastern Region's acceptance criteria, the maximum allowable air voids for full payment (i.e., 90 percent within limits) is approximately 5.7 percent, assuming a 1.0 percent standard deviation for in-place air voids.

However, the percent compaction actually achieved will vary both with the Marshall air voids achieved during production and with the in-place air voids resulting from field compaction. The percent compaction can be estimated by subtracting the Marshall air voids from the in-place air voids. For example, applying the acceptance criteria for laboratory Marshall air voids, an acceptable average Marshall voids content to meet 90 percent within limits criteria would be approximately 1.7 percent at 90 percent within limits, using the 0.6 percent standard deviation for Marshall air voids suggested in the Eastern Region specification. Therefore, translating to percent Marshall density, the Eastern Region specification would allow full payment with an average percent Marshall density from compaction of approximately 96 percent. For convenience, if 2.0 percent is considered a minimum practical average for Marshall air voids, a minimum average required Marshall density from compaction of 96.3 percent would be required to meet the upper limit 5.7 percent in-place air voids for full payment.

A comparison of the criteria from each specification is presented in Table 3.

In applying the extreme limits of the acceptance criteria from both specifications, the two specifications may have different criteria for defining acceptable material. The acceptance data for all projects, calculated using the nuclear density device, also suggest inconsistency between the two requirements. These data are summarized as follows:

Nuclear Device Tests	*Percent*
Average Marshall air voids	2.8
Average in-place air voids	6.2
Standard deviation for in-place air voids	1.1
Average percent Marshall density	96.6
Standard deviation for percent Marshall density	1.0
Percent asphalt content	5.3

Further, applying the payment schedule formulas from each specification would result in different contractor payments. Assuming a normal distribution for the combined data collected during the study, the percent of material within specification limits would be less than 50 percent under the National specification versus approximately 77 percent of material within limits with the Eastern Region specification. This would result in 50 percent payment under the National specification versus 94 percent payment under the Eastern Region specification for the same material.

On the basis of the preceding discussion, it appeared that both specification requirements should be reevaluated to ensure consistency and establishment of an appropriate, acceptable quality level. This was confirmed by a recently completed follow-up study (*15*), which examined the basis for the Eastern Region and National P-401 acceptance plans. The new study resulted in the following conclusions and recommendations:

TABLE 3 COMPARISON OF COMPACTION STANDARDS

	Specification (%)	
Criterion	National	Eastern Region
Midrange Marshall air voids	4	3
Midrange in-place air voids	6	4
Midrange percent Marshall density	98	99
Minimum Marshall air voids	3	2[a]
Maximum in-place air voids	7	5.7
Minimum average Marshall density	98	96.3

[a]Considered minimum practical; theoretical minimum is 1.7 percent.

1. The Eastern Region procedures can result in a lesser density standard than normally achieved with the National specification. This decrease can have an adverse effect on other quality considerations, such as stability.
2. Either a unified FAA density standard based on percentage of Marshall density should be adopted for all FAA regions or the Eastern Region should modify its Marshall air voids limits to ensure that a minimum of 98 percent of Marshall density is achieved at all Marshall and in-place air voids contents. Revised Marshall air voids acceptance limits are recommended by McQueen (*15*).
3. Acceptance and quality control characteristics were delineated for both specifications. On the basis of the literature search, and pending the outcome of Strategic Highway Research Program (SHRP) research, Marshall stability, flow, and air voids were suggested for statistical acceptance of plant-produced material, with percent Marshall density suggested for statistical acceptance of field material. Tests for gradations and asphalt content were delineated as part of the contractor's quality control responsibility.
4. A new payment plan was suggested because of the nonuniformity in payment when based on density. The density payment plan contains crediting provisions (*16*) to ensure 100 percent contractor payment at the specified AQL. A reduced payment plan was not suggested for compliance with Marshall characteristics at this time.

CONCLUSIONS

On the basis of the study results, the following conclusions are offered:

1. At the Leesburg Airport, a significant correlation between unit weight determined by the Troxler 3411-B nuclear density device and that determined from cores was established similar to that reported by Burati (*13*).
2. The Troxler 4640 Thin Lift nuclear gauge densities correlated with core densities at Teterboro without correction. However, poor correlations were obtained at the Leesburg and Ocean City projects.

3. From the compaction data obtained at the three airports, a 1.0 percent standard deviation was found for percent Marshall density from both core and nuclear devices. A 1.1 percent standard deviation was found for in-place air voids with the nuclear devices and a 1.2 percent standard deviation was found from cores.

4. For the Teterboro project (see Table 2), significant (i.e., $p < 0.05$) correlations between DSM and all y parameters (i.e., percent Marshall density, in-place air voids, and unit weight) were found. However, because the average percent Marshall density for the project (96.5 percent) is less than FAA's target average of 98.0 percent, the predictive equations may not be appropriate.

5. For the Leesburg, Ocean City, and combined data bases, no correlations were found between DSM and any of the y parameters.

6. In grouping the data into class intervals as a function of DSM, no trends are readily apparent from comparison of averages or coefficients of variance of the data bases. This lack suggests that, under the conditions of the study, a stiffer support system does not necessarily result in a higher degree of asphalt compaction.

7. On the basis of the data analysis, base pavement stiffness appears to have no consistent effect on asphalt compactibility. It is assumed that other outside variables (e.g., equipment, mix characteristics, temperature, or quality control) masked the effect of base pavement stiffness or simply that base pavement stiffness is not a significant variable in influencing compaction.

8. Existing support conditions at airports designed to FAA standards may be too stiff to reveal any loss in compactibility caused by weak or yielding base pavements.

9. Evaluation of acceptance procedures for mat density required by the National and Eastern Region P–401 specifications suggested that the specifications may be inconsistent in requiring different acceptable quality levels, resulting in different contractor payments for the same material. This assumption was confirmed during a follow-up study (*15*), which recommended changes to acceptance procedures for both specifications.

RECOMMENDATIONS

Although the study provided useful information, the data did not conclusively prove the existence or nonexistence of a robust correlation between base pavement stiffness and asphalt compactibility. The data obtained at one airport did suggest that the compactibility of an asphalt overlay can be influenced by base pavement support conditions, but no correlations were obtained at the other two airports. Investigation of the interrelationship between stiffness and compaction will require data acquisition under a more controlled environment.

Useful information was provided on the use of nuclear density devices for acceptance or quality control testing during asphalt overlay construction. If the Thin Lift nuclear gauge proves to be more reliable than other gauges (*17*), yielding results consistent with core densities, then the Thin Lift gauge will enable a greater number of acceptance tests to be performed at little or no additional cost.

Further, this study and a follow-up study suggested that apparent inconsistencies exist between the density acceptance procedures required by the National and Eastern Region P–401 specifications. These inconsistencies can result in different acceptance decisions and different payment for material of equal quality, depending on which specification is applied.

Additional research efforts may provide more definitive data on quantifying the interrelationships between base pavement stiffness and asphalt compaction, as well as refinement of National or Eastern Region density acceptance procedures to ensure consistency in quality and payment.

The following recommendations are offered:

1. Because of the good correlation between unit weights obtained by the Troxler Thin Lift nuclear gauge and core weights at Teterboro, the use of the Thin Lift gauge as an acceptance or quality control tool should be further researched.

2. In order to research the interrelationship between base pavement stiffness and asphalt compaction, the effects of extraneous variables should be eliminated by construction of several test strips, using the same material, contractor, construction equipment, and quality control procedures.

3. The National and Eastern Region acceptance plans should be reevaluated to define a consistent AQL and basis of payment for materials of the same quality. This recommendation was accomplished in the follow-up study (*15*), resulting in definitive recommendations for each acceptance plan with regard to (a) acceptance requirements, (b) quality control requirements, (c) acceptance limits for air voids, stability, flow, and density, and (d) a payment adjustment plan for density.

ACKNOWLEDGMENT

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Construction of Large-Stone Asphalt Mixes (LSAMs) in Kentucky

KAMYAR MAHBOUB AND ELLIS G. WILLIAMS

Today, local highway departments are faced with the challenge of designing asphalt pavements for heavy truck loads and high tire pressures using traditional design methodologies that lack the necessary tools to account for these modern situations. Large-stone asphalt mixtures (LSAMs) are gaining popularity among highway agencies that are charged with designing heavy-duty asphalt pavements. High resistance to deformation in these mixes makes them attractive candidates for construction in heavy-truck traffic routes. As a pavement layer, LSAM develops strength by the stress-bridging effect and stone-to-stone contact. Several unique features about LSAM exist that make their construction more of an art than a science. Problems with LSAM are discussed, including segregation, poor compaction, low density, and particle crushing. Each can be avoided during large-stone–mix construction provided that appropriate countermeasures are used. Specific recommendations for large-stone–mix construction and quality improvement are given on the basis of Kentucky's experience with LSAM construction.

Kentucky has long been known for its coal, of which approximately 160 million tons are produced each year. Kentucky's highway system is an integral part of the coal transport network, carrying about 70 percent of the coal.

The structure of Kentucky's weight-distance tax law and its level of enforcement have resulted in coal truck traffic with gross weights ranging from 90,000 to 150,000 lb and tire pressures ranging from 100 to 130 psi. For pavement designers, these loads translate into truck factors ranging from 5 to 50 equivalent single axle loads (ESALs). Kentucky's asphalt pavements are designed with projected lives of 10 to 15 years. With some exceptions, these structures prove adequate for their design traffic. Unfortunately, the design process does not always identify the increased gross loads and tire pressures, and pavement design life may be reached in less than 6 years. In some locations, excessive rutting develops within the design life, which is prevalent on long, steep grades carrying a high percentage of coal trucks. This problem severely affects the top 3 to 5 in. of the flexible pavement thickness in which the shear stress reaches its maximum. Generally, this condition is considered to be a mixture problem rather than a structural problem.

The asphalt pavement community in Kentucky accepted the challenge of designing and constructing a mix that would handle such heavy and severe highway loads. A task force addressing this challenge was formed with individuals representing the Kentucky Department of Highways (DOH), Kentucky Plantmix Asphalt Industry, Kentucky Transportation Center at the University of Kentucky, Asphalt Institute, National Asphalt Pavement Association, Chevron USA Inc., and Ashland Oil Co.

The task force recommended that alterations in the aggregate gradations could provide more stone-to-stone contact and higher stress resistance (especially shear stress) and thereby yield the needed improvements in rutting and shoving resistance.

LSAM CONSTRUCTION PROJECTS

The Kentucky DOH selected several coal haul highway sections for field testing of an experimental LSAM. The large-stone gradations tested, shown in Figure 1, were selected from an initial group of 12 aggregate blends. Mix design and other laboratory activities were coordinated by researchers at the Kentucky Transportation Center, University of Kentucky.

The following large-stone construction projects in Kentucky were included for review—Louisa Bypass on US-23 in Lawrence County and Mountain Parkway in Powell County. A summary of mix design parameters is presented in Table 1. Aggregate gradation distribution of these two projects is shown in Figure 1.

Louisa Bypass Project

This 3.7-mi, four-lane project is a newly constructed pavement section located deep in the heart of the coal country in eastern Kentucky. The original subgrade (CBR 4) was modified and upgraded by 8 in. of granular subbase for half of the project's length and by a shale subbase for the remaining half. Shale was used because of an on-site shortage of rock during the subgrade and subbase construction. Variation in the subbase material provides an opportunity to evaluate long-term performance variation caused by structural arrangement. Originally, the pavement was intended to be a full-depth asphalt structure; however, because of the presence of shale in the subgrade, which is prone to rapid strength deterioration, a granular subbase layer was included. The subbase layer consisted of 4 in. of dense-graded aggregate (DGA) covered with a 4-in., open-graded, large-stone drainage layer.

Twelve inches of LSAM base layer was constructed on top of the subbase layer in three 4-in. lifts. A 1-in. surface wearing course completed the project. Asphalt grade used was AC–20, and for half of the project the asphalt in the surface wearing course was modified with a polymer. Use of polymerized

K. Mahboub, Materials Research Section, Kentucky Transportation Center, Transportation Research Building, 533 S. Limestone St., University of Kentucky, Lexington, Ky. 40506–0043. E. G. Williams, Duo Services, Inc., 121 Wood Road, Louisville, Ky. 40222.

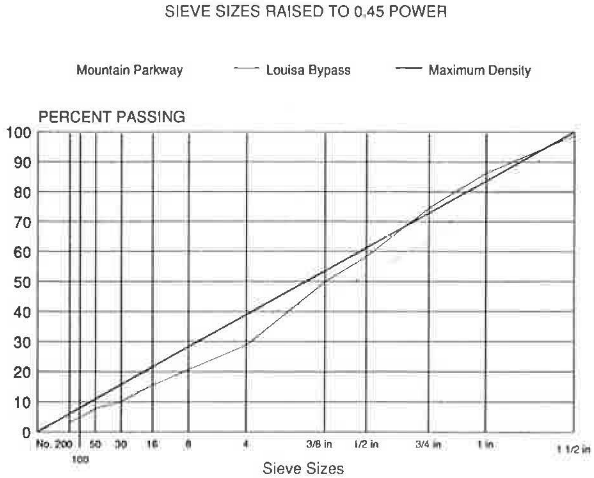

FIGURE 1 Large-stone asphalt mix project gradations.

asphalt was part of Kentucky DOH's experiment with modified asphalts.

Mountain Parkway Project

This project consisted of a series of rehabilitation projects totaling 25 mi to upgrade an old portland cement concrete (PCC) section of a four-lane parkway. The existing PCC pavement was cracked and seated and then overlayed with 6.5 in. of LSAM base layer. The LSAM was constructed in two lifts and a 1-in. surface wearing course completed the construction. Asphalt grade used was AC–20 throughout the project.

TIPS ON CONSTRUCTION OF LSAM

The following items are the result of many observations made during construction of the two previously discussed LSAM projects. Some may apply to all types of hot-mix asphalt (HMA) construction; however, in many instances large-stone mixes are more sensitive to construction errors than their conventional counterparts. Therefore, maintaining close, technical supervision over mix design, plant mixing, mix laydown, and compaction operations is important during the construction of LSAM.

Mix Design

The 6-in.-diameter by 3.75-in.-thick modified Marshall method of mix design (*1*) was adopted by the Kentucky DOH. Several factors contribute to successful LSAM mix design. For instance, adequate asphalt film thickness (9 to 11 microns) is necessary to ensure workability and durability. Film thickness is controlled by asphalt content and percent mineral filler in the aggregate. In conventional HMA construction, asphalt film thickness ranges from 6 to 8 microns and fine materials act as asphalt extenders (*2*). A thicker film is desirable to assist compaction of rather harsh LSAM mixtures.

Percent voids in the mineral aggregate (VMA) must be enough to accommodate the desired film thickness at maximum field density without excessive reduction in air voids. The VMA of the Kentucky LSAM was 11.5 percent, which is consistent with the widely accepted criteria set by the Asphalt Institute (*3*) and the National Asphalt Pavement Association (*4*).

Laboratory compaction of 6-in.-diameter by 3.75-in.-high Marshall specimens can be achieved at 112 blows per side using a 22.5-lb Marshall hammer (*1,5–7*). Densities achieved in the laboratory can be closely duplicated in construction (see Figure 2).

Air voids should be in the 3.5 to 5.5 percent range with an average of 4.5 percent. This range will minimize both air and water permeabilities. Figure 3 shows the variations in the air void content of the laboratory and field specimens.

Plant Mixing

Plant mixing time may need to be slightly adjusted for LSAM. A longer mixing time, as compared with conventional HMA, may become necessary to ensure coating of larger aggregate particles.

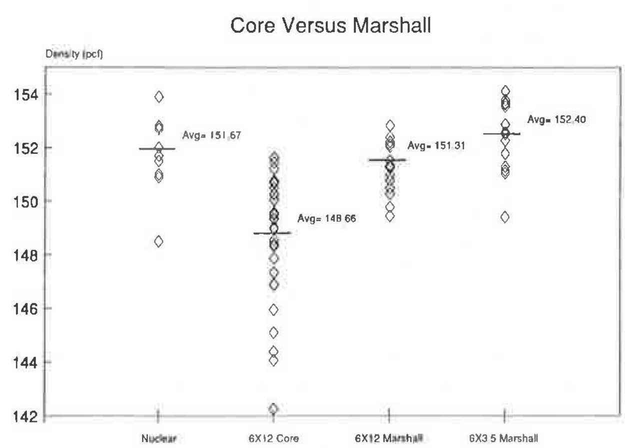

FIGURE 2 Laboratory and field density data—Louisa Bypass project.

TABLE 1 MIX DESIGN PARAMETERS FOR TWO KENTUCKY LSAM PROJECTS

	Stability (lb)	Flow (0.01 in.)	Air Voids (%)	Asphalt Content (%)	VMA (%)	Retained Tensile Strength (%)
Louisa Bypass	5,300	16	3.6	3.6	13.1	Pass
Mountain Parkway	5,900	19	4.4	3.5	13.0	Pass
Criteria	3,000 (min.)	28 (max.)	3.5–5.5	3–6	11.5 (min.)	70

NOTE: Data are based on 6-in.-diameter by 3.75-in.-thick modified Marshall specimens compacted at 112 blows per side using a 22.5-lb hammer.

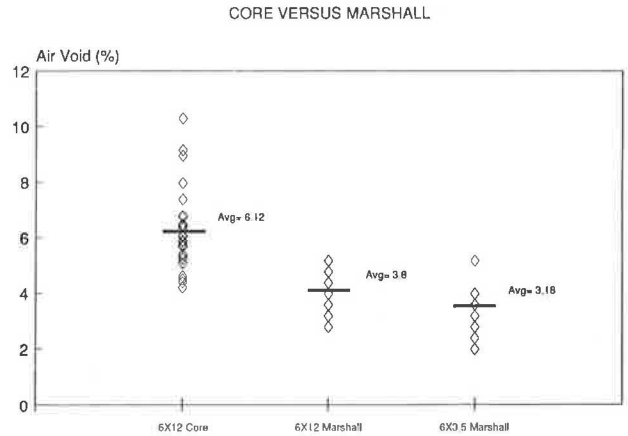

FIGURE 3 Laboratory and field air voids data—Louisa Bypass project.

In addition, careful attention to aggregate feeding and mixture handling to avoid segregation is essential. Cone formation of aggregate and mixture can be avoided by multiple material drops, which will minimize segregation.

Lastly, attention is necessary to ensure that the LSAM mix did not induce any unusual amount of wear on the plant's mixing equipment.

Laydown Operations

Several important laydown operational details exist that can be used to minimize segregation in the LSAM. For instance, coarse particles that accumulate in the paver wings should be discarded and never incorporated into the flow of the mix to the screed hopper.

The mixture in the receiving hopper bed should be maintained at a minimum depth of 18 to 24 in. to prevent accumulated coarse particles from reaching the slat conveyor. Receiving hopper gates should be set to provide as nearly a continuous flow of mixture as possible. Continuous operation of the distribution augers at full capacity is required to ensure mass movement of material for the entire screed.

Paver speed should be regulated to accommodate the mixture production and transport rates. Stop-and-go operation of the paver should be avoided to reduce segregation, improve spread texture, and eliminate any tendency for screed settlement (*5,6*).

A minimum lift thickness of 3.5 in. will minimize the effect of large aggregate boundary restrictions.

Compaction Operations

Although most LSAM gradations are coarsely graded and tend to be harsh, required density can be readily achieved with proper use of a variety of suitable, conventional compaction equipment (*5–7*).

Primary compaction should start immediately after mixture spreading. Density can be readily achieved at compaction temperatures ranging from 250°F to 300°F. Compaction at lower temperatures requires a considerable increase in roller coverage and is not recommended. Lateral displacement of this rather harsh mix was not a problem. For example, a successful compaction sequence included the following: (a) two passes of vibratory roller in the static mode for breakdown rolling, (b) six passes of vibratory roller at high frequency and low amplitude for primary compaction, (c) four passes of pneumatic roller to complete compaction, and (d) two passes of a static roller to smoothen the surface.

Because the stone-to-stone contact structure of LSAM can produce high point stresses on large aggregate particles during compaction, the frequency and amplitude of the vibratory roller may need to be adjusted to reduce particle breakage and to optimize compaction. This practice is especially necessary whenever relatively rigid bases (Mountain Parkway LSAM overlay on cracked and seated PCC project) are encountered.

Quality Control

A quality control routine should follow the construction of LSAM closely in order to ensure adherence to design parameters such as aggregate gradation, asphalt content, density, and air void content. Moving averages should be maintained and used as the basis for evaluating variability of mixture parameters. A schematic of the concept of moving averages that is recommended for quality control is given in Figure 4.

Asphalt extraction and gradation tests should be conducted on as large a quantity of LSAM material as the equipment will permit to ensure that the samples will be representative of the bulk material. In lieu of time-consuming extraction tests, total daily mixture output of the plant and asphalt cement tonnage is a convenient and relatively accurate way of determining the average daily asphalt content.

Compaction pattern is a function of the equipment that is available at the construction site and should be determined initially by construction of a test section (at least 500 ft long and 12 ft wide). Construction of a control strip is also useful for detecting potential segregation problems. Rolling patterns and coverages that are needed to produce the desired density should be maintained throughout the job. The Kentucky projects used a target density on the basis of 93 to 94 percent of

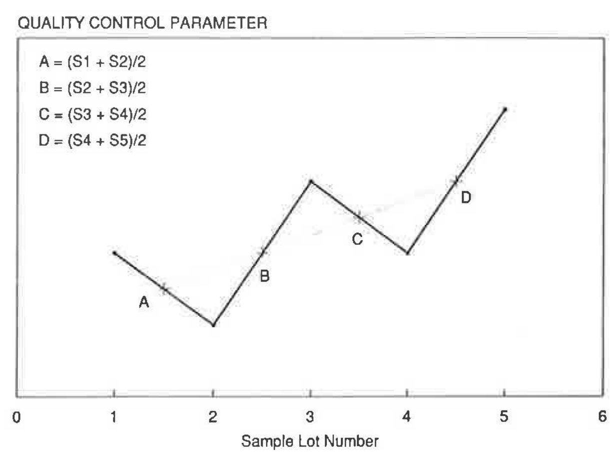

FIGURE 4 Schematic representation of the moving average concept.

solid volume (i.e., 6 to 7 percent air voids). Control range was set at 92 to 97 percent of solid volume.

Field density evaluation must be made frequently to ensure that the compaction procedure is adequate. If the desired density is not being achieved, adjustments to roller coverage should be made. If large adjustments are required, a new test section should be constructed.

CONCLUSIONS AND RECOMMENDATIONS

The experience in Kentucky indicates that LSAM can be designed and constructed with minimum modifications to existing procedures. Special attention must be devoted to plant and paver operations for reducing the probability of segregation. Lift thickness should not be reduced below 3.5 in. (for 1.5-in. top size gradation) to ensure adequate degrees of freedom for particle reorientation during compaction.

Mix design procedures for LSAM are not well developed. The modified Marshall stability test (6-in.-diameter by 3.75-in.-thick specimen) is the best mixture design procedure currently available for LSAM. However, further development and standardization of mix design procedures on the basis of the 6-in. modified Marshall approach is needed. Current construction equipment and procedures are appropriate for LSAM. Careful attention to production and construction details is essential for providing a uniform mixture and an effectively constructed LSAM pavement layer.

ACKNOWLEDGMENT

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ODOT–OSU Quality Assurance Training Program: An Example of Successful Outreach Activities

ROBERT K. HUGHES AND SAMIR A. AHMED

Recently, the Oklahoma Department of Transportation (ODOT) has adopted statistically based quality control and quality assurance (QC–QA) specifications for their highway construction projects. In response to training needs, ODOT entered into an agreement with the school of civil engineering at Oklahoma State University to develop and implement a QC–QA training program. The program consists of seven instruction modules that cover different aspects of construction methods, process control, and acceptance sampling and testing procedures. The development, organization, implementation, and evaluation of the QC–QA training program are described. The new specifications are briefly reviewed.

For a number of years, the Oklahoma Department of Transportation (ODOT) has pondered the idea of adopting statistically based quality control and quality assurance (QC–QA) specifications for their highway construction projects. But not until 1987 did ODOT decide the time was right to convert their recipe-type specifications to QC–QA specifications. Under the new specifications, the contractor is responsible for quality control of processes, whereas ODOT is responsible for assurance and acceptance of the finished product. In a sense, the QC–QA specifications are not end result specifications because ODOT will continue to monitor the contractor's process activities and to place some restrictions on materials and methods used.

The primary advantage of the QC–QA specifications for ODOT is the placing of responsibility for process control, including mix design, inspection, and testing, in the hands of the contractor or producer. Oklahoma's healthy increase in highway construction projects has overtaxed ODOT manpower resources needed to run the job for the contractor under the old specifications. Figure 1 shows the dollar amount of highway contracts awarded by ODOT from FY1982 to FY1989. During FY1988, contracts averaged over $28.5 million per month and at any point in time as many as 500 major construction projects were underway throughout the state. Other advantages for ODOT are the reduction in the number of disputes between the owner and contractor and the ability of the owner to enforce the contract requirements more effectively. Terms such as "within reasonably close conformity," which leave considerable room for personal interpretation, are no longer part of the specifications.

Advantages for contractors and producers accrue from their ability to choose materials, methods, and equipment, thereby enabling them to gain an edge over competitors. The risk involved in having an acceptable material or construction rejected by the engineer is defined a priori before bidding, and ambiguous interpretation or enforcement of specifications is eliminated. Most advantageous of all is the fact that acceptance sampling and testing is based on sublots. Sublots give the contractor an early warning and an opportunity to take corrective action before large quantities of nonconforming material or construction are produced.

In planning for QC–QA specifications, ODOT sought input from contractors and other states, which have implemented similar programs. The Association of Oklahoma General Contractors (AOGC), Oklahoma Asphalt Pavement Association, and Oklahoma Concrete Paving Association have been involved in the development of the new specifications. The ultimate goal has been to ensure that highway products built in Oklahoma will have good serviceability during their design lives at minimum overall costs.

Recognizing that attainment of quality depends on the motivation, knowledge, and attitude of all parties involved, ODOT sponsored a QC–QA training program for their testing and inspection employees as well as the contractor's production and quality control personnel. The school of civil engineering at Oklahoma State University was chosen by ODOT to accomplish the training tasks.

OVERVIEW OF ODOT QC–QA SPECIFICATIONS

In order to facilitate future revisions of the specifications, ODOT chose the special provision to introduce QC–QA specifications. Five special provisions have been developed for asphalt concrete pavement, portland cement concrete (PCC) pavement, structural concrete, embankments, and bases. The first two special provisions have been included in four contracts awarded in 1989. Special provisions for embankments and bases are still under review by ODOT. For each type of construction, six or seven quality characteristics have been identified for acceptance purposes. Table 1 presents the applicable characteristics for asphalt pavement, PCC pavement, and structural concrete. In addition, requirements for other material characteristics as described in the ODOT Standard Specifications must also be satisfied.

Contractors are responsible for furnishing and maintaining a quality control system that will provide reasonable assurance that the product will meet contract requirements. Contractors are required to submit a quality control plan outlining inspec-

Oklahoma State University, School of Civil Engineering, Stillwater, Okla. 74078.

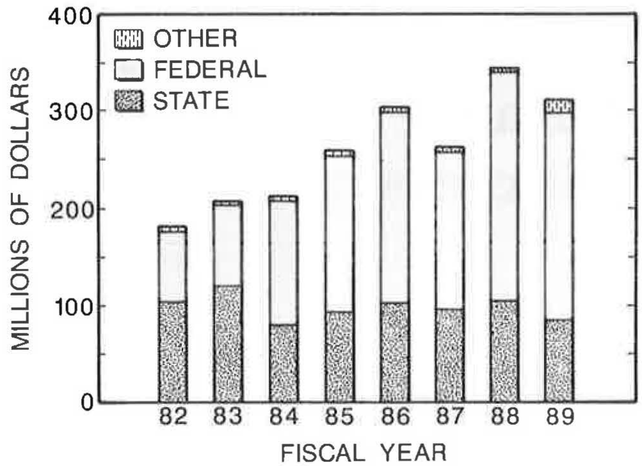

FIGURE 1 Dollar amount of highway construction awarded.

TABLE 1 QUALITY CHARACTERISTICS FOR PRODUCT ACCEPTANCE

Product Type	Quality Characteristics
Asphalt Pavement	Asphalt Cement Content Gradation Air Voids Hveem Stability Roadway Density Surface Smoothness
P.C. Concrete Pavement	Unit Weight Gradation Air Content Strength Thickness Surface Smoothness
Structural Concrete	Unit Weight Air Content Strength Cover on Steel Cracking Surface Smoothness

tion and testing procedures to be used. The special provision for asphalt concrete requires the contractor to formulate a job mix formula acceptable to the engineer.

Acceptance decisions are based on a prescribed amount of material or construction, which is referred to as a lot. Lot size is 4,000 tons for asphalt pavement, 10,000 square yards for PCC pavement, and 200 cubic yards for structural concrete. These lot sizes were determined based on economic, engineering, and statistical considerations. For acceptance testing purposes, a system of stratified random sampling is used by the resident engineer. Each lot is divided into four equal sublots and sample units are selected from each sublot using a random number table.

Pay schedules have been established for each of the quality characteristics presented in Table 1. Each pay schedule is used to determine an individual pay factor (PF) for the corresponding characteristic. Table 2 presents example pay schedules for asphalt content and Hveem stability of asphalt concrete pavements. The value of PF depends on the average of the deviations of test results from a specified standard, which may be a maximum, minimum, or target value. When a target value is specified, such as in asphalt cement content, deviations above or below the target are considered equally bad. An incentive pay factor of up to 1.05 is allowed for exceptional surface smoothness. A weighted average of individual PFs, known as the combined pay factor (CPF), is used to adjust the contract price. If a test result appears to deviate markedly from the majority of test results used for acceptance purposes, the suspected value must be closely examined by the engineer to determine its validity. The examination covers the procedures used in sampling and testing and, if necessary, a statistical analysis performed in accordance with ASTM E178-80 using an upper significance level of 2.5 percent.

ESTABLISHING A TRAINING PROGRAM

ODOT's decision to adopt QC–QA specifications was met with skepticism and concern by highway contractors. The fact that the proposed specifications would establish variable pay factors to be determined by ODOT quality assurance testing was of paramount concern. Also, the contractors were concerned about identifying a sufficient number of qualified personnel to perform their own quality control responsibilities. The questions most frequently asked by contractors were, "Who would do the ODOT quality assurance testing and how would these tests be done?" A number of individuals stated that their experience was that the testing and sampling procedures used by ODOT and contractor personnel were not uniform across the state, which tended to increase the variability of test results far beyond the inherent variability normally expected in testing and sampling results.

Contractor personnel were not alone in their concerns about the new specifications and procedures to be implemented by ODOT. Many of ODOT's district and residency personnel were also skeptical of the proposed changes. As with the contractors, ODOT personnel doubts probably resulted from a lack of understanding of the proposed changes as much or more than from the changes themselves.

The lack of understanding and resistance to the proposed changes that would arise among ODOT and contractor personnel, once the proposed changes were announced, were anticipated early in the planning stages by ODOT, AOGC, and FHWA management. Representatives from the school of civil engineering at Oklahoma State University were invited to a meeting with ODOT, AOGC, and FHWA personnel to discuss the proposed QC–QA specifications and the anticipated ramifications among the affected personnel.

After several meetings and lengthy discussions, the decision was made to develop a training program in asphalt and PCC testing procedures that would be required by the QC–QA program. Personnel from ODOT, contractor organizations, suppliers of highway construction related products, and testing concerns would be invited to participate in the proposed training. A major goal of the training would be to improve uniformity in testing and sampling procedures throughout Oklahoma's highway construction industry while simultaneously garnering synergistic effects to abate the concerns of

TABLE 2 EXAMPLE PAY SCHEDULES

Quality Characteristics	Pay Factor	1 Test	2 Tests	3 Tests	4 Tests*
		Average of Deviations From Target (Without Regard to Signs)			
Asphalt Cement	1.00	0.00-0.70	0.00-0.50	0.00-0.40	0.00-0.35
Content (Extraction	0.95	0.71-0.80	0.51-0.57	0.41-0.46	0.36-0.40
or Nuclear)	0.90	0.81-0.90	0.58-0.64	0.47-0.52	0.41-0.45
	0.80**	0.91-1.00	0.65-0.71	0.53-0.58	0.46-0.50
	unacceptable	Over 1.00	Over 0.71	Over 0.58	Over 0.50
Target JMF, Percent (%)					
		Average of Deviations From Target (Without Regard to Signs)			
Asphalt Cement	1.00	0.00-0.30	0.00-0.21	0.00-0.17	0.00-0.15
Content	0.95	0.31-0.35	0.22-0.25	0.18-0.20	0.16-0.18
(Digital Printout)	0.90	0.36-0.41	0.26-0.29	0.21-0.24	0.19-0.21
	0.80**	0.42-0.46	0.30-0.33	0.25-0.27	0.22-0.23
	Unacceptable	Over 0.46	Over 0.33	Over 0.27	Over 0.23
Target JMF, Percent (%)					
		Average of Deviations From Minimum (Considering Signs)			
Hveem Stability	1.00	(-)1	(-)0	(-)0	(-)0
(Lab. Molded	0.90	(-)3	(-)2	(-)1	(-)1
Specimens)	0.80**	(-)4	(-)3	(-)2	(-)2
	Unacceptable	Over (-)4	Over (-)3	Over (-)2	Over (-)2
Minimums					
2500 ADT or more & All City Streets	40				
Less than 2500 ADT	35				

* If more than four tests are conducted, the allowable deviations will be determined by dividing the allowable deviations for one test by the square root of the number of tests actually conducted.

** Unless otherwise directed by the Engineer, products testing in this range are unacceptable, and shall be removed and replaced at no additional cost to the Department.

the affected personnel and to improve ODOT and contractor communication.

The school of civil engineering at Oklahoma State University was asked to prepare a proposed training program outline. In order to assist in the proposal preparation and to serve as a starting point, ODOT provided a list of the tests that it felt would be required to achieve a satisfactory QC–QA program. A team was assembled to begin developing the training program syllabi. Working with ODOT Materials Division personnel through the spring of 1988, with frequent meetings to report progress and to solicit input from ODOT management and AOGC and FHWA personnel, the program outline was completed and accepted by ODOT in the summer of 1988.

The accepted recommendation included five-day training programs in aggregates, asphalt materials, asphalt paving, concrete materials, concrete construction, and soil mechanics, and a 3-day course in statistical quality control in highway construction. A list of laboratory equipment required for each course (or module as they soon came to be called) was developed by the faculty, coordinated with the ODOT Materials Division, and accepted by ODOT management. A cost estimate of $40,000 to develop course materials, purchase training aids and materials, and convert an open bay area of 2,500 ft^2 into a suitable training facility was presented to ODOT. Because of the number of training sessions anticipated, a training facility separate from the school's teaching and research facilities was felt essential. ODOT also was requested to purchase and loan laboratory equipment worth approximately $250,000 to the school of civil engineering. Figure 2 shows a wide-angle photograph of part of the training facility.

In October 1988, the decision to move forward with the training program was received and work began in earnest to develop course materials and visual aids at OSU while ODOT began ordering the laboratory equipment. Priorities for ordering equipment and course development were established for the seven modules on the basis of ODOT's plans for timing the introduction of the new QC–QA specifications in their contract lettings. The order of priority approved by ODOT was aggregates, asphalt materials, concrete materials, asphalt paving, soil mechanics, concrete construction, and statistical quality control. A target date of February 1, 1989, was established to have the first module (aggregates) ready for presentation.

The aggregate module was conducted during the week of February 6, 1989, and the other modules, with the exception of concrete construction, were offered on a schedule agreed to by ODOT and the school of civil engineering. The structural concrete module was delayed three months because the course content needed revision to satisfy both ODOT and contractor concerns that arose. After several meetings with ODOT and contractor personnel, all of whom had extensive experience in the design and construction of concrete pavements and structures, the course outline was approved. At this time all courses are being offered.

PROGRAM CONTENT

In meeting the major goal of the QC–QA training program, which is to improve uniformity in testing and sampling procedures in highway construction throughout Oklahoma, each participant must understand the reason for each test, the proper way to perform the test, the range of results for each test, and the consequences of not achieving specified or desired results. Participants also need to become familiar with variations in test results that will result from deviations in acceptable and uniform sampling and testing procedures and the impact on product quality, and ultimately, on the pay factors.

All modules except the statistics module begin on Monday morning and run through noon on Friday. Depending on the tests being presented, students are frequently required to arrive by 7:00 a.m. and often depart as late as 6:00 p.m., but efforts are made to limit the lecture portion of the day to no more than three 50-min sessions. The statistics module is a 3-day course, but a 1-day introductory course is also offered. The statistics courses are offered at various locations around the state. A brief description of module content follows.

- Aggregates. Instruction includes familiarization with aggregate types and sources in Oklahoma. Production, processing, and handling of aggregates are thoroughly discussed. Aggregate properties, with heavy emphasis on sampling and testing procedures, are covered. Tests include sieve analysis and materials passing the No. 200 sieve, specific gravity and absorption, sand equivalent, fractured faces, clay lumps, soundness, alkali reactiveness, freeze-thaw, Los Angeles abrasion, liquid limit, plastic limit, standard proctor compaction, and a number of related sampling and field lab tests. This course must be taken before attending asphalt, concrete, and soils courses.
- Asphalt Materials. Instruction emphasizes ODOT asphalt concrete mix design procedures. Production and testing of asphalt cement and methods of combining aggregates to meet specifications are included. The Gyratory-Shear Method of mix design is thoroughly covered. All pertinent ODOT and AASHTO test methods are presented. These tests include temperature and volume relations, testing asphalt cements, combining aggregates, gyratory shear mix design, mixing and

FIGURE 2 Training facility.

molding test specimens, bulk specific gravity, density and void analysis, and Hveem stability tests and other related procedures.

- Asphalt Paving. This module provides course participants with instruction in the operations of the various types of hot-mix asphalt plants and hot-mix sampling and testing procedures. Paving operations including delivery of hot-mix asphalt to the job site, placement of mixture on the roadway, and compaction of the mix to target density are covered. Other topics include roadway preparation, paved and unpaved surface inspection, prime and tack coats, heater planer and cold milling specifications and operations, compaction principles, inspecting the finished pavement (surface texture and tolerance, pavement density), and other related topics.
- Concrete Materials. This course is designed to give participants a working knowledge of concrete used in highway construction. Mix design of concrete according to the American Concrete Institute's ACI211.1 is introduced. Topics covered include important properties of concrete; PCC and admixtures; fineness of cement; field tests for concrete; laboratory testing; mix design; batch proportions and adjustments for moisture; adjusting trial mix for yield, slump, and air; tests on hardened concrete; and special testing.
- Concrete Construction. Instruction expands on the material presented in the concrete materials module to include producing, transporting, placing, curing, and testing of PCC both for paving and structural projects. Topics covered include preplacement activities such as grade requirements and methods of checking line and grade, form requirements, steel placement, admixtures, joint details, ordering and receiving concrete, placement and consolidation, finishing, construction problems in concrete pavements and concrete structures, and a number of related tests and topics.
- Soil Mechanics. Instruction is directed toward exposing participants to the common soils tests used in earthwork construction quality control programs. Tests include Atterburg Limits, compaction, relative density, and in-place methods of testing. Compaction quality control procedures and soil modification methods are discussed in detail. Attendees receive instruction in all pertinent standard test methods and ODOT standards.
- Statistical Methods of QA–QC. This program is designed to give participants familiarity with quality control and quality assurance through process control and acceptance sampling. Topics covered include statistical sampling, methods of describing data, sources of variability in highway products, and the basics of normal distributions. Control charts, acceptance plans, producer's and buyer's risks, specifications, and tolerances are introduced. Attendees learn to do manual calculations and are then given instruction in the use and application of calculators to the QA–QC process.

PROGRAM PRESENTATION

The requirements for administering the program are met by the college of engineering's extension office, which provides accounting and billing services, prepares mailers and publicity, accepts registrations, and conducts registration and distribution of program materials on the first morning. Additionally, extension staff provide some personal services for the participants, including assisting in obtaining room reservations, identifying restaurants and other services, delivering messages, and other assistance as needed to make the student's stay on campus more enjoyable.

Classes are conducted in a classroom equipped with the latest in projection equipment, cushioned seating, and large work tables. The classroom is located across the hall from the laboratory facility. The general format of the instruction is to introduce the test or sampling procedure to be taught, show a video and discuss the proper procedures for conducting the test, adjourn to the laboratory where the tests are run individually or in teams of four students, and reassemble in the classroom to discuss the test results and lessons learned. The 24 students, which is maximum enrollment, are placed into teams of 4 students on the first morning on the basis of questionnaires completed by the students as they sign in. A special effort is made to ensure a mix of the students from ODOT with those from contractors, suppliers, and testing organizations. Also, the more experienced students are spread throughout the six laboratory teams to take advantage of their expertise and experience by assisting the less experienced students.

Laboratory teams formed with personnel from various organizations were intended to improve communications between personnel from throughout the highway construction industry, and the practice has been successful. The fact that the experience level of the students varies from extensive to none at all creates a challenge for the instructor to present the material so as not to bore the former or totally confuse the latter. Experienced students are called on to relate their personal experiences or observations and requested to assist the less experienced students, resulting in an interest level that has remained high for most. Many experienced technicians, who discovered that they had not been following correct procedures over the years, began to develop a better understanding of the purpose of the tests and the effect of poor test results. Such realizations also contributed to maintaining a high level of interest in the instructional material for the experienced students.

The training program was designed primarily for technicians who would be performing the tests in the field or at the plant for the contractor's process control and for ODOT personnel who would be conducting the QA testing. However, a number of organizations have elected to have engineering personnel in decision-making positions attend as well. Most of these individuals have been favorably impressed by the program and have expressed gaining a greater appreciation for the data they were required to examine and evaluate. Likewise, many of the engineers who attended the technician training program felt that the experience would be valuable in fulfilling their supervisory responsibilities as well as in decision making related to the determination of pay factors or in negotiating claims.

The 24 slots for each module are generally committed on a first-come basis with the exception that seven positions are reserved for ODOT. The first time that aggregate, asphalt materials, and concrete materials were taught, the 24 slots were divided equally between ODOT and AOGC. ODOT personnel from the division in which the first QC–QA specification contracts were to be let received these slots, and contractors who were on the prequalified bidding list and were expected to bid on the contracts were offered the others. By the end of 1989, 11 months into the training program, 600 students will have completed one of the modules.

Instructor recruitment for this extensive and continuing training program has proven to be an enlightening experience. ODOT provides several lecturers for specific topics. Others come from the Oklahoma Geological Society, the Oklahoma Asphalt Paving Association and other trade associations, active contractors with extensive experience, and personnel from materials suppliers associated with highway construction in the state. But most are retired ODOT, FHWA, and contractor personnel with extensive knowledge of highway construction who want to make a contribution to the training of young engineers and technicians. The honorariums they receive for their participation appear to be far less important to them than the feeling of satisfaction they derive from making a contribution to the efforts to raise the quality of construction in Oklahoma.

Students are given a 2- to 3-hr written examination at the conclusion of the course. This examination is open book with objective-type questions on the various tests conducted during the course. A number of problems are given that require manipulating data and plotting test results. The failure rate has ranged from 0 to 25 percent of a class, but the failure rate for all modules since the program began in February is 5 percent. Students who fail the examination are allowed to retake the examination the next time that particular examination is offered. To date, no one who has retaken the examination has passed although some improvement in their scores was noted.

PROGRAM EVALUATION

Following the final examination, students are asked to submit a completed evaluation form, which was distributed on the first day, on the course content and each instructor who lectured during the module. Most of the comments have some degree of validity and many have resulted in modifying course content, procuring additional equipment, implementing additional safety precautions, and replacing visual aids. Additionally, three instructors have not been invited back for further participation on the basis of consistently poor student evaluations. Each module is continuously examined and evaluated to improve the quality of the level of instruction and technical content.

Occasionally, a negative comment is received from an employer who felt that by sending an employee to one of the 5-day training modules a totally proficient and technically competent materials technician would be gained. However, in these training modules, as in most learning experiences, the student can only be exposed to the information. Constant application of the information over a period of time is required to gain or develop any degree of proficiency. Thus, ODOT and AOGC are striving to educate their constituents that the purpose of the QC–QA training program is not to create instant technicians, but to provide their employees with a firm technical base from which to develop.

As a result, one lesson learned is making clear the purpose of the training program early so as not to distort individual or corporate expectations. Another is that the modules should be scheduled to concentrate on the nonconstruction season from November through March. From April through October, meeting the minimum number of students required to break even economically was difficult.

It was anticipated that some individuals might have difficulty with the basic mathematics required to manipulate the data resulting from testing and sampling which would be required in the training modules. In order to ensure that all students possessed the basic mathematics skills to successfully complete the training, a 20-question mathematics test was developed and administered by the state's extensive vocational-technical network. All individuals, except those with proof of having taken a college algebra course, are required to take and achieve a designated score on the test before being accepted into a module. One problem area not anticipated was that several students have had reading problems. Usually, these students are experienced technicians who have a firm grasp of the technical procedures and can work with numbers but cannot use reference specifications or read and comprehend examination questions. A solution for this problem has not been identified, but consideration is being given to adopting a comprehensive reading examination requirement.

In the future, the number of students to be trained is expected to remain constant for the next year. After that, the number of module offerings will probably decline to a level where the number of technicians in the industry can be maintained as individuals enter and leave the industry.

CONCLUSIONS

A training environment in which ODOT, contractor, and other industry personnel study and socialize together improves communications and enhances professional respect. This benefit was evident by the number of business cards and telephone numbers exchanged each Friday as the participants were preparing to depart.

Effective training is feasible in which the students have varying levels of education and experience. Students prefer a curriculum heavy on hands-on laboratory experience and light on formal lecture. Video tapes are not well received for test demonstration purposes and slide presentations get only marginal acceptance. Live demonstrations and presentations are greatly preferred by most participants.

Instructors having diverse backgrounds and proven field experience are important. However, expertise, experience, and knowledge of the subject alone are not sufficient to make an instructor well received by the students. Instructors must have the ability to speak publicly, remain on the subject being presented, and not run over the allotted time. Instructors should not be used for extended periods of instruction or laboratory work because a variety of personalities and personal demeanors seems to keep the interest level of the participants higher.

Only time will tell the impact of the ODOT QC–QA training program on the quality of highway construction in Oklahoma. Similarly, the impact of the QC–QA specifications on the highway construction industry has yet to be determined. However, a great deal of personal and professional satisfaction has been derived by a substantial number of individuals from academia, state and federal governments, and the private construction industry who were responsible for developing and initiating this program to improve the quality of highway construction projects in Oklahoma.

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Unique Methods Used in Constructing the Robert E. Lee Bridge

THOMAS P. MCCARTHY, RALPH SALAMIE, AND W. R. NASH

The Robert E. Lee Bridge in Richmond, Virginia, consists of twin trapezoidal concrete box structures 3,400 ft long and consisting of constant-width and varying-width cross sections placed with form travelers. This bridge represents two firsts in North America: (a) the largest number of form travelers (eight) ever used concurrently, and (b) the first use of a strand stability system in segmental concrete balanced cantilever construction. The use of portable fabric cofferdams was a unique method for constructing the bridge piers in the James River. These fabric cofferdams provided a safe, environmentally sound structure within which to drill and blast for rock excavation and to place concrete for the pier footing and the first lift of the pier columns in the river.

The existing Robert E. Lee Bridge is a concrete spandrel arch structure constructed in 1935. Functionally obsolete, it had four traffic lanes and a load limit of 20 tons. Starting from the south, both bridges (new and old) span the Southern Railroad's main yard (20 tracks), the south channel of the James River, Belle Island, the north channel of the James River, C&O Railroad's track lines (two each), two city streets, and the historic Kanawha Canal. The upstream drainage area of the James River at Richmond exceeds 3,500 mi^2.

The new Robert E. Lee Bridge is a twin-trapezoidal box girder structure 3,400 ft long. The replacement structure provides two independent bridges (one northbound and one southbound). Each bridge has three 12-ft lanes, a 10-ft bicycle lane, and a 4-ft sidewalk. The bridge rises vertically 20.5 ft from its south to its north abutment. The new bridge's trapezoidal box girder cross section varies in width and depth.

SUBSTRUCTURE

The majority of the bridge foundations are spread footings placed directly on the underlying granite rock. The concrete footings in the river were to be keyed into the granite river bottom. These river footings required drilling and blasting of the bedrock to excavate to the required plan depth. Weathered granite was found in some footings, and the plan depths were lowered to provide bearing on competent rock. Concrete for the footings was specified to be 3,000 psi.

Fabric Cofferdams

The foundations for the river piers posed special problems. The granite river bottom was composed of an irregular solid granite surface with erosion-cut boulders, cobbles, and some sand. This configuration precluded the use of sheetpiling, because the overlying soil (sand, clay, and silt) was not thick enough to anchor sheetpile. A normal river stage provided water depths between 2 and 10 ft. Rigid environmental restrictions concerning erodable material in the river made any fill-type cofferdam unacceptable.

A common British concept for cofferdams, not yet applied in the United States, was used for the main channel river footings (Figure 1). A fabric cofferdam was installed in the river to provide a work area around each pair of footings, one northbound lane (NBL) and one southbound lane (SBL). The cofferdam consisted of steel frames and a rugged polyvinyl material draped from the frame to the river bottom. After the survey location was determined, divers installed the cofferdam in the following sequence of operations:

1. Cleaned the river bottom of cobbles along the frame lines.
2. Installed the steel frames, which interlocked and rigidly held the fabric. A hydraulic crane on the causeway helped rig the frames to the water. Typical spacing for the frames was 2 ft.
3. Installed the fabric. The fabric, in sheets 30 × 50 ft, was attached to the outside of the frame and rolled down the sloped outward leg to the river bottom and then along the river bottom. The divers weighted this fabric to the river bottom with a chain fixed to the end of the fabric. The fabric was overlapped and connected along the vertical fabric joints with velcro-type fasteners.

An 8-in. diameter discharge hydraulic pump was used for the initial dewatering of the cofferdam. After the fabric was in place and a pressure differential established, a minimal inflow leakage was sustained between the river bottom and the fabric. Leakage from the fabric was channeled by sandbags to sumps. Once water in the portadam was pumped down, a 6-in. hydraulic pump adequately handled cofferdam seepage. The three main pier cofferdams averaged 200 ft wide by 75 ft long.

The rock excavation required drilling and blasting operations. Water gel explosives and nonelectric blasting caps were used. The portadam was partially flooded before shooting the footing to help absorb the concussion from the blast (Figure 2).

On completion of one pair of footings, the cofferdam was quickly and easily removed and reinstalled. Typical installation time was 4 weeks—3 weeks of single shift work and 1 week for dismantling. The fabric cofferdam was environmentally advantageous in that it eliminated the use of steel boxes

McCarthy Brothers Company, 1341 N. Rock Hill Road, St. Louis, Mo. 63124.

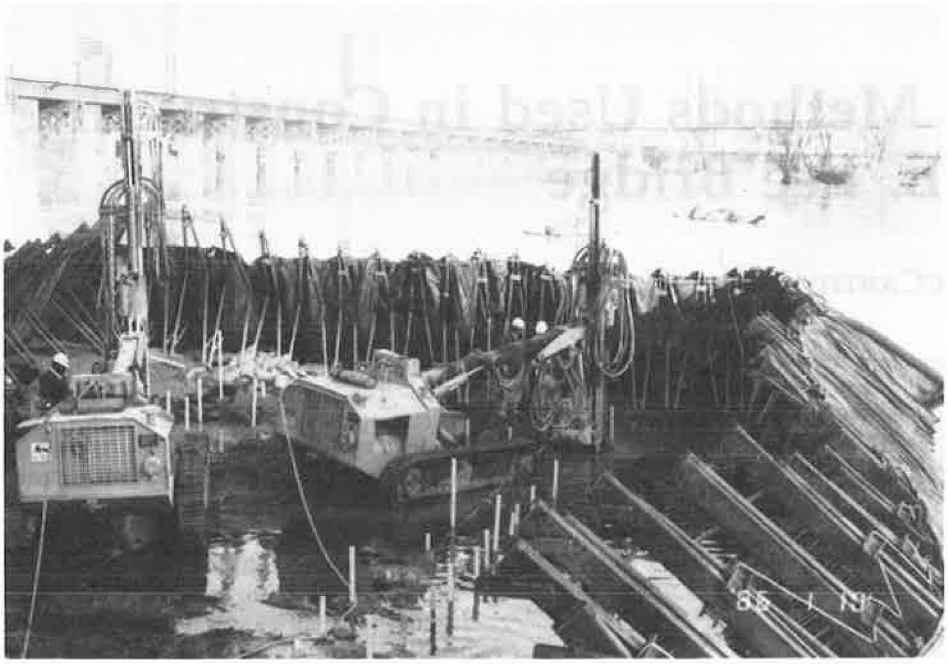

FIGURE 1 Fabric cofferdam drilling of rock excavation.

FIGURE 2 Fabric cofferdam on steel frames after rock blasting.

with tremie concrete, timber cribs, impervious fill, and other such materials (Figure 3).

The working limitation of the fabric cofferdam is a 9-ft head of water. The portadam was overtopped on two occasions during construction. The worst flood in 25 years caused irreparable damage to the frames and fabric.

Piers

The pier columns were octagonal in shape and ranged in height to 100 ft. Concrete for the piers was 4,500 psi and was placed either by pump or crane. Steel-plate girder concrete forms were used to form the piers.

FIGURE 3 Placing of pier concrete in footing with protection of cofferdam.

Bearings

Pot bearings were specified for the three ramps and main bridge to allow free rotation at the piers. Guided pot bearings were specified at four locations to allow for expansion and contraction in the longitudinal direction only.

Pier Tables

The pier tables (the superstructure section cast at each pier to permit erection of the form travelers) were constructed by using falsework supported by steel brackets posttensioned to the piers with threadbars (Figure 4). Wood forms were used to build the pier tables. Concrete was placed in three stages—the bottom slab, the walls, and then the top slab. A typical two-web pier table required 243 yd^3 of concrete; a three-web pier table required 436 yd^3. Superstructure concrete strength was specified to be 5,000 psi.

SUPERSTRUCTURE

The main bridge consists of two 15-span structures separated by a gap ranging from 1 in. to 9 ft. The original superstructure construction method called for Span 1 to be constructed on falsework, Spans 2 through 8 to be constructed by the cast-in-place balanced-cantilever method, and Spans 9 through 15 to be constructed on falsework. The three on-and-off ramps were designed to be built on falsework.

After studying the terrain and construction obstacles under Spans 9 through 15, a decision was made to use balanced-cantilever construction methods. The major concern was that falsework under Spans 9 through 12 would be positioned in the highly volatile James River. The consequences of damage to the falsework during flooding was unacceptable. A balanced-cantilever system would transmit all construction loads through the permanent piers, eliminating the need for intermediate supports between piers.

The configuration and design of the mainlane bridge superstructure presented challenges to construction by the balanced-cantilever method. The first order of work was to design a stability system for the slender 9-ft-thick pier stems that would be capable of carrying out-of-balance moments created by cantilevered construction.

Stability System

Conventional methods used to resist out-of-balance moments during cantilever construction are either brackets fixed to the top of the pier or props from the pier footing (or another foundation) to the box girder soffit.

A bracket system was studied, but the cross section of the piers did not provide a practical means of anchoring the brackets to the piers. The slender cross section of the piers did not provide enough room to lock the connection between the pier and the bottom soffit of the box. In addition, steel was congested in the top of the pier from rebars, pot-bearing anchor bolts, and surrounding spiral reinforcement.

A prop-type stability system was ruled out for two reasons. First, the props would have to support the cantilever at a height of 100 ft if they were to be reused throughout the structure. This required prop sections to be added and subtracted to accommodate the varying girder heights. Secondly, the props could not be used at the 10 piers in the river.

FIGURE 4 Pier table (two-web) with brackets awaiting traveler erection.

Exposing the props to forces from the James River during flooding was an unacceptable risk.

The selected stability system incorporated brackets designed to act as pier-table falsework and yet resist the out-of-balance moment resulting from the first segment placement. The pier-table moments were much smaller than the maximum moments produced by the full cantilever. Before the second segment was cast, brackets were replaced by routing the piers through a duct to its anchor plate (Figure 5). The upper anchorage was made in a concrete wall blister that was added and post-tensioned to the web, inside the box at a distance of 28 ft from the centerline of the pier. With completion of the last cantilever, using multistrand stays, the system worked without failure, despite some obstacles.

The stability tie system was costly in time and money. Similar to a prop setup, two separate systems were fabricated and erected. Special tapered concrete shoes were fixed to the faces of the pier-table stability brackets, which mount on a tapered face of the pier. Concrete anchor blocks were required at both the live and dead ends of the ties. The strands and wedge plates could not be reused. Casting the anchor blocks and replacing the brackets with ties took 5 days on a two-web cantilever section and 9 days on a three-web section (Figure 6).

With the stability tie system, the cantilever is not fixed against rotation, as it is with more conventional systems. Provisions were made to compensate for deflections of up to 24 in. at the cantilever tips caused by elongation of the ties that would allow the entire cantilever to pivot at the pier. These rotations were calculated before casting the first segment to ensure that, when the last segment was cast, the tips of the cantilever would be aligned for the closure segment. The rotation of the system added another variable to the camber analysis and increased the risk of actual tip location's not matching the theoretical position. Although the tips were easily adjusted by counterweighting at either end of the cantilever, this fix

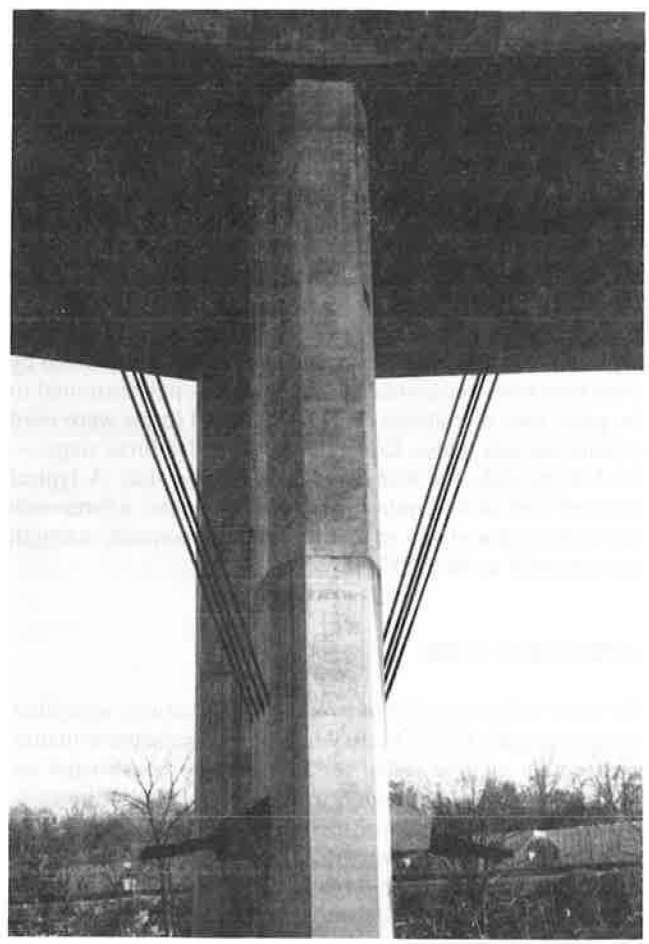

FIGURE 5 Stability strand ties restraining a three-web cantilever.

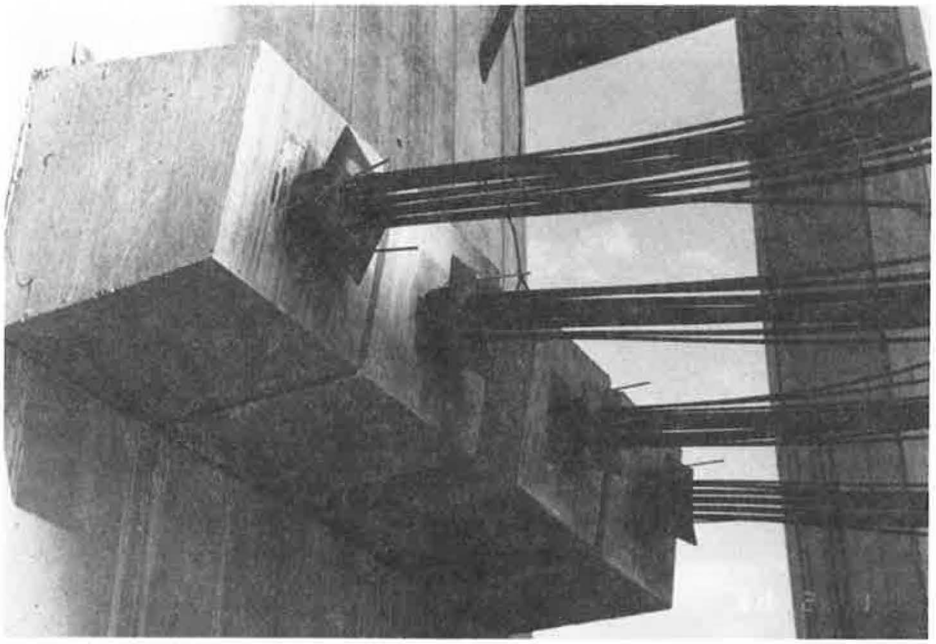

FIGURE 6 Lower anchorage of strand stability system.

was limited by the amount of reserve capacity in the ties. If rotations became excessive, the only answer was to discover the rotation problem in the early cantilever segments and replace the stability ties one side at a time to correct the rotation. This type of rotation fix was required on one cantilever.

The wedge-type anchoring system is not designed for repeated load cycles, so the risk of the failure because of strands slipping through the wedges was of constant concern. Care was taken to ensure that the strand, wedges, and wedge plates were free of any dirt and rust before installation. The ties were continually monitored for any signs of slippage. They were designed to have a minimum of 2.50 kips per strand at all times to keep the wedges seated in the anchor plate. The allowable load on the ties was set at 50 percent of their ulti-

FIGURE 7 Single-cell (two-web) box—bottom drive deck erection of A side.

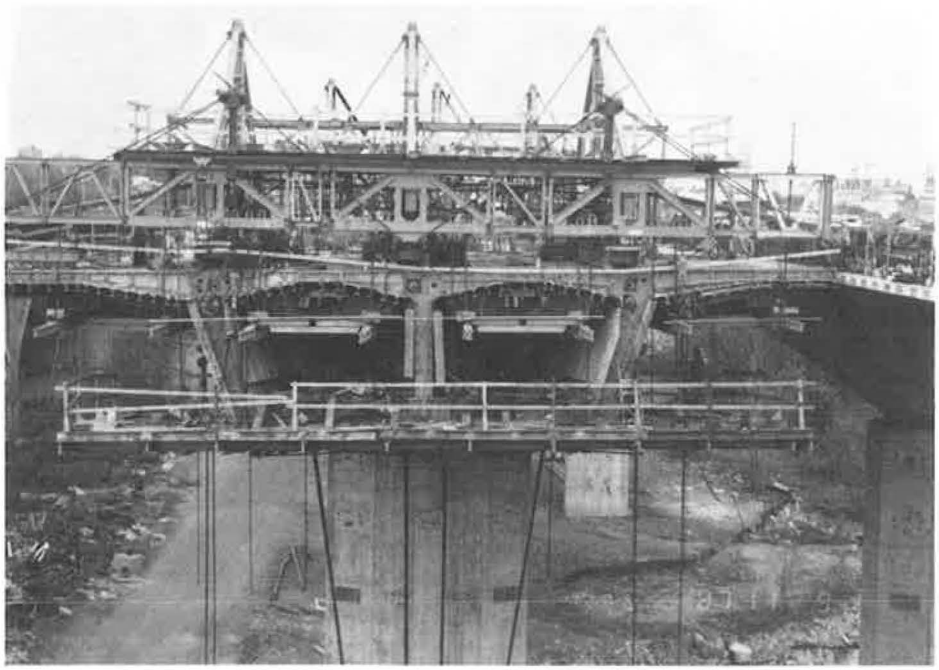

FIGURE 8 Double-cell (three-web) box construction.

mate strength. Some strand slippage did occur, and in an isolated single strand, slippage was corrected by restressing the strand with a monostrand ram. In two instances, the entire 19-strand stability ties were replaced because of multiple-strand slippage.

Travelers

The maximum out-of-balance moment was 44,861 ft-kips at Pier 10 NBL and the maximum rotation under this moment was 0.428° at Pier 2 NBL. Two pairs of constant-width form travelers and two pairs of variable-width form travelers were used to construct the balanced-cantilever portions of the bridge (Figures 7 and 8). The weight characteristics of these are as follows:

Type	*Traveler Weight (kips)*	*Segment Concrete Weight (kips, maximum)*
Constant width (two-web)	186.5	348
Variable-width (three-web)	279.75	488

Segment concrete was placed monolithically using a concrete mix with a specified strength of 5,000 psi. The mix design was modified by the contractor to use superplasticizer. Wood forms were chosen to provide the maximum flexibility in forming the varying soffit width and decreasing wall height of the segments.

A typical segment construction cycle duration was 1 week for the two-web travelers and 1½ weeks for the three-web travelers. Concurrent work was performed on both the A and B travelers on each pier.

CONCLUSIONS

1. The fabric cofferdams provided a safe, environmentally sound structure within which to excavate and place concrete for pier construction.
2. The inverted-stay stability system was chosen for the Robert E. Lee bridge because conventional systems were not practical. The stability ties worked without failure on 22 cantilevers.

ACKNOWLEDGMENTS

The McCarthy Brothers Company wishes to acknowledge the efforts of all site construction personnel, subcontractors, and others who contributed to the task of bridge construction. Special acknowledgments are due to the Virginia Department of Transportation; Hayes, Seay, Mattern, & Mattern (principal consultant and substructure designer); DRC Consultants (superstructure designer); Tony Gee + Quandel (construction engineer); and DSI International (posttensioning).

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Computer-Aided Construction Planning for Boston's Central Artery Project

BRIAN R. BRENNER

Some new computer applications are being developed for and applied to the construction planning and traffic studies associated with the design of the Central Artery and Third Harbor Tunnel in Boston. This project is one of the most massive and complex urban highway projects in the country. It features an immersed-tube harbor tunnel, depression of the downtown expressway in a cut-and-cover tunnel without interrupting service on the existing overhead viaduct, major new or revised urban interchanges, and long-span river crossings. The construction site, with its old existing and historic buildings and subways, permits limited clearances, posing formidable challenges to engineers planning the work. The computer applications described include computerized composite mapping for archeological planning; analysis of multilevel curved-girder viaducts; transportation forecasting using highway network models; planning associated with the construction of the depressed Central Artery tunnels; and computer-aided planning and mitigation measures for underpinning, protection of a subway tube, and other construction problems.

The Central Artery and Third Harbor Tunnel (CA/THT) project is a massive urban highway improvement job being planned and designed in Boston. A key map is shown in Figure 1. The work features the addition of a third harbor tunnel, a new immersed tube tunnel that will cross Boston Harbor and extend I-90 to Logan Airport and Route 1A. The six-lane Seaport Access Road will connect this tunnel to the end of the Massachusetts Turnpike. In downtown Boston, the existing elevated Central Artery expressway (I-93) will be depressed and widened in a cut-and-cover tunnel. In between these major components of the project, engineers are planning four new or extensively redesigned expressway interchanges, two long-span river bridges, and an assortment of underpinning and construction mitigation measures. Automated analysis and computer-aided design and development (CADD) methods are playing a major role in the project design. Some applications related to its construction planning and traffic mitigation are described.

ARCHEOLOGICAL PLANNING: THE OLD AND THE NEW

In the study of archeology, digging is customary in Egypt or Persia. Downtown Boston is not a site that usually comes to mind. Nevertheless, construction of Boston's CA/THT project will involve miles of excavation in land that has some of the oldest settlements in the country. The potential for archeological discovery along the new highway's alignment is enormous, although not at every site. Project archeologists have had to select their sites carefully before heavy construction damages what records remain underground. Computerized composite mapping is assisting this effort.

Bechtel/Parsons-Brinckerhoff, 1 South Station, Boston, Mass. 02110.

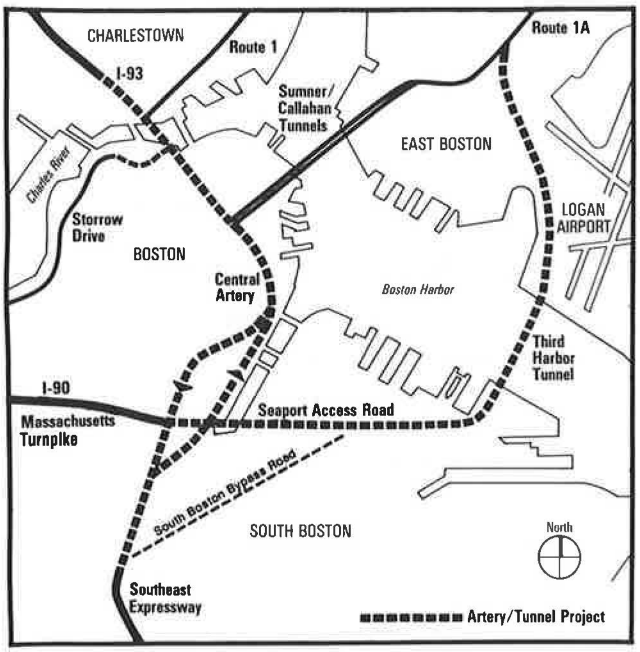

FIGURE 1 Key map, CA/THT project.

From experience, archeologists know that several factors can reduce the potential for significant finds at an excavation. In Boston, the first and most obvious factor is whether or not the site was on land during the earlier periods. Downtown Boston today is constructed mostly on landfill. Figure 2 shows the original shore outline of Boston superimposed on today's shoreline. In the 1700s, Boston occupied a jutting peninsula with a narrow neck. The "One if by land, two if by sea" story helps in understanding the limited choice when looking at this map. The area around the Back Bay really was a bay at that time, and most of the Sound End was under water. During the decades that followed, new land was formed by landfills and construction of wharves, with sedimentation. Although portions of colonial wharves might be worthwhile finds, unfilled harbor areas probably are not.

The alignment of the depressed Central Artery indicates that the planned highway clearly crosses an area that is land today but was largely in Boston Harbor for much of its past. The longer a site was underwater, the less likely anything of value will be found during an archeological excavation. The

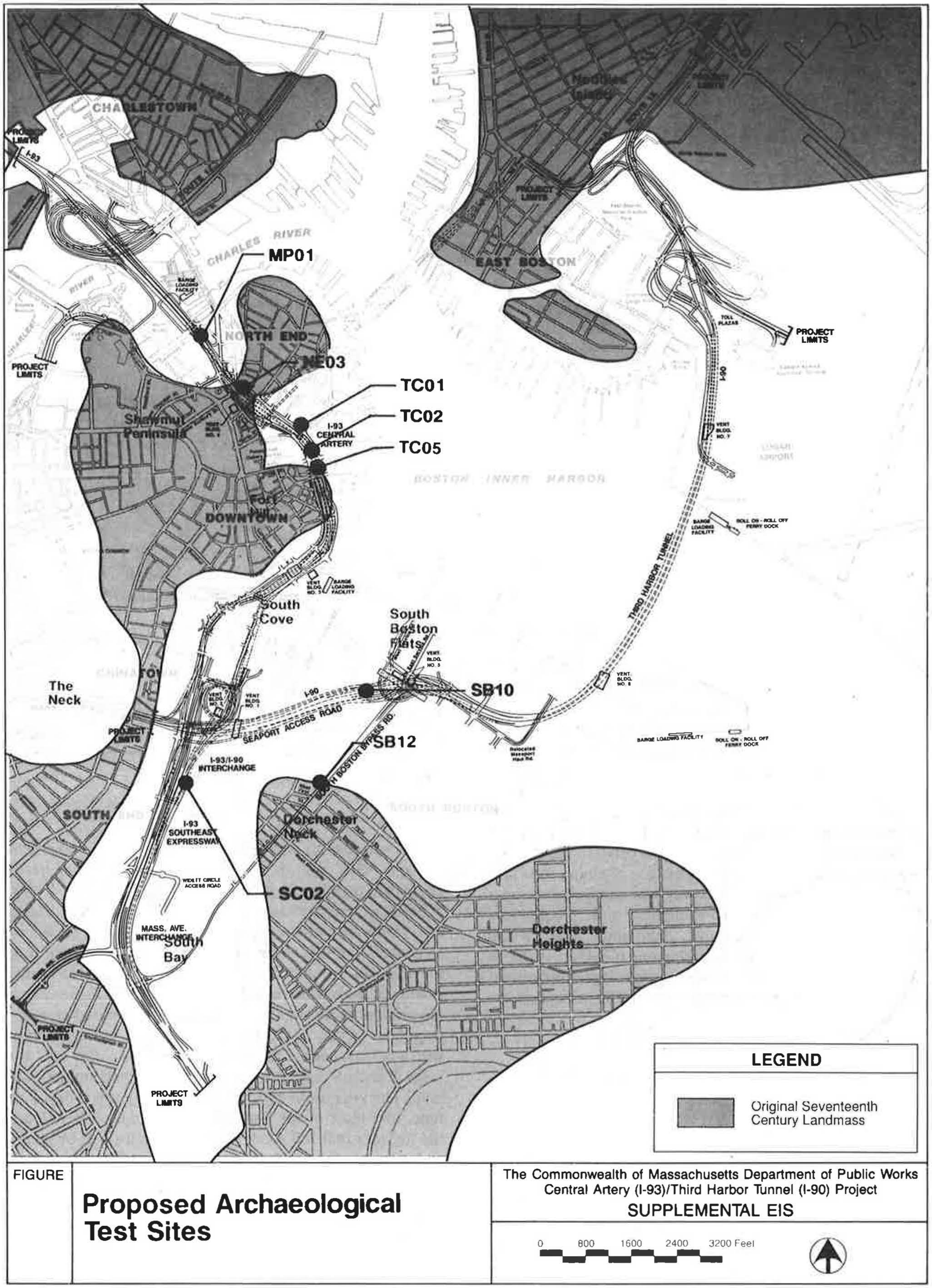

FIGURE 2 Boston shore outlines.

highway will cross what was a neck of original, unfilled land in the North End (at the point *NE03* in Figure 2). This area would seem to have better potential for archeological exploration than the other areas, which were out in the harbor until 1850 to 1860.

In the archeological probability game, modern civilization is another factor to deal with in selecting sites. Areas that may have had excellent underground deposits can be despoiled by subsequent construction. Boston is a continuously settled and densely populated urban area, and the digging required for building basements often will scuttle a possible site.

To make things more complicated, consider the effect of utility installation. Use of this form of underground construction has a long history in downtown Boston. The installation process is anathema to archeological exploration.

Considering all of these effects and impacts, project archeologists prepared composite maps by computer for the various areas of the project. Figure 3 shows a sample with a plan of 1929 basement footprints. Using the composite mapping, the archeologists selected sites with the greatest potential for finds. One prospective site rests on the original Boston neck near the North End (indicated by point *NE03*, Figure 2). The site is located in what was an airwell between three 19th century buildings. It is away from the path of water, sewer, gas, and power lines. Conveniently enough, today the site is a parking lot beneath the existing elevated Central Artery expressway.

Digging at this site proceeded last fall. Interesting finds include the well-preserved remains of a wharf that jutted out into the adjacent Mill Cove and an assortment of ceramic pieces representative of early American colonial life. Composite mapping for the archeological planning was set up on a NASA satellite mapping program. This approach was workable, but it had some drawbacks. One key problem was that the software lacked adequate screen-editing capability. The user interface for this program was too cumbersome to handle frequent editing of the maps. Another problem was getting all the old maps in the same coordinates. Predicting the location of juicy excavation sites would have been considerably easier if archeologists could find old landmarks with certainty. However, satellite mapping, and even good surveying information, are fairly recent innovations. A big part of the challenge in this effort was to get the old shoreline, locations of now-demolished buildings, locations of utilities (past and present), and the position of the highway all in line on the same map. Without clear-cut information, this feat required considerable judgment and educated guesswork. Project archeologists attempted to locate landmark points, constantly adjusting the maps to achieve a better fit with the available information. Having the maps computerized simplified this procedure considerably. The mechanics of trying to adjust the overlays manually would have precluded much of the trial-and-error adjustment that went into the successful archeological planning work.

CURVING MULTILEVEL RAMPS: FROM THE GROUND UP

Subsequent iterations of the revised intersection of Routes 1 and 93 in the project north area have featured increasingly complex ramp layouts. The selected alignment features curving exit ramps with as many as five levels on some support bents. For preliminary design, project engineers relied heavily on computer-aided analysis of the ramp superstructures and foundations. Ramp model geometry generators were created to save time and to help keep up with the civil layout iterations.

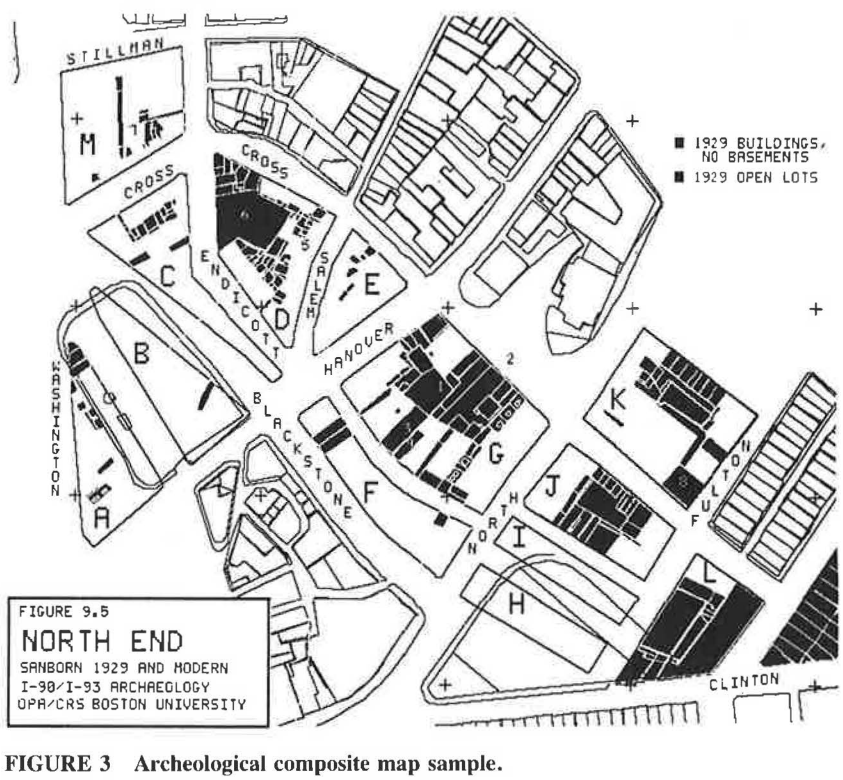

FIGURE 3 Archeological composite map sample.

Ramp plans were mathematized using the program COGOPC. From a mathematized baseline, framing for girders, diaphragms, and support bents was located. A plot of a sample layout is shown in Figure 4. Without a plotter connected directly to the PC programs, the COGOPC plan had to be translated to International Graphics Exchange System (IGES) format, a standard computer graphical syntax that many CADD programs recognized. The drawing was then loaded up using the CADD package, Graphics Design System (GDS), on the project VAX minicomputer.

To design the column support frames, structural models were generated for the structural analysis package, STAAD III. A bent model generator was programmed in BASIC language. The trick was to take the geometry already available from the COGOPC runs and automatically translate it to a form digestible by STAAD III. For each ramp level, mathematized coordinates and framing information was sent to ASCII files, standard computer text files that are conducive to translation for use in different software packages. The bent model generator program then used these files to assemble a three-dimensional STAAD III frame model with node coordinates, member numbering, and command syntax all in the proper form for STAAD III.

In a highly iterative design process, with viaduct layouts changing on an almost daily basis, the advantages of this type of approach are obvious. Mathematizing the geometry of the bridges at the preliminary design level allowed quick and accurate adjustment of the layout design. A structural model generator custom-tailored to this design permitted quick regeneration of the computer analysis and testing of each new scheme. These computerized tools saved hours of manually performed labor and even made possible a level of design effort that otherwise would have been impossible at this time.

One of the first goals in preliminary design was to size some column footings for the enormous curved girder bents. The STAAD III models returned axial loads and biaxial moments at the base of the support frames, using a variety of AASHTO loading conditions. The footings were to feature pile groups, a simple but tedious design problem. To speed the pile group selection, some Lotus 123 spreadsheet templates were developed. Part of template printout is shown in Figure 5. The

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- BECHTEL/PARSONS BRINCKERHOFF     CENTRAL ARTERY ENGINEERING -
- PILE GROUP ANALYSIS SPREADSHEET       BY:                   -
- DATE:   03/19 /88                CHECKED:                   -

- FILE:TRB_SAMP           SUBJECT:  SAMPLE FOR TRB PAPER      -

SUMMARY DIAGRAM

                                                                     ROW
:ON       ON       ON        ON        ON        ON       ON     :    SPACING
:170.6    144.8    118.9     106       118.9     144.8    170.6  : 1  -ON--
:                                                                :     5
:159.4    133.5    107.7     94.75     107.7     133.5    159.4  : 2  -ON--
:                                                                :     5
:148.2    122.3    96.45     83.52     96.45     122.3    148.2  : 3  -ON--
:                           -------------                        :     5
:136.9    111.1    85.22  :             :85.22   111.1    136.9  : 4  -ON--
:                           -------------                        :     5
:148.2    122.3    96.45     83.52     96.45     122.3    148.2  : 5  -ON--
:                                                                :     5
:159.4    133.5    107.7     94.75     107.7     133.5    159.4  : 6  -ON--
:                                                                :     5
:170.6    144.8    118.9     106       118.9     144.8    170.6  : 7  -ON--
:
:
 A        B        C         D         E         F        G        P:  3470 :
 SPAC:  5          5         3         3         5        5       MX: 11006 :
                                                                  MY: 15840 :
```

Above is a summary sheet for a layout of piles. The group was analyzed for moments and axial force applied at the centroid. Numbers above are calculated pile loads, in kips. The geometry for this group is identified by row and column spacings for the piles in plan, shown above in feet. Notice that the center of the summary is blank. This arrangement does not include a pile at the center.

FIGURE 5 Spreadsheet template used for pile group analysis.

spreadsheets featured preprogrammed menus and design screens. To operate a sheet, designers entered such parameters as the design loads and moments and a trial footing and pile group layout. With this information, the design screen automatically returned pile group analysis results. Using the menus, the geometry could be quickly adjusted to achieve the optimum design.

The spreadsheets reduced the iteration time of the pile group analysis from hours to minutes. As an added benefit, the sheets could be written in such a way as to look like manually written calculations. These calculation pages could be checked easily, avoiding the computer black-box problem.

For the curved ramp superstructures, some alternatives involved the use of steel box sections. Accurate accounting of the structural effects of curvature on the box sections was

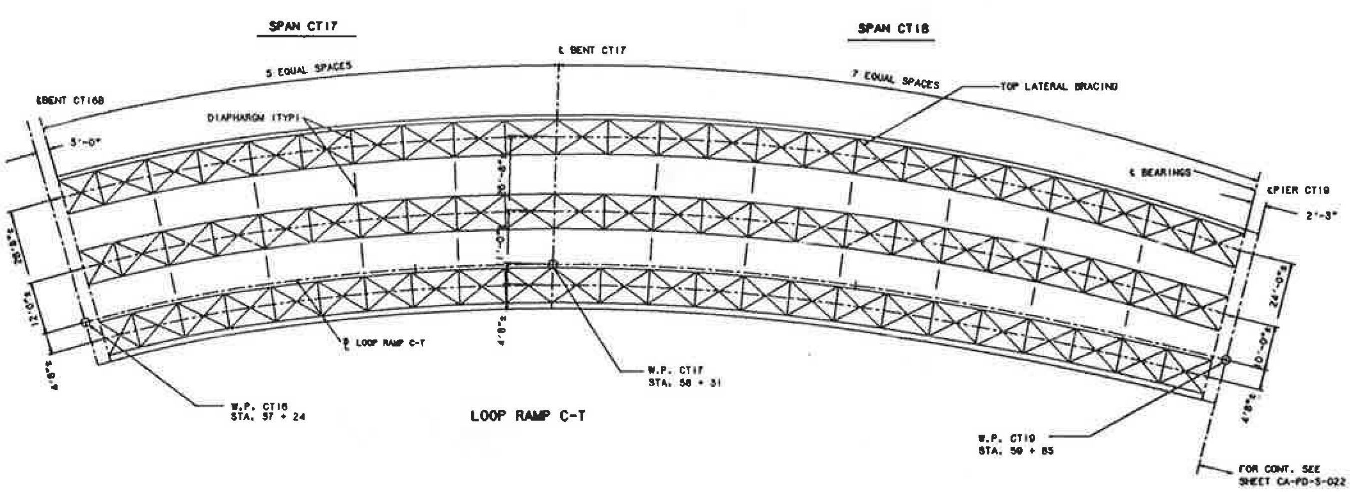

FIGURE 4 Highway framing layout plan.

beyond the capabilities of the STAAD analysis. To further study this problem, project engineers used Bridge Software Development International's system, BSDI–3D. This software has, at its core, a sophisticated finite element module. Other parts of the program preprocess the bridge layout and then enable the engineer to iteratively size the section once the analysis is run. Although the central module is too large to fit on a PC, the preprocessors and postprocessors are operated from a PC, giving the bridge designer access to the power of mainframe analysis and the flexibility of PC data processing.

BSDI had some drawbacks for this use. Because of the way the software is currently configured, performing several analyses of a structure is not practical. This limitation means that, although the design module can be used to iteratively size a section, the iteration is based on analysis results from the first trial. Results are not automatically updated for each trial section. Therefore, the first guess must be a good one. Also, the preprocessor is somewhat limited in the range of structures it can handle. The bents to be analyzed were unusual in that the support box frames tie directly into the curving box superstructure. The effort to isolate the superstructure for this detailed computer analysis was slightly unrealistic because, in actuality, members of the entire multilevel structure behave interactively. To focus on the box beams alone, special spring support conditions must be incorporated into the computer model. This was possible with direct help from the support staff from BSDI but not through the preprocessors.

TRANSPORTATION PLANNING: BREAKING UP THE BACKUPS

Imagine, for a moment, a society in which all transport is planned. In this unusual world, planners know beforehand where each person will go, what route that person will take, and what vehicle will be used. Of course, events like car breakdowns and meteors landing on transit facilities would be foreseen. This transportation planning utopia also features some database software that can store, digest, and interpret the incredible load of information and then spit it back out in some meaningful form. Under these conditions, each transit facility could be perfectly planned and designed. In such a utopia, subways are just large enough at rush hour (the air conditioners always work, and everyone has a seat!), and highways all feature Level-of-Service A at all times.

Returning to the real world, project transportation planners have had to solve difficult problems that do not work out as neatly as in the scenario just described. Sophisticated software is helping to forecast Boston's transportation demand in the next century, and it is assisting the effort to fine-tune alignments to maximize the results of transportation project spending.

Early planning efforts went into modeling the performance of new expressways. Planners used FREQ8, a freeway modeling package from the University of California at Berkeley. For input to this software, a highway is modeled as a series of segments. The definition of the segments depends on geometric characteristics such as number of lanes, entrance and exit ramps, profile, and plan curvature. Each segment is chosen to have similar characteristics. The freeway model is then loaded with projected trip demands. An origin-destination table can be explicitly entered, or it can be generated synthetically by the program. The traffic demand is input in chunks of time, typically 15-min intervals. On the basis of this information input, the computer algorithm simulates the performance of the expressway with time. If a segment, with its associated penalties for excessive curvature, limited lane capacity, or weaving problems, is overloaded, a computerized queue forms, and this backup is sent to the next time chunk. In this way, one can almost watch traffic jams forming during rush hour, only to dissipate when the traffic demand diminishes.

Although quite powerful, FREQ8 is not particularly user-friendly. Preprocessors and postprocessors were written to dissipate the FREQ8 run backup. For data input, the current version uses the PC version of the old FORTRAN punched-card method. In the input file, all data must line up by column and position, a nuisance in the age of menu-driven software. Also, feeding trip demand data to the program for all the time chunks can be tedious, especially when you have to extrapolate from hourly data anyway (typically the case). Lotus 123 spreadsheets were written to handle the extrapolation automatically. Results from the sheet were sent to an ASCII file, which was read into a preprocessor program written in BASIC. This program created a FREQ8 input file, with all the variables in the right place.

FREQ8 output is useful, featuring tables displaying numerical ratings of freeway segments during the model period and other information. The volume of output is toggled by input variables, which control the various degrees of detail the program is to prepare. To make the output easier to digest, BASIC postprocessors were written. These programs read in output files and split them up for easier handling. One feature of the postprocessor read in a numerical freeway performance table and translated it to a color plot on the screen. Segments with modeled performance speeds of 10 mph or less were displayed in red. The screen plot helped to illustrate quickly which areas of a particular model were failing and at what times during the model run.

Because highway design is an iterative process, different expressway layouts could be tested and fine-tuned on the basis of the model results. FREQ8 runs indicated, for example, that some severe weaving distances between ramps in the depressed artery could be expected to cause the highway to fail during evening rush hours. In part, on the basis of this assessment, the decision was made not to use a series of highway layouts featuring this unacceptable weave. The combination of FREQ8 and our supplemental software proved to be a valuable tool in tracking down alignment problems and optimizing the design.

The FREQ8 model is effective, but it is limited in its evaluation of the overall transportation picture. The depressed Central Artery, the new Third Harbor Tunnel, the Seaport Access Road, and all of the interchanges in between form only a part of the complex transportation grid that Bostonians rely on to move around in the metropolitan area. Project planners also needed to develop models to predict the way the transportation grid as a whole would behave, given the highway improvements. This computer job was accomplished with TRANPLAN. This software was used to develop an elaborate grid representative of Boston's city streets with the addition of the new links. The models were loaded up with traffic demand forecasts by the Commonwealth of Massachusetts Central Transportation Planning Staff.

With street geometry and traffic demand electronically developed by TRANPLAN, elaborate what-if scenarios were tested for the highway network as a whole. The results for the various model runs were used to move exit ramp locations, to propose the modification of some city boulevards (in some cases, changing the direction of one-way streets). The data were also used extensively in project environmental reports to predict quantities such as air quality.

TRANPLAN was operated on Compaq 386 personal computers. To develop the node and link map, planners used the NEDS program, which has a screen editor for the network map.

TRANPLAN is an unwieldy program, but project planners tried to tame it with an array of preprocessor and postprocessor programs, developed in-house. These programs were written in C and dBASE programming codes. The processors read ASCII files produced by TRANPLAN and, for example, created a database for dBASE III. Once the data were in dBASE, the options for processing multiplied rapidly. Sorting information and preparing reports for various conditions and project locations was easy. The processor programs also featured elaborate customized screens to simplify the work.

AUTOMATED CUT-AND-COVER TUNNELS: FROM THE GROUND DOWN

Much of the planned CA/THT construction will be underground. In fact, the project features miles of cut-and-cover highway tunnels, some with unusually large design loads and dimensions. In the central area of the project, tunnels will be constructed underneath the existing Central Artery viaduct while this expressway remains in service. In addition to the usual tunneling concerns, a massive underpinning job in this area will be required.

Computers are helping with several aspects of the preliminary design. A variety of automated tools have been developed to help the design effort. Figure 6 shows a series of tunnel section drawings. The sections were developed in CADD at every 100 ft along the depressed I-93 alignment. A quick review of these sections demonstrates the complexity of the planned underground work, especially at the new interchange with the existing Calahan and Sumner cross-harbor tunnels. At this area, five ramp tunnels peel off the mainline and go in various directions.

The tunnel sections were particularly useful for preliminary quantity estimates. Using GDS commands, engineers created graphic objects for tunnel walls, excavation zones, and other areas of interest. The computer then automatically calculated cross-sectional areas. These results were sent to a Lotus 123 spreadsheet for additional calculation and display. Tabulating the numbers in this way was particularly convenient for reporting on the results for portions of the alignment. Quantities were required for different design and construction contracts, which would have area boundaries at different stations. Using the computerized spreadsheet, the data could be manipulated easily in whatever fashion was required.

Tunneling design is a multidisciplinary task, perhaps to a greater degree than other civil engineering work. While the layout group is planning the highway, structural engineers must be sure that what is laid out in plan and profile can be designed. Ventilation and electrical engineers hover in the background, reserving room for their ductwork in each scheme. In an older, intensively developed area like downtown Boston, utility engineers will shoot down several preliminary concepts in their quest to keep the sewers and water lines flowing during and after construction. Construction engineers must be sure that any preliminary design can be built in the first place. Urban planners and architects are also involved, especially on a project like the Central Artery, which will create acres of developable urban land.

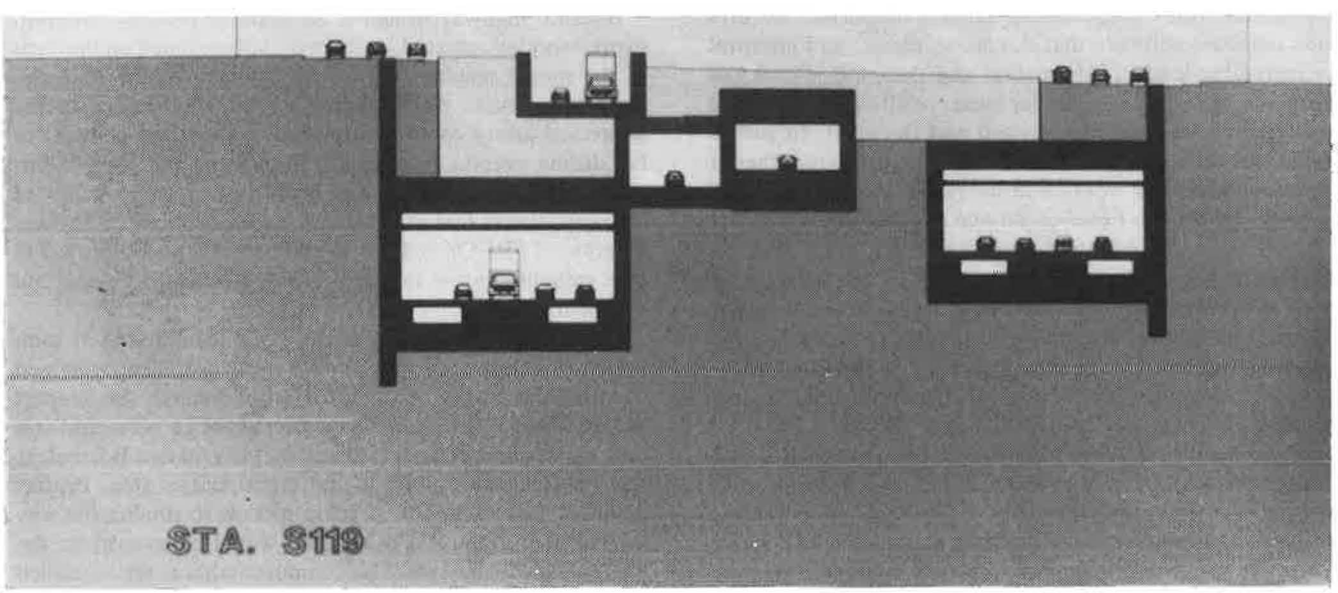

FIGURE 6 CADD tunnel sections.

For a tunneling job of this magnitude, a major problem with manual methods is the difficulty of moving tremendous quantities of information quickly to those designers who must see it. Moving a tunnel alignment might involve weeks of redrawing sections and accounting for all the pieces that make up this multidisciplinary design puzzle. A great advantage of automation is that it cuts down the iteration time. During preliminary design, new schemes can be accurately analyzed faster. For example, the station tunnel sections were started by structural engineers to help develop structural analysis models for the flying ramps. The urban planning group adapted the sections and developed a tunnel drive-through slide show, in which one section faded into the next as they were flashed on the screen. This display made an effective presentation for community meeting groups. The construction team used the material to develop staging plans.

CONSTRUCTION PLANNING: MITIGATING THE DIGGING

The task of planning the CA/THT construction is complex and iterative, two good motives for relying on computer-aided design. The work features difficult scheduling problems, traffic maintenance challenges, and construction next to sensitive older structures.

Construction of the I-93/I-90 project will require underpinning solutions of every kind used in construction. Perhaps the most difficult underpinning job will be building the newly depressed Central Artery in a tunnel under the existing expressway while it remains in service. This viaduct, built in the 1950s, has more than 70 bents that must be underpinned. Complicating the design task is the fact that the geometry of the existing bents and the planned tunnel, with its exit ramps, varies at each site. No one standard design will work for each bent because, in fact, each site is an exception to any standard detail.

Project engineers have considered several options, including extension of existing artery cross beams laterally, construction of new support bents, grade beams in steel and posttensioned concrete, and a scheme involving deep caissons. Because each site must be examined separately and checked with traffic maintenance schemes and staging plans, engineers must process a tremendous amount of data for each preliminary design.

Figure 7 shows a sample Lotus spreadsheet used for a series of underpinning options involving posttensioned grade beams. Many features of this analysis were automated. Loads from

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::::::::::::::::::::::::::::::::::::::::::::::::::::::::::::::::::::::::
:  BECHTEL/PARSONS BRINCKERHOFF              SUBJECT:                  :
:  CENTRAL ARTERY/ TUNNEL PROJECT            SAMPLE FOR TRB PAPER      :
::::::::::::::::::::::::::::::::::::::::::::::::::::::::::::::::::::::::
POST TENSION UNDERPINNING DESIGN  PROGRAM
BY: JOHN G. HARDING


SECTION PROPERTIES  ** INDICATES USER INPUT
================================
**DEPTH (in.)    h =        78
**WIDTH: (in.)     b=      126
**ECCENTRICITY:(in.)        30
**d : (in.)                 69


MATERIAL PROPERTIES                    COFFICIENTS
================================       =========
**f'c (psi.)              6000
**fy (ksi)                  60
**fu (ksi.)                270         **a  =      1.76 (SEE ACI HANDBOOK, FUNCTION OF fc & fs)
**fci (psi.)              4000
**Aps (in.^2)            0.215


LOADS (KIPS,FEET)                                  CALCULATIONS
=====================                              =========
                                                   IZ = b*h^3/12 =           4982796 in^4
**MOMENT (DL)                     22150 k '        IY = h*b^3/12 =          13002444 in^4
**MOMENT (LL + I)                 14407 k '        AREA  A = b*h =              9828 in^2
**MOMENT (SELFWEIGHT)           16244.5 k '        SZ = IZ/c =                127764 in^3
                                                   MT = MDL+MLL =              36557 k '
**SHEAR (LL + I)                   1332 k          VT = VDL+VLL =             3219.8 k
**SHEAR (DL)                     1887.8 k
```

FIGURE 7 Spreadsheet template used for underpinning analysis.

the existing artery were calculated by computer, and then the grade beam was analyzed by STAAD III. Input for STAAD was automatically generated by the customized preprocessor program, written in BASIC. From this analysis, results were imported into the posttensioning spreadsheet to study this grade beam option. In general, studies revealed that, in cases in which the new depressed artery will be high in profile, posttensioned grade beams may be a valid underpinning solution, because they can support the overhead viaduct and form the new tunnel roof in the same limited structural clearance. Difficulties in placing precast, posttensioned members, however, may preclude their use for other deeper sites.

Underpinning an expressway viaduct is difficult enough, but recent highway plans require that the existing expressway

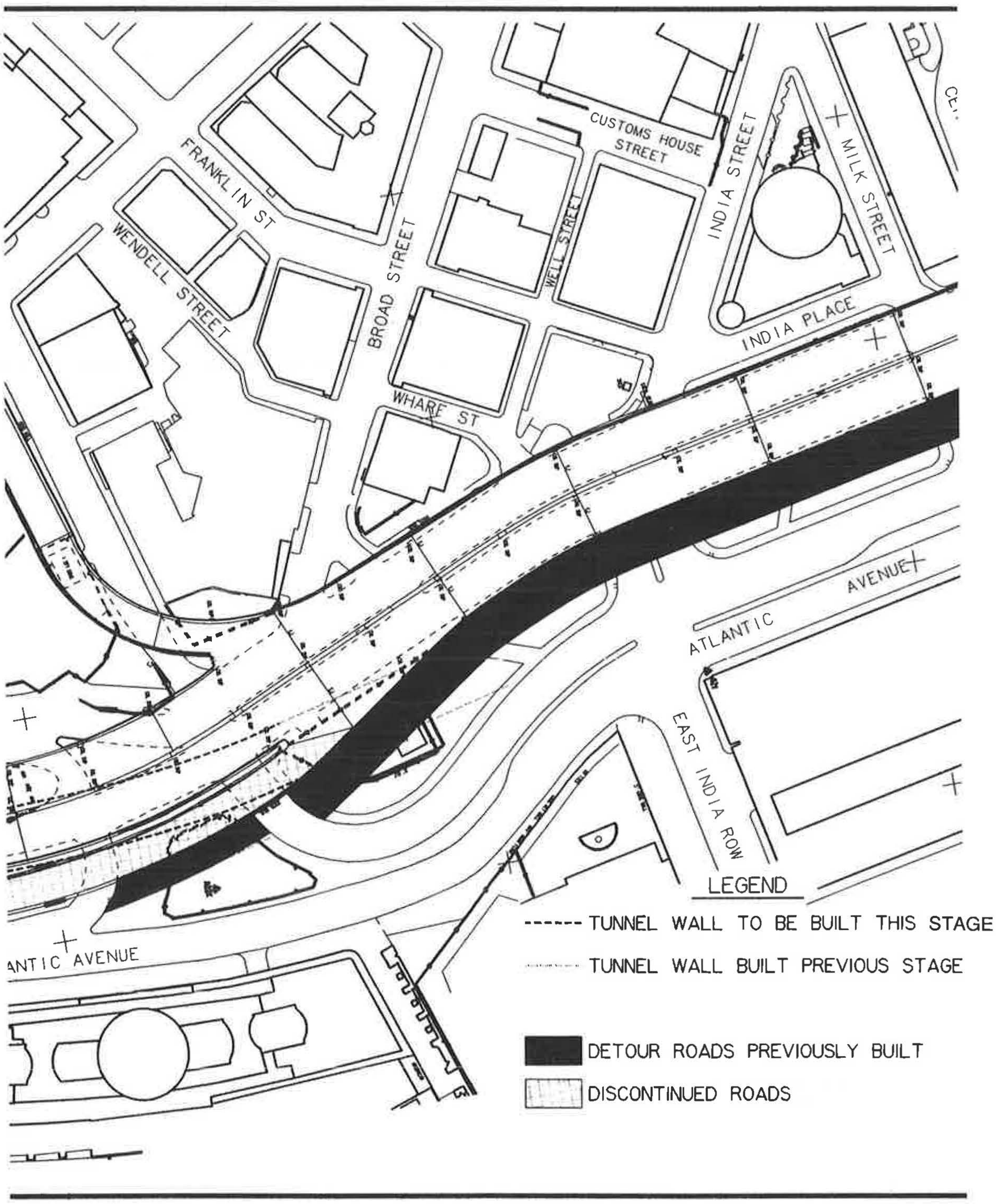

FIGURE 8 Sample of staging plan.

transition section also requires support during construction. This transition occurs when the viaduct drops to fill before diving into the Dewey Square Tunnel. The construction staging sequence for this task is elaborate, requiring several temporary structures to move traffic back and forth away from the area of construction. To assist with these studies, a series of staging plans was developed in CADD, using GDS (Figure 8). This process proved to be an efficient way to get at the problem. Because each stage is a variation on the previous one, the graphics were easily manipulated. Also, adjusting the staging plans to account for any one of a number of problems—utilities in the way, alignment changes, and so on—was fairly easy. The resulting drawings, plotted in color, compared well with what would have been the alternative—dozens of smudged and fading onion skins.

Included on the long list of vulnerable older structures that must be protected during CA/THT construction is a twin-tube, unreinforced concrete subway line built in 1914. This subway lies in the path of the Seaport Access Road, which must fit above it with only a few feet of clearance. To make matters worse, the whole site rests in 25 ft of water in the Fort Point channel. Subway service must not be interrupted. On the contrary, the subway will be relied on heavily to move people in and out of Boston once heavy construction begins.

Using the program SOILSTRUCT, a finite-element model was prepared for the subway tunnel section and surrounding soil mass. The software has several features useful for this complicated task, including inelastic properties for soil layers, interface elements to be placed at the boundary of the soil and a structure, and capability to model excavation. This last feature allowed the testing of various construction sequences and prediction of the effects on the vulnerable tunnel below.

Like many of the computer programs used on the project, the software was customized to handle specific design tasks. In this case, mesh generators were written in BASIC and PASCAL to produce the geometry and ASCII files required for SOILSTRUCT. After running the program, postprocessors helped to interpret the data. Some of these exported the data for plotting in AUTOCAD. For example, nodal movement could be plotted at exaggerated scale to illustrate quickly how the tunnel and surrounding soil mass behaved during excavation. Other processors sent SOILSTRUCT results to Lotus 123 for plots of node movement along particular lines of nodes and had other uses as well.

The finite-element soil-structure interaction analysis was not without pitfalls. One problem was that SOILSTRUCT appears to have difficulty modeling the behavior of soil in tension (that is, tension cracks). This problem led project engineers to be suspicious of some of the tunnel movement predictions. Also, even with customized preprocessors and postprocessors, the SOILSTRUCT results were somewhat difficult to verify and manipulate. For one series of runs, as an example, full pore pressure was mistakenly specified on the inside of the subway tunnel as well as the outside. In other words, the model was run predicting soil-structure behavior with a flooded subway tunnel. For a third problem, the analysis was only predicting two-dimensional behavior in section. In reality, the three-dimensional soil behavior of the actual planned excavation is much more complex. To effectively model this condition, with all three-dimensional anisotropic soil properties accounted for, is perhaps beyond the state of the art.

Even with the problems, the construction modeling was useful because it highlighted problems with the planned excavation at this difficult site. For now, project engineers are focusing on use of an immersed tube section to bridge across the top of the subway. This plan will avoid dewatering the site, therefore reducing the potential for unacceptable stresses on the subway tubes.

SUMMARY AND CONCLUSION

The following CA/THT computer applications were reviewed:

- Computerized composite mapping for archeological planning,
- A system of customized software applications for analysis of curved-girder bent structures,
- Transportation planning software used for modeling elaborate networks of Boston's streets and freeways,
- Cut-and-cover tunnel analysis for preliminary design via programmed spreadsheet and CADD sections, and
- Automated construction planning for underpinning highway viaduct bents, sections on fill, and protection of an existing subway tunnel.

Engineers for the CA/THT project are continuing to develop an array of sophisticated computer applications to assist the planning effort. The conceptual design work has not been without pitfalls, but all involved with the work agree that the computerized approach, tailored by engineers and planners to solve project problems, offers tremendous advantages over traditional manual methods. The automated design is not only more efficient but in several cases it also permitted the design team to study more options for problems than are usually studied under the given constraints.

Currently, the CA/THT project is well into the preliminary design phase. As the work progresses, the project staff intends to rely increasingly on a computerized approach to manage final design by section design consultants and to check their work.

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Decision Analysis Framework for Evaluating Highway Contractors

JEFFREY S. RUSSELL

Contractor prequalification is not an accepted practice among all state departments of transportation (DOTs). Nevertheless, numerous states have formal procedures that require highway contractors to be prequalified in addition to submitting a project bid bond. These prequalification procedures are based on state laws and were developed in an ad hoc manner. A new prequalification model was based on data collected from prequalification officials of state DOTs. The Indiana Department of Highways has applied the model to a bridge construction project for which five contractors are seeking prequalification. Recommendations are presented to facilitate the model's implementation into existing state DOT prequalification procedures.

Before permitting a highway contractor to bid for public transportation works, a review of qualifications is usually performed. This process is not agreed on by all state departments of transportation (DOTs). Consequently, many state DOTs (e.g., Alaska, California, and New York) do not have formalized contractor prequalification procedures but rely on the contractor's ability to secure a bond. Thus, contractor prequalification is performed by a surety company.

Articles regarding the prequalification of highway contractors appeared as early as 1939, when Harrison (*1*) wrote a proposed standard for the qualification and prequalification of contractors. In 1948, Nettleton (*2*) described experiences with prequalification, and the process was reviewed again in 1964 by Plummer (*3*). Corwin (*4*) presented an analysis of pre- versus post-qualification practices, and in 1970, Burrill and Poe (*5*) performed a survey of state prequalification procedures. The topic remains an important and controversial issue in the highway construction industry. Netherton (*6*) analyzed state prequalification procedures in 1978, and as recently as 1985, Nittany Engineers and Management Consultants (*7*) evaluated the prequalification procedures of six state DOTs.

States currently performing prequalification use a formula to determine the contractor's maximum financial capabilities (*6*). This calculated value, based on parameters from a financial statement, is used to establish the maximum aggregate amount of uncompleted work (in most cases both public and private) a contractor may have on hand at any one time. These formulas have been developed on an ad hoc basis and can take a variety of forms.

For example, Ohio's formula incorporates a contractor's net current asset value multiplied by a coefficient of 10 (Revised Code 5525.02–5525.09, State of Ohio, 1963). Other states, such as South Dakota, use net current assets minus net current liabilities plus 80 percent of the book value of the contractor's construction equipment multiplied by a coefficient of 10 (South Dakota Department of Transportation, Regulations 70:01:05:02, 1975). Net worth multiplied by a coefficient of 10 represents the maximum amount of work a contractor can have under contract in the state of Alabama (Code Title 46, 65–83, State of Alabama, 1975). The formula used in Iowa includes total net current assets minus current liabilities plus one-half of noncurrent assets minus noncurrent liabilities (Iowa State Highway Commission, 1972).

The formulas use financially based values that are often subjectively modified (i.e., reduced by other items perceived to impact them). For example, contractor safety, organization strength, past performance, personnel experience, and cooperation with the owner's representatives are subjectively modified items. The study by Nittany Engineers and Management Consultants (*7*) revealed that some formulas do not adequately measure the capacity of work a contractor can perform. Thus, a prequalification model based on a more scientific approach needs to be developed.

Results of the state DOTs' evaluation of the perceived effect that various factors and subfactors have on contractor prequalification decision making are presented in the following sections. The mean impact responses of the questionnaire items have been calculated and are provided. A statistical technique, factor analysis, applied to the collected data provides the basis for the creation of a computerized linear prequalification model. This model represents a decision template that can aid in performing the prequalification task. The model and an example application are described, and recommendations for using the model with states' existing prequalification procedures are presented.

QUESTIONNAIRE STRUCTURE AND RESULTS

A sample of 50 prequalification officials was compiled from the AASHTO 1987 Reference Book, and each official was mailed a questionnaire to complete. A total of 45 questionnaires were returned, representing an overall return rate of 90 percent, with five states (Connecticut, Maine, New Hampshire, New York, and Rhode Island) not responding. Of the returned questionnaires, 34 were complete; therefore, in terms of usable data the response rate was approximately 68 percent. The other 11 respondents (approximately 22 percent of the sample) indicated that they do not currently prequalify contractors. These states were Alaska, California, Idaho, Louisiana, Maryland, Minnesota, Mississippi, Missouri, Montana, New Mexico, and Oregon.

Department of Civil and Environmental Engineering, University of Wisconsin-Madison, Madison, Wis. 53706.

The questionnaire consisted of three parts: respondent classification (i.e., organization type), rating of the impact of major decision factors (DFs), and rating of the impact of decision subfactors used in prequalification decision making. The format, structure, DFs, and subfactors used in this questionnaire were based on 8 personal and 24 telephone interviews with construction professionals who currently perform prequalification (*8*).

The first part of the questionnaire (respondent classification) included the type of organization represented, type of construction activity typically engaged in, type of contracting strategy typically used on projects, and annual volume of construction. The questionnaire's second part contained 20 DFs relevant to contractor prequalification. These DFs were presented so that individuals could describe the impact of each DF on their decision process. The response alternatives ranged from 0 (little or no impact) to 4 (high impact), with 2 being moderate impact. The third part of the questionnaire contained 67 decision subfactors that could be used to make a more refined decision associated with a major factor. The questionnaire also requested information describing the respondent's prequalification process. A copy of the questionnaire can be obtained by contacting the author.

The contract method typically used by the sample was competitively bid unit price. The average annual construction volume was $375 million, with a standard deviation of $350 million and a range of $53 million (Hawaii) to $2,000 million (Texas). Only 41 of the respondents indicated their annual volume of construction work. The mean impact for each questionnaire item, presented in Table 1, was calculated to identify any trends in the data. The 10 DFs and subfactors with the highest and lowest mean impacts are presented in Tables 2 and 3, respectively.

Factor analysis was performed on the first 20 questionnaire items (major factors) using principal component factor analysis with varimax rotation. The theoretical description of this technique is presented in Harman (*9*). A reduced number of distinctive composite decision factors (CDFs) relevant in prequalification decision making were identified in the factor analysis procedure. Each CDF revealed through this analysis was assigned a descriptive name, on the basis of the nature of questionnaire items showing the highest correlations with the considered CDF. Because of the small number of transportation-related data points, other governmental agencies responding to the questionnaire were combined before performing this statistical technique, enlarging the sample by 25.

The CDFs presented in Table 4 include performance, type of contractor, capacity for assuming new projects, location, percentage of work performed, third-party evaluation, and financial capability. The correlation between the factor and a particular questionnaire item is defined as a loading factor. The questionnaire items with significant loading (greater than 0.4 out of 1.0) on a factor are presented in Table 4. Also shown is the standardized factor score determined by a regression procedure. The score or factor loading can be interpreted as being indicative of the contribution that the item makes to the determination of the factor's nature. The scores were standardized to facilitate a comparison across factors. The results presented in Table 4 represent the basis for the computerized prequalification decision support model described in the following section.

PREQUALIFICATION MODEL DESCRIPTION

The prequalification model, as described by Russell and Skibniewski (*10*), incorporates a dimensional weighting procedure. The model parameters are based on the factor analysis results of the questionnaire. Two terms are essential in the model: CDF and DF. A CDF represents a single underlying construct made up of interrelated decision factors. A DF is defined as a criterion that can be used to evaluate candidate contractors. An example using these terms is shown in Figure 1. As shown, the Type of Contractor CDF consists of the following DFs:

1. Experience. Amount of expertise a contractor has achieved within a certain class of projects (e.g., size and construction type);
2. Equipment Resources. Type, amount, availability, and suitability of construction equipment the contractor possesses; and
3. Company Organization. Type of ownership, management experience and leadership, and ability to respond to the owner's requests, among others.

The linear prequalification model is formalized by the following equation:

$$AR_k = \sum_{i=1}^{n} W_i \left[\sum_{j=1}^{m_i} (w_{ij})(R_{ijk}) \right] \tag{1}$$

where

AR_k = aggregate weighted score of candidate contractor k;
n = number of CDFs;
W_i = weight of CDF_i on a scale from 0.0 to 1.0 (0.0 is unimportant and 1.0 is important), where the summation of $W_i = 1.0$ for $i = 1$ to n;
m_i = number of DFs describing CDF_i;
w_{ij} = weight of DF_j describing CDF_i on a scale from 0.0 to 1.0, where the summation of $w_{ij} = 1.0$ for $j = 1$ to m_i and for $i = 1$ to n; and
R_{ijk} = score of the DF_j describing the CDF_i on a scale from 1.0 to 10.0 (1.0 is unsatisfactory and 10.0 is excellent) for candidate contractor k.

The assumption of additivity of the decision criteria for this model has been made, thus requiring independence of the model's criteria.

The CDF's weights (W_i) were obtained using the following equation:

$$W_i = \frac{\overline{CR_i}}{\sum_{i=1}^{n} \overline{CF_i}} \tag{2}$$

where

$$\overline{CF_i} = \frac{\sum_{j=1}^{m_i} \overline{DFMI_{ij}}}{m_i} \tag{3}$$

TABLE 1 MEAN IMPACT OF QUESTIONNAIRE ITEMS FOR STATE DOT RESPONDENTS

QUESTIONNAIRE ITEM NAME (1)	MEAN IMPACT[a] (2)
MAJOR FACTORS	
Financial Stability	3.82
Experience	3.44
References	2.06
Past Performance	3.09
Capacity of Firm	2.60
Current Work Load	2.18
Project Control Procedures	1.97
Staff Available	2.23
Location of Home Office	0.29
Experience in Geographic Location of Project	0.68
Safety Performance	1.70
Substance Abuse Policy	0.62
Project Management Capabilities	2.47
Quality Performance	2.85
Manpower Resources (Labor)	1.82
Company Organization	2.29
Amount of Work Performed with Own Forces	2.32
Contractor has Failed to Complete a Contract	3.35
Equipment Resources	2.79
Bonding Capacity	2.14
SUBFACTORS	
FINANCIAL STABILITY	
Credit Rating	1.72
Banking Arrangements	1.47
Bonding Capacity	2.32
Financial Statement	3.58
EXPERIENCE	
Success of Completed Projects	3.18
Size of Completed Projects	2.79
Number of Similar Completed Projects	2.56
Types of Projects Completed	2.91
INFORMATION OBTAINED FROM REFERENCES	
Review of Reputation and Ethics of Contractor	2.18
Willingness to Resolve Conflicts and Problems	2.24
Change Orders Frequency	1.44
Schedule Performance	2.47
Number of Times Claims Have Gone to Litigation	1.55
PAST PERFORMANCE	
Actual Quality Achieved (within specifications)	2.88
Actual Schedule Achieved	2.70
Number of Times Contractor has met Cost, Quality and Schedule	2.12
CAPACITY OF FIRM	
Last Year's Construction Volume in $	1.74
Construction Volume $ Averaged over the Last Three Years	1.62
Current Backlog of Work $	2.18
% of Current Backlog that an Additional Job Represents	1.68
This Year's Employment (Number of People)	1.06
Employment Averaged Over the Last Three Years	0.82
Employment Trends and Fluctuations	1.00
Staff Available for this Specific Project	1.68
The Number of Professional Personnel	1.68

TABLE 1 *(continued on next page)*

TABLE 1 (*continued*)

QUESTIONNAIRE ITEM NAME (1)	MEAN IMPACT[a] (2)
PROJECT CONTROL PROCEDURES	
Type of Control Procedures	1.70
Type of Safety Program	1.67
Type of Cost Control and Reporting System	1.25
Type of Scheduling System	1.42
Type of Quality Program	1.79
Sophistication of Control Procedures	1.38
Previous Experience with these Procedures	1.48
Your Judgement as to Whether Management is Able to Use the Procedures Effectively	1.70
LOCATION OF HOME OFFICE	
Home Office Location Relative to Job Site Location	0.42
GEOGRAPHIC LOCATION OF PROJECT	
Contractor's Familiarity with Weather Conditions	0.68
Contractor's Familiarity with Local Labor Agreements	0.88
Contractor's Familiarity with Local Politics	0.44
Market Conditions of the Geographic Area	0.68
Contractor's Familiarity with Subsurface Characteristics	0.91
SAFETY	
Existence of Contractor Safety Program and Director	1.44
Contractor's Experience Modification Rate (EMR) for the Last Three Years	0.88
Information from OSHA Log 200 Accident Reports	0.70
Apparent Management Awareness of Safety Issues in Contractor's Organization	1.20
Contractor's Faithfulness in Conducting Tool Box Meetings	0.79
PROJECT MANAGEMENT CAPABILITIES	
Key Personnel Experience, Include Number of Years in Construction & Projects Worked on	3.09
Complexity of Past Projects	2.64
Appropriateness of Project Organizational Chart	1.35
Track Record of Quality of Job (Length of Punchlist)	2.03
Track Record-Schedule	1.97
Track Record-Cost	1.41
Ability to Deal with Unanticipated Problems	1.79
Amount of Decision-making Authority in the Field	1.70
Amount of Work Performed with Own Forces on Past Projects	2.26
MANPOWER RESOURCES	
Amount of Manpower Available	1.68
Quality of Manpower	2.03
Existence or Effectiveness of Company Training Program	1.24
Whether the Contractor is Union or Open Shop	0.44
COMPANY ORGANIZATION	
Type of Ownership (Partnership, Corporation, Sole Owner, etc.)	0.65
Number of Years in Construction	2.12
Contractor's Licenses Held (by State and/or by Category of Work)	1.76
Number of Failures to Complete a Contract	3.06
Appropriateness of Company Organizational Structure	1.41
EQUIPMENT RESOURCES	
Type of Equipment	3.03
Size of Equipment	2.38
Condition of Equipment	2.54
Availability of Equipment	2.79
Suitability of the Equipment for this Project	2.56

[a]Rating scale used was 4 = High Impact, 2 = Moderate Impact, and 0 = Little/No Impact.

TABLE 2 RANK ORDER OF THE 10 DECISION FACTORS AND SUBFACTORS WITH THE LARGEST MEAN IMPACT BASED ON STATE DOT RESPONDENTS

Questionnaire Item Name (1)	Mean Impact (2)
(a) DECISION FACTORS	
Financial Stability	3.82[a]
Experience	3.44
Contractor has Failed to Complete a Contract	3.35
Past Performance	3.09
Quality Performance	2.85
Equipment Resources	2.79
Capacity of Firm	2.60
Project Management Capabilities	2.47
Amount of Work Performed with Own Forces	2.32
Company Organization	2.29
(b) DECISION SUBFACTORS[b]	
Financial Statement	3.58
Success of Completed Projects	3.18
Key Personnel Experience	3.90
Number of Failures to Complete a Contract	3.06
Type of Equipment	3.03
Types of Projects Completed	2.91
Actual Quality Achieved (within specifications)	2.88
Size of Completed Projects	2.79
Availability of Equipment	2.79
Actual Schedule Achieved	2.70

[a]Rating scale used was 4 = High Impact, 2 = Moderate Impact, and 0 = Little/No Impact.

[b]Decision subfactors characterize a major decision factor and aid in making a decision regarding that factor.

where $\overline{CF}_i$ equals the mean impact value of CDF_i and $\overline{DFMI}_{ij}$ equals the mean impact of DF_j describing CDF_i (on the basis of state DOT respondents).

The weights established by using Equation 2 are presented in Table 5. The weights (w_{ij}) for the DFs were calculated using the following equation:

$$w_{ij} = \frac{\overline{DFMI}_{ij}}{\sum_{j=1}^{m_i} \overline{DFMI}_{ij}} \qquad (4)$$

The weights established using this equation are also presented in Table 6.

COMPUTER PROGRAM DESCRIPTION

The computer program implements the model and calculates an aggregate weighted rating for each candidate contractor on the basis of the DF's subjective input rating. These aggregate weighted ratings are ordered by rank, and relevant statistics are calculated. A facility to review each rating input for a candidate contractor's DF along with statistics containing the complete sample of candidate contractors is provided. The decision system embedded in the program consists of three major parts:

1. Decision Parameters. Providing CDFs and DFs, which each candidate contractor will be rated against;
2. Decision Parameter Weight. Quantifying the perceived importance of each CDF and DF for the prequalification decision; and
3. Decision Parameter Statistics. Providing relevant statistical data for prequalification analysis and decision making.

The program has been implemented on the Gould PN-9080 UNIX-based and IBM PC DOS-based computers and was written in FORTRAN 77 and Microsoft FORTRAN (*11*), respectively.

A diagram of input requirements and the resulting outputs for the computer program is shown in Figure 2. The program needs several inputs before performing an evaluation. First,

TABLE 3 RANK ORDER OF THE 10 DECISION FACTORS AND SUBFACTORS WITH THE SMALLEST MEAN IMPACT BASED ON STATE DOT RESPONDENTS

Questionnaire Item Name (1)	Mean Impact (2)
(a) DECISION FACTORS	
Location of Home Office	0.29[a]
Substance Abuse Policy	0.62
Experience in Geographical Location of Project	0.68
Safety Performance	1.70
Manpower Resources (labor)	1.82
Project Control Procedures	1.97
References	2.06
Bonding Capacity	2.14
Current Work Load	2.18
Staff Available	2.23
(b) DECISION SUBFACTORS[b]	
Home Office Location Relative to Job Site Location	0.42
Whether Contractor is Union or Open Shop	0.44
Contractor's Familiarity with Local Politics	0.44
Type of Ownership	0.65
Contractor's Familiarity with Weather Conditions	0.68
Market Conditions of the Geographic Area	0.68
Information from OSHA Log 200 Accident Reports	0.70
Contractor's Faithfulness in Conducting Tool Box Meetings	0.79
Employment Averaged Over the Last Three Years	0.82
Contractor's Experience Modification Rate (EMR) for the Last Three Years	0.88

[a]Rating scale used was 4 = High Impact, 2 = Moderate Impact, and 0 = Little/No Impact.

[b]Decision subfactors characterize a major decision factor and aid in making a decision regarding that factor.

project-specific data containing the project name and number are required. Next, decision criteria are selected. These criteria include the following available alternatives:

1. Accept system-specified CDFs and DFs on the basis of program-supplied factor analysis results;
2. Modify the program-supplied CDFs and DFs by (a) deleting CDFs, DFs, or both; (b) adding up to 10 additional DFs to an existing system-specified CDF; or (c) creating and adding additional CDFs and DFs to the system (up to 10 individual CDFs characterized by up to 10 individual DFs each can be added); or
3. Create a personal prequalification decision system including CDFs and DFs. Twenty individual CDFs, characterized by up to 10 individual DFs each, can be input into the system.

Once the decision parameters have been specified, their weighting scheme must be selected. The user has two options in order to weight the CDFs and DFs established in the previous step:

1. Accept system-specified weights on the basis of the program-supplied mean impact results; or
2. Provide user-specified weights. This procedure allows the user to input the perceived importance on a scale from 0.0 to 1.0 for each CDF and DF. All CDFs are normalized so that the sum of the weights equals 1. All DFs that characterize a CDF are also normalized so that the sum of the weights equals 1. The system allows for the user to input the weights and then view the normalized weights. A facility permits the normalized weights to be modified by reentering the perceived importance on a scale from 0.0 to 1.0. A user can continue to modify input values until satisfied with the normalized weights given to each decision parameter.

The system-specified weights can only be chosen if the system-specified CDFs and DFs are selected. Otherwise, the weights must be input by the user. Once the selection has been made with appropriate queries and inputs, the program rates the candidate contractors.

The user is queried to input the contractor's name and a subjective rating for each DF using a scale of 1.0 to 10.0 (1.0 equals unsatisfactory and 10.0 equals excellent). The aggregate weighted rating is then calculated using Equation 1. This process is repeated until the list of candidate contractors has been exhausted.

TABLE 4 PUBLIC OWNERS FACTOR ANALYSIS RESULTS

Questionnaire Item	Factor Score
CDF 1—Performance	
Contractor has failed to complete a contract	0.15
Past performance	0.13
Quality performance	0.20
Project management capabilities	0.11
Staff available	0.06
Project control procedures	0.15
Safety performance	0.20
CDF 2—Type of Contractor	
Experience	0.21
Company organization	0.33
Equipment resources	0.46
CDF 3—Capacity for Assuming New Projects	
Capacity of firm	0.40
Current work load	0.42
Manpower resources	0.18
CDF 4—Location	
Location of home office	0.54
Experience of geographical location of project	0.46
CDF 5—Percentage of Work Performed	
Amount of work performed with own forces	1.00
CDF 6—Third Party Evaluation	
References	0.46
Bonding capacity	
CDF 7—Financial Capability	
Financial stability	1.00

NOTES: The results include 25 additional data samples obtained from other governmental agencies (e.g., state Departments of Administration and the U.S. Department of Defense). The questionnaire item "Substance Abuse Policy" failed to show up in any of the factors listed. Factor scores are based on regression results that have been standardized.

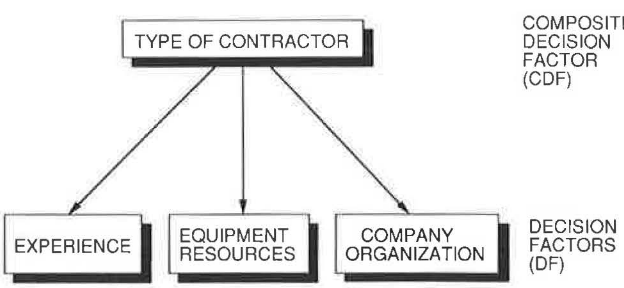

FIGURE 1 Example of the decision model structure.

Outputs of the program consist of two parts: (a) rank-ordered aggregate weighted ratings and (b) statistics for each DF and aggregate rating. Aggregate ratings are obtained using the previously described relevant inputs along with Equation 1. These values are then ordered by rank from highest to lowest. The program's final output consists of statistics for all input DF data and the aggregate weighted rating for all candidate contractors. The statistics calculated for each DF include the mean, $\overline{R}_{ij}$, and standard deviation, $\hat{\sigma}_{ij}$. The mean is calculated using

$$\overline{R}_{ij} = \sum_{k=1}^{n} R_{ijk}/n \tag{5}$$

where

$\overline{R}_{ij}$ = mean rating of the jth DF, which characterizes the ith CDF;
n = total number of candidate contractors; and
R_{ijk} = rating of the jth DF, which characterizes the ith CDF for candidate contractor k.

The standard deviation is calculated from

$$\hat{\sigma}_{ij} = \left[\frac{\sum_{k=1}^{n} (R_{ijk} - \overline{R}_{ij})^2}{n-1}\right]^{1/2} \tag{6}$$

For the aggregate weighted ratings, the mean $\overline{\text{AR}}$ and standard deviation $\hat{\sigma}_{\text{AS}}$ are also calculated. The mean aggregate weighted rating is calculated from

$$\overline{\text{AR}} = \sum_{k=1}^{n} \text{AR}_k/n \tag{7}$$

where $\overline{\text{AR}}$ is the mean aggregate weighted rating and AR_k is the aggregate weighted rating of candidate contractor k. The standard deviation of the aggregate weighted ratings is calculated using

$$\hat{\sigma}_{\text{AS}} = \left[\frac{\sum_{k=1}^{n} (\text{AR}_k - \overline{\text{AR}})^2}{n-1}\right]^{1/2} \tag{8}$$

A more complete description of this computer program was given by Russell (*12*). In the next section, an example application is provided.

EXAMPLE PROGRAM APPLICATION

The program was applied to a sample case by two prequalification officials from the Indiana Department of Highways (IDOH). Five previous bidders on bridge construction projects were evaluated using 3 years of information they had submitted on the IDOH Contractor's Statement of Experience and Financial Conditions forms. Information available from other sources was also used to determine subjective inputs for these contractors. The contractors are referred to as A, B, C, D, and E.

Table 7 presents the subjective ratings given to the DFs for each contractor by the prequalification officials. The CDF third-party evaluation was deleted from the analysis because of insufficient information. Thus, the weights input into the program for the CDFs were 0.19, 0.19, 0.16, 0.05, 0.16, and 0.25. A sample format of the program's output using these data is presented in Table 8. A minimum allowable aggregate rating or threshold can be established in order to select highway contractors permitted to bid on contracts. This threshold is established on the basis of the decision maker's judgment. For example, if a rating of 6.0 was chosen, contractors B, E, C, and D would be permitted to bid.

TABLE 5 WEIGHTS OF CDFs FOR STATE DOT RESPONDENTS

	CDF Index and Name (1)	Weight (2)
1	Performance	0.16
2	Type of Contractor	0.17
3	Capacity for Assuming New Projects	0.14
4	Location	0.03
5	Percentage of Work Performed	0.14
6	Third Party Evaluation	0.13
7	Financial Capability	0.23

TABLE 6 WEIGHTS OF DFs FOR STATE DOT RESPONDENTS

Questionnaire Item	Weight
CDF 1—Performance	
Contractor has failed to complete a contract	0.19
Past performance	0.17
Quality performance	0.16
Project management capabilities	0.14
Staff available	0.13
Project control procedures	0.11
Safety performance	0.10
CDF 2—Type of Contractor	
Experience	0.40
Company organization	0.27
Equipment resources	0.33
CDF 3—Capacity for Assuming New Projects	
Capacity of firm	0.39
Current work load	0.33
Manpower resources	0.28
CDF 4—Location	
Location of home office	0.30
Experience in geographical location of project	0.70
CDF 5—Percentage of Work Performed	
Amount of work performed with own forces	1.00
CDF 6—Third Party Evaluation	
References	0.49
Bonding capacity	0.51
CDF 7—Financial Capability	
Financial stability	1.00

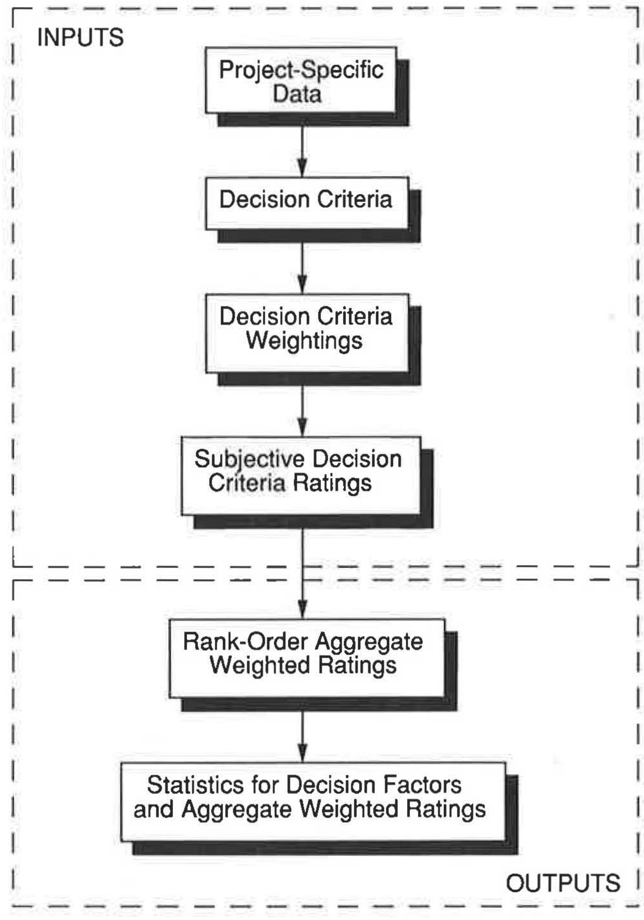

FIGURE 2 Input requirements and outputs of computer program.

SUMMARY AND PRACTICAL APPLICATIONS

The study that was the basis for a formalized computerized prequalification model was described. The linear prequalification model and the developed computer program were also described. The model's advantages are (a) its simplicity, which results in ease of understanding and use by prequalification officials; (b) its structured, systematic, and rational approach to contractor analysis and the subsequent decision making; and (c) its documentation of the reasons contractors were selected for the bid list.

One major disadvantage of this computerized prequalification model is that the rating inputs (R_{ijk}) depend on the user's ability to analyze and synthesize available contractor data and assign a value ranging between 1.0 and 10.0 for each decision parameter. A low rating on one decision parameter can be compensated by a high rating on another, thus impacting the calculated aggregate weighted rating. Consequently, a candidate contractor may have a high aggregate weighted rating even though certain significant decision parameters may have low ratings. The model also suffers from an inability to

TABLE 7 SUMMARY OF THE RATINGS INPUT FOR EXAMPLE APPLICATION

Parameter Name	Rating for Contractor A	B	C	D	E
(1)	(2)	(3)	(4)	(5)	(6)
(a) CDF 1. - Performance					
Contractor has Failed to Complete a Contract	7	8	8	8	7
Past Performance	7	8	8	8	8
Quality Performance	7	8	8	7	8
Project Management Capabilities	7	8	8	7	8
Staff Available	7	8	8	7	8
Project Control Procedures	7	8	8	7	8
Safety Performance	7	8	8	7	8
(b) CDF 2. - Type of Contractor					
Experience	6	8	8	7	8
Company Organization	7	8	8	7	8
Equipment Resources	6	8	7	6	8
(c) CDF 3. - Capacity for Assuming New Projects					
Capacity of Firm	6	8	8	8	8
Current Work Load	7	8	8	8	8
Manpower Resources	6	8	8	8	8
(d) CDF 4. - Location					
Location of Home Office	7	8	8	7	8
Experience in Geographical Location of Project	7	8	8	7	8
(e) CDF 5. - Percentage of Work Performed					
Amount of Work Performed with Own Forces	5	5	5	5	5
(f) CDF 6. - Third Party Evaluation					
References	-	-	-	-	-
Bonding Capacity	-	-	-	-	-
(g) CDF 7. - Financial Capability					
Financial Stability	5	8	7	5	8

TABLE 8 PROGRAM OUTPUT FOR EXAMPLE APPLICATION

Rank	Contractor Name	Aggregate Weighted Rating
(1)	(2)	(3)
1	Contractor B	7.52
2	Contractor E	7.48
3	Contractor C	7.21
4	Contractor D	6.34
5	Contractor A	5.93

Statistics on aggregate ratings are:

Mean Rating = 6.90 Standard Deviation = 0.72

adequately represent the risk profile of the decision maker and the uncertainty associated with the data collected on candidate contractors.

State DOT prequalification officials can use the model in several ways. These alternatives include (a) using the model to qualify contractors annually, (b) using the model to arrive at the modifying factor applied to the calculated value from the formula, and (c) abandoning current prequalification procedures (performed annually using a formula) and using the model to prequalify highway contractors on a project-by-project basis. These alternatives are described in detail in the following paragraphs.

First, the model could be used to perform an annual review of highway contractors. The current procedure determines a maximum amount of work a contractor may have under contract at any one time. The evaluation process would involve the establishment of a minimum threshold value to permit highway contractors to bid on forthcoming contracts. However, considerable thought is needed on the legal implications of this alternative.

Second, current formulas to prequalify contractors use modifying criteria applied to financially derived values, which have been developed in an ad hoc manner. The proposed model is based on a systematic study of expert opinion. The results obtained from the application of the model can represent or correspond to a modifying coefficient. For example, an aggregate rating of 5 can result in a 50 percent reduction in the value calculated from the formula. A formula can state that working capital is multiplied by 10 and then modified 40 percent by past performance, 20 percent by experience, 20 percent by equipment resources, and 20 percent by cooperation. Using the model, if the highway contractor's working capital is $100,000, his maximum allowable work program would be $1,000,000 ($100,000 multiplied by 10) times 0.50, or $500,000. A similar approach is used by the state of Utah.

Changing annual highway contractor prequalification formulas to prequalification on a project-by-project basis with the model described is a third alternative. In this case, an aggregate rating would be used to determine whether a contractor is permitted to submit a bid on a project. Similar to the application of annual prequalification, a minimum aggregate rating can be established to select contractors. Practical application considerations of this alternative include the level of detail contained within the model. Applying the model to every project let by a state DOT is not feasible; consequently, estimated project cost and complexity thresholds must be established to permit an efficient use of resources. This procedure would also eliminate the need to classify the type of work a contractor is permitted to do under the current procedure, which in many instances is a difficult task to accomplish accurately.

Before implementing the model in any of the proposed alternatives, further calibration through example applications should be performed. In addition, a contractor prequalification questionnaire would need to be redesigned to facilitate the gathering of relevant data needed to support the model.

ACKNOWLEDGMENTS

The decision analysis study that led to the development of the prequalification program was supported by the U.S. Army Construction Engineering Research Laboratory. The author thanks Chang-hsin Hsieh for his assistance in coding the model as well as Michael R. Beuchel, Prequalification Engineer, and Howard W. Bushman, Prequalification Auditor, of IDOH in Indianapolis, Indiana, for their review and development of an example application using the computerized prequalification model.

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Integration of Positioning and Computer-Aided Drafting and Design Technologies for Transportation Facilities

YVAN J. BELIVEAU

The advent of improved computers has brought about unique potential for improved control of construction projects. One area of control is in the integration of computer-aided drafting and design (CADD) and positioning of data to provide real-time data to equipment and personnel on the site. The components required for real-time data about positions within a site include CADD and animation software as well as specialized hardware. Some current research has taken place at Virginia Polytechnic Institute in the area of the interface between CADD and positioning. Three areas of current research in the area of positioning are of interest. Potential applications and benefits of the interface between CADD and positioning range from surveying to controlling autonomous vehicles.

The power of computing is continuously changing perceptions of what is and is not possible. Raw processing speeds and graphic acceleration seem to be growing at an ever-increasing rate. This growth is enabling computing applications that were impossible a few years ago.

Some applications that have become possible because of this increased speed are in the areas of computer-aided drafting and design (CADD) technology and near real-time positioning technology. The CADD technology to be discussed is based on hardware and software in the Civil Engineering Department of Virginia Polytechnic Institute (VPI). The hardware and software to be discussed include CADD and CADD model animation capabilities. The positioning technology to be discussed is limited to current research at VPI. These areas of research include positioning with radio waves, lasers, and charge-coupled device (CCD) cameras. The potential applications of the interface between CADD and positioning will be limited to the area of construction improvements in transportation facilities.

HARDWARE AND SOFTWARE DESCRIPTION

Real-time positioning of moving objects or equipment requires fast graphics processing with state-of-the-art animation capabilities. The following subsections describe the components being used at VPI.

Software

The software includes three basic systems. These systems include a CADD package called Microstation, a CADD library and overlay called 3DM™, and an animation and visualization package called WALKTHRU™.

Microstation is a CADD product designed to run on Intergraph equipment as well as 386-based personal computers (PCs). Microstation offers excellent flexibility and ease of use for generating models. Because Microstation can be transferred from a PC to an upper-end Intergraph workstation, less complex work can be done on low-end workstations and moved to other workstations for more complex activities. Microstation is currently running on several platforms, including Intergraph Models 120 and 360, and several 386 PCs.

The 3DM™ product, made by Bechtel Software, runs concurrently as an overlay to the Microstation software. The 3DM™ product runs on all the platforms that run Microstation. Its system is primarily designed to facilitate rapid model generations and to extend the functionality of the Microstation software. The 3DM™ system runs at several different utility levels, depending on the machine on which it is being operated. The bulk of the 3DM™ program and library is resident on a Microvax. From the Microvax, the required components are accessed and down-loaded to each workstation via an Ethernet connection.

The WALKTHRU™ product, also by Bechtel Software, is an animation and visualization package that can rapidly manipulate CADD models given graphic acceleration capabilities. This program is designed to allow the user to graphically walk through a CADD-generated model. This walkthrough allows for improved constructability review, as well as improved understanding of the overall model. The WALKTHRU™ animator retains dimensioning and coding details from the CADD-generated model. The dimensioning and coding details are required to interact with the positioning system.

Hardware

The hardware being used at VPI includes Intergraph and 386 PC workstations, a SUN 4/Raster Technology minicomputer, and a Microvax. All the hardware is linked via an Ethernet connection. The linkage allows for rapid file transfer from any system to any other system.

The Intergraph equipment includes several Model 120 and one Model 360 workstations. The primary use for this equipment is the generation of CADD models. Several 386 PC workstations can also generate CADD models.

The Sun 4/Raster Technology equipment is the platform used for graphics manipulation. The Raster Technology

Department of Civil Engineering, Virginia Polytechnic Institute, Blacksburg, Va. 24061-0105.

equipment is a graphics accelerator that allows for real-time viewing and manipulation of complex images.

The Microvax is used as a file server and storage device. Larger models can be stored and accessed through the Ethernet connection. The access and retrieval of data and models are transparent to the user.

POSITIONING DATA

The last pieces of data required for the integration of CADD and positioning technology are the positioning data. This positioning data must by x, y, and z coordinates of multiple identifiable objects. The data must be acquired and interfaced in real time, which is more easily said than done.

Several positioning systems being studied have the potential to provide real-time positioning data. These systems include scanning lasers, global positioning satellites, and laser bar codes. These three systems offer various advantages for differing applications.

The three positioning systems being studied at VPI are the automated positioning and control system (APAC), the automated laser positioning system (ALPS), and CCD cameras. The three systems are described in the following paragraphs.

APAC

The APAC system is a result of a National Science Foundation construction automation grant. The system consists of components shown in Figure 1. The APAC project has tested a series of prototypes and should provide accuracies in the 12-in. range given a marketed product development. The work has demonstrated the feasibility of the system and provided confidence in the system. The project is expected to be completed about March 30, 1990 (*1,2*).

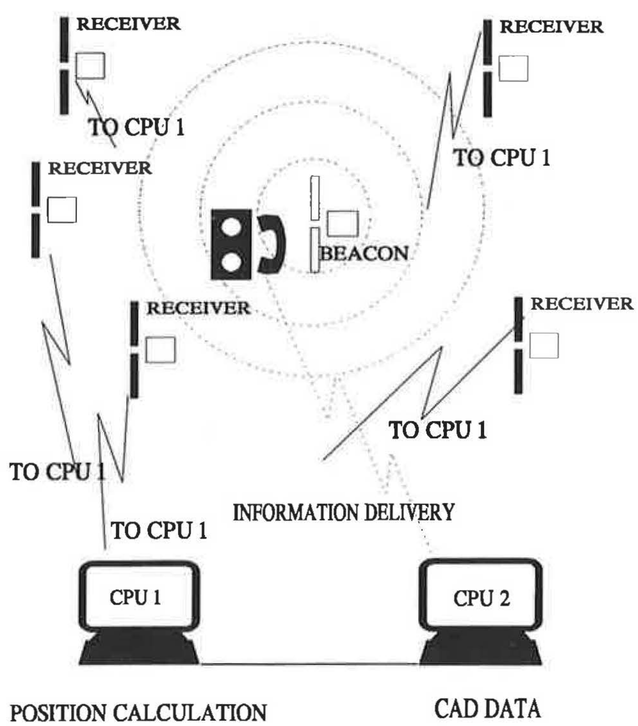

FIGURE 1 APAC system components.

The envisioned APAC system will provide for a frequency-hopping transmission of radio waves at a point within the site. A series of prelocated receivers with known x, y, and z coordinates will be around the site. The relative phase differences between each pair of receivers will be determined. CPU 1 will calculate the position of the beacon. The position data will be sent to the CADD data base in CPU 2. The CADD data base can be used to inform the individual or machine at the beacon of its current location or task. The positioning data integration with CADD data will be accomplished via the WALKTHRU™ animation system (*3*).

The APAC project has tested a 500- by 325-ft site. The site is in rolling terrain with a maximum of 35 ft in elevation change. Two receivers are at each corner of the site. One is 5 ft above the ground, and the other is 22 ft above the ground.

A test in early December 1989 demonstrated an accuracy of ±1.5 ft in the x and y coordinates and ±3.5 ft in the z coordinates. This test also confirmed some unresolved signal drift. This drift problem should be resolved shortly, and a more robust test of the system will be performed.

ALPS

ALPS is a method of determining the x, y, and z coordinates of objects using lasers. Accurate positioning is possible using a system with a rotating laser and laser detectors. By providing continuous position information on the x, y, and z coordinates, the system is ideal for point positioning and tracking moving objects, such as construction equipment. The currently envisioned ALPS will use a laser mounted on a vehicle; however, refinement to the system indicates that a system with lasers at the perimeter and detectors on the vehicle can be used.

The laser used in ALPS produces a narrow beam of light that is rotated perpendicularly about the vertical axis at a constant angular velocity. The laser beam strikes laser detectors in known locations. On being struck, the laser detectors become excited, allowing electrical pulses to flow through momentary circuits. Figure 2 shows the basis of ALPS in the horizontal plane, and Figure 3 shows the z coordinate.

Each laser detector is excited and creates an electrical pulse. Measuring the time difference between the electrical pulses provides the information of calculating the angle between the photodetectors from the rotating laser source. Using these angles, the x and y coordinates are determined by trigonometry. The z coordinate is determined by resolving the position at which the laser beam strikes the detector's vertical axis. This process is shown in Figure 3. The laser is mounted on a servo-stabilized platform, which allows for a level or nearly level plane. The platform is similar to those used by movie crews to film high-speed car chases. The platform is in a vehicle and can provide a completely smooth ride for the camera crew as the scene is filmed. The microprocessor converts the time and electrical impulse data into coordinate information. This system is expected to provide accuracies of less than ¼ in. in the x and y coordinates and ⅛ in. in the z coordinate.

ALPS seems to be an ideal method to get extremely accurate data on a real-time or nearly real-time basis. Given a 1-

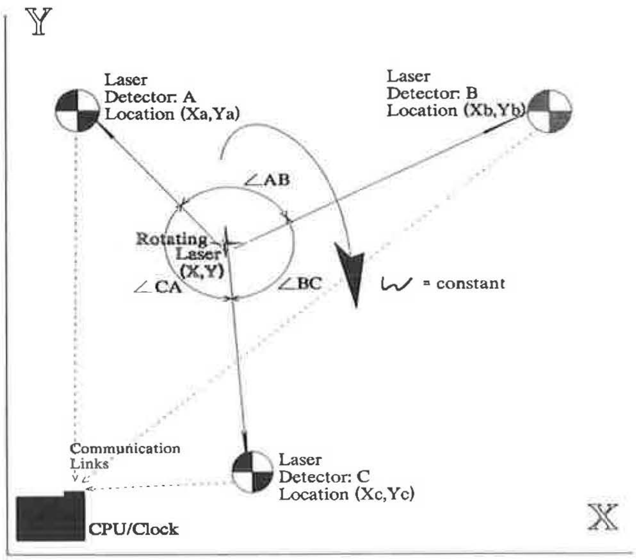

FIGURE 2 Determination of x and y.

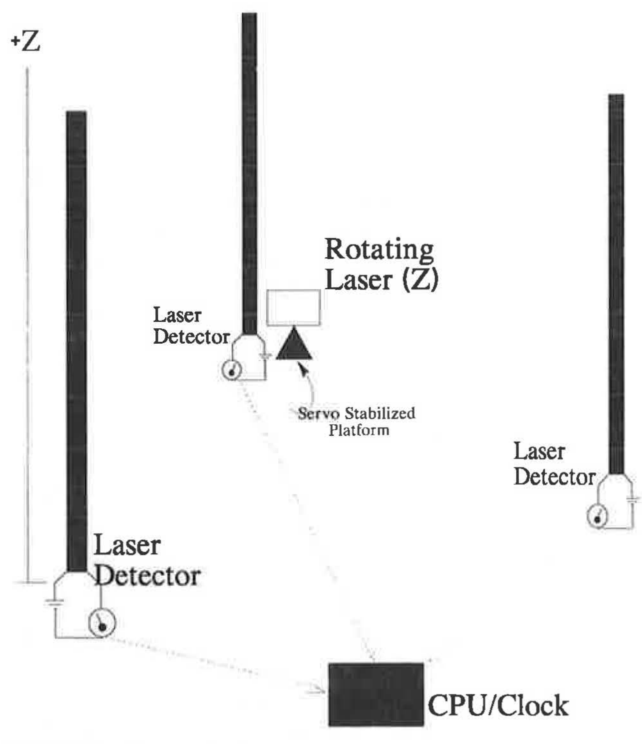

FIGURE 3 - Determination of z.

km² site, accuracies in the 1-cm range are expected to be possible. Accuracy would improve as the size of the site is reduced.

Several tests have been performed to determine the physical limitations of existing lasers and detectors. These tests were documented by Lundberg (*4*). The tests and a review of the existing technology verify accuracies in the ±¼-in. range on a 1,600- by 1,600-ft site. Additional work is being done, and a prototype system is being envisioned.

CCD Cameras

CCD cameras are currently being used on satellites to analyze existing surface features and variations from past surface feature analysis. A CCD camera uses photosensitive cells that are activated by light. Momentary circuits provide the capability of measuring light intensity and color. The 6,000-cell linear array cameras are in an orbit 500 mi above the earth. This system can provide 35-ft accuracy in x, y, and z coordinates given stereoscopic views of similar areas. A 4,000-cell linear-array CCD camera is currently available from Kodak at a price of about $12,000. This camera would provide theoretical accuracies of ¼ in. if cameras were positioned 1,000 yards from the objects of interest.

The Kodak CCD camera takes 1.5 min to sweep a 4,000- by 5,200-cell array. This length of time would preclude using the camera for real-time information gathering; however, this type of CCD camera would be ideal for as-built data analysis, which does not require real-time information.

Several 2,000- by 2,000-cell CCD cameras are available and can provide four updates per second. These CCD cameras would provide accuracies in a ⅛-in. range from 100 yards away.

The amount of processing required to analyze a number of 2,000- by 2,000-cell digital data components would be extremely large. Parallel processing or reduction in the area of study would be required to increase the system's performance to allow for real-time processing.

The interest in positioning using CCD cameras is mitigated by the cost of developing such a system and the speed of existing processors. The speed of the existing processors would have to increase a hundredfold to adequately attempt the solution without parallel processing. With parallel processing, the system could be accomplished today but at significant cost. The ever-increasing speed in processing capabilities may make real-time processing using CCD cameras possible in the near future.

The current approach at VPI is to work in peripheral areas of CCD technology. Specifically, these technologies are in the areas of stereoscopic image analysis and CADD interface. The envisioned system would be reversed engineering of existing facilities.

POTENTIAL USES OF THE CADD AND POSITIONING INTERFACE

A CADD and positioning interface is possible with today's technology. How the positioning data are obtained is subject to the system chosen. Each system described in the preceding section should provide varying degrees of accuracy. The potential usefulness of the interface is limited by the system's accuracy and update time.

The positioning systems were assumed to provide real-time positioning data with accuracies in the ¼-in. range. This accuracy is given as a target for several positioning systems. Such systems have several potential uses in the construction of transportation facilities. These uses are listed in order of accuracy demands, from the least to the most complex.

Cross-Sectional Analysis of Site

The simplest application of the CADD and positioning link would be to use it for quick and accurate topographic analysis. This analysis would be accomplished by driving a measurement vehicle with a beacon mounted on it. The beacon would continuously provide a method of recording *x*, *y*, and *z* data about the terrain. Although initial surveys would probably have been done already, this method could easily determine the actual site terrain at any point in time.

Current work at VPI includes the development of a graphics output module. The graphics module will directly output a terrain model from random *x*, *y*, and *z* position data. The model will be a three-dimensional representation and can be updated and recorded as the project is constructed.

Quantity Survey for Work

The next extension of the CADD and positioning link would be to provide a survey of the amount of work accomplished. This survey would use a measurement vehicle and beacon, like the cross-sectional analysis, but before starting the movement of the vehicle, a specific item would be identified and the survey would respond to that item. For example, the extent of subbase accomplished to date could be measured. The driver of the vehicle would first access the CADD data and enter the work item "subbase." All subsequent position data would be in reference to the subbase. All other work items of interest would be recorded in a similar fashion.

Laydown Yard Control

On larger projects, especially projects with multiple structures, the location and management of individual components can become extremely difficult. The current practice is to provide laydown yards around the site for these materials. Components can be temporarily or permanently lost, causing production and planning delays.

A laydown yard control system using positioning technology and bar-code reading technology is a viable alternative, especially if the entire site has been equipped with a positioning system. Figure 4 shows such an inventory control system (*5*).

The radio frequency (RF) portable terminal in Figure 4 would be a communication device as well as a beacon transmitter for positioning. In addition to the RF portable terminal, a laser bar-code scanner would read and record the identifying bar-code. The data display would include a map of the pertinent laydown areas with *x* and *y* coordinates for the specified component.

The laydown yard control system would be able to provide position coordinates for any item in the laydown yard. When a component is delivered to the laydown yard, a bar-code would be affixed to identify its specific characteristics and

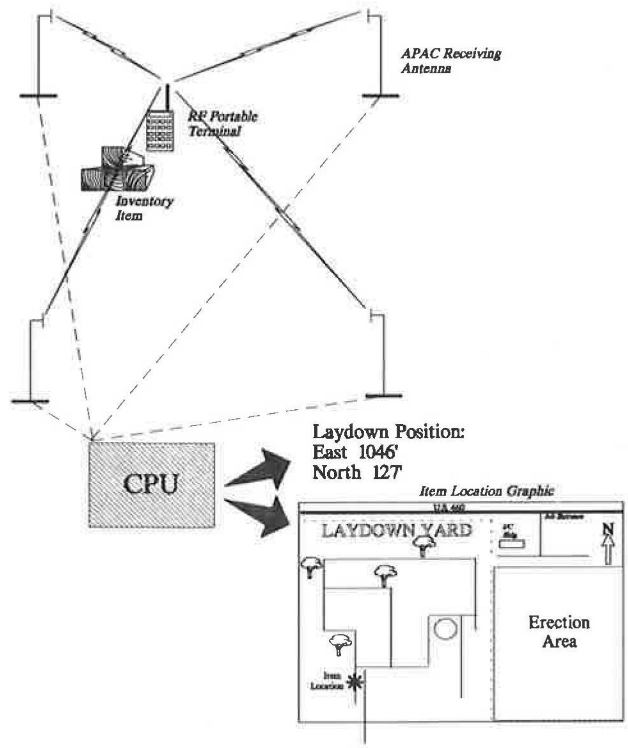

FIGURE 4 APAC system location of inventory.

object identification. After the component is placed in the laydown yard, a beacon would be activated, along with a readout of the bar-code. The object identification and its position coordinates would be stored for future retrieval. When the object is needed, the object identification would be accessed and the position coordinates would be obtained. A view of the laydown yard with roads and other pertinent terrain would be provided, with a point locating the position coordinates. A hand-held beacon would be able to direct the individual looking for the item to the required position.

A small-scale, 20- by 20-ft site has been tested. The position system used was the APAC system. A bar-code and inventory system from Omni Computer service of Piney Flats, Tennessee, was linked to the positioning system, and a two-dimensional output screen provided a visual representation of an automated vehicle. This test is further described by Lundberg and Beliveau (*5*).

Equipment Tracking

The positioning system will be able to track and monitor equipment movement through the construction site. Cycle times, queue lengths, and other delays can be immediately observed and researched. Traffic congestion and bottlenecks can easily be viewed.

The position for each piece of equipment will be recorded throughout a work day. Knowledge-based systems could be devised to study patterns and cycles. The output produced would be information on optimal equipment mix and rerouting schemes.

Grading Control

Grading control can be provided to equipment on a real-time basis as excavation work is done. The update rates for the position could be as high as 45 times per second. This update rate is significantly greater than the needs of most equipment; however, the rate would provide sufficient accuracy for all normal construction activities.

The system would work as follows:

1. The equipment operator would identify the grade activity of work being done. For example, a bulldozer is doing fine gradings for a roadbed.
2. As the equipment moves around, its current x, y, and z coordinates, including its slope and orientation, can be calculated and continually updated.
3. Sensing components would determine the elevation and orientation of the blade in reference to the positioning hardware on a piece of equipment. (These types of sensing components are currently available.)
4. The CADD data would constantly be polled in reference to the position and orientation of the blade with continuous readout of actual blade elevation as opposed to desired subgrade elevation.

Such grading control could provide a significant increase in productivity and work quality over present-day construction methods. The extension into partially automated or fully autonomous vehicles could further increase productivity and quality.

SUMMARY AND CONCLUSION

Research at VPI in the area of the CADD and positioning interface is proceeding. Information on the hardware and software required for real-time data availability to site personnel was presented, and some potential applications were provided. The status of the research on various systems was also provided.

CADD is currently approaching some maturity in the area of design. However, its use in the areas of planning and construction is in its infancy. A CADD and positioning link that offers real-time data will significantly change the productivity and quality of the construction process.

Positioning systems are the primary missing link to accomplish this construction improvement. Significant efforts using a number of different methods are underway. Researchers at VPI are studying the APAC system, ALPS, and CCD cameras for positioning. Several other positioning systems have been demonstrated to provide x, y, and z position data. Most notable of these are globally positioning satellites and scanning lasers. The systems being researched at VPI are other methods to obtain positioning data. However, the applications will work with any positioning system. As these positioning systems evolve, their accuracy will increase and cost will decrease. Consequently, methods of equipment control and surveying will be in for a potential revolution.

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ODEPSI: An Experimental Object-Oriented Data Base Management System

JAE-JUN KIM AND C. WILLIAM IBBS

ODEPSI is an experimental object-oriented data base management system. Developed as an extension to Smalltalk-80™, ODEPSI is currently designed for the management of design and construction project data. ODEPSI represents and manages data items on the basis of the object-oriented paradigm; all data items are in objects, and each object has a set of properties and related procedures. The current version of the system emphasizes automatic constraint enforcement and semantic modeling. ODEPSI provides facilities for defining and maintaining constraints on property values, object types, and object relationships. Semantics, which imply the meaning of the real-world subject being modeled, are represented through the use of object properties and interobject relationships. Major system components of the current version include a data definition language and a data manipulation language.

Data base technology is an ideal tool for integrating various design and construction process functions (*1–3*). Its ability to store, retrieve, sort, analyze, update, delete, and trace data of various kinds, which are generated through all phases of the building cycle, provides a crucial foundation for integration.

A data base is developed with a certain structure that defines the relationships among its entities or record types (*4*). This structure is called a data model. Notable data models in use are the network, hierarchical, and relational forms. Software that implements one of these data models is called a data base management system (DBMS) and is used for the creation, manipulation, and maintenance of a data base.

The relational data base management system (RDBMS), which is based on the relational data model, is the most favored DBMS in construction today. The RDBMS gained popularity over other data base systems for its simplicity and ease of use. Many construction organizations now use data bases as integral parts of their project control systems.

The problem with the relational data model is that it lacks semantic expressiveness, which is an ability to convey the meaning of a real-world subject through the represented data elements. Dynamics of construction projects are not properly depicted in current relational data bases.

The object-oriented data model is an alternative to the traditional pure relational data model for managing construction project data. An experimental object-oriented data base management system (OODBMS) called ODEPSI has been developed. ODEPSI is an acronym for Object-oriented data base management system for DEsign and Project Planning and Control System Integration. The benefit of this approach to project and construction managers is a homogeneous environment for programming and data base that allows better representation of the semantics involved in their projects.

Department of Civil Engineering, University of California, Berkeley, Calif. 94720.

OBJECT-ORIENTED DATA MODELING

Object-oriented data bases are a data modeling and manipulation concept that is based on the concept of an object. Although this concept originated in the 1960s with the SIMULA effort (*5*), it was not fully appreciated until several years later, during the Smalltalk research project at Xerox Palo Alto Research Center. The principal conceptual and functional elements of an object-oriented data base that distinguish it from other data base types are objects, data abstraction, and inheritance.

All data items in an object-oriented data base are treated as objects. Each object is uniquely identified by the system. Any real-world entity, either conceptual or physical, can be represented as an object. Each object can carry and determine its own behavior. This concept of object is an important one, and its benefits will be explained using construction project data.

The existence of an object suggests an interesting comparison from the modeling perspective. In any conventional data base approach, a typical construction project may represent the concept of Activity with a set of data items. Application programs then manipulate this data set to get a meaning of Activity. Because this manipulation process is varied by or within the application programs, careful coordination between the data base and the application programs is needed. However, in an object-oriented data base environment, this external coordination problem no longer exists. The conceptual entity Activity is explicitly represented as an object and its operational behaviors, such as calculating its progress or determining float time, are included within itself.

Other important modeling advantages of an object-oriented data base are its data abstraction and inheritance capabilities. These capabilities are important because they bring efficiency and integrity into the data representation task. Consider a data base storing a structural configuration of a building. Scores of walls may be included. These walls may have instances of two types—exterior and interior. Each wall may have various properties to be described, such as identification, size, material, schedule, cost, and location. One efficient way of organizing these wall data is to make an abstract data object called Wall and to use that object to describe those characteristics common to all walls. Then, this Wall object can be further specialized to differentiate exterior and interior walls—ExteriorWall and InteriorWall. Properties, including behavior, common to these two wall types are defined in the Wall object and inherited to its specializations. Only the necessary characteristics of the two wall types are carried to the lower level.

From this example, the two important concepts of interobject relationships—generalization and specialization—can be observed. Through the data abstraction process of producing a Wall object, a generalization relationship is established. Through the process of subdividing the object into ExteriorWall and InteriorWall objects, a specialization relationship is established. As indicated from this example, one is the mirror image of the other.

In general, the semantics of a real-world subject includes two more concepts of interobject relationships—aggregation and association. In current object-oriented data base environments, these relationships are not explicitly represented. However, using the generalization and specialization concepts as a foundation for object classification, the other relations between objects, aggregation and association, can be represented and maintained.

The importance of this point is indicated by Figure 1, which shows several semantic relationships between building objects. Floor-1 contains three Room instances, Room-1, Room-2, and Room-3. Corridor-1 is part of Floor-1 and adjacent to the three room instances. The three semantic relationships in this example are "contains," "isPartOf," and "isAdjacentTo." The first two relationships, contains and isPartOf, show an example of aggregation; Floor-1 aggregates Room-1, Room-2, Room-3, and Corridor-1. Another relationship, isAdjacentTo, shows an example of association; Corridor-1 is associated with Room-1, Room-2, and Room-3. The following is a list of interobject relationships in the example:

- Floor-1 contains Room-1,
- Floor-1 contains Room-2,
- Floor-1 contains Room-3,
- Floor-1 contains Corridor-1,
- Room-1 isPartOf Floor-1,
- Room-2 isPartOf Floor-1,
- Room-3 isPartOf Floor-1,
- Corridor-1 isPartOf Floor-1,
- Corridor-1 isAdjacentTo Room-1,
- Corridor-1 isAdjacentTo Room-2, and
- Corridor-1 isAdjacentTo Room-3.

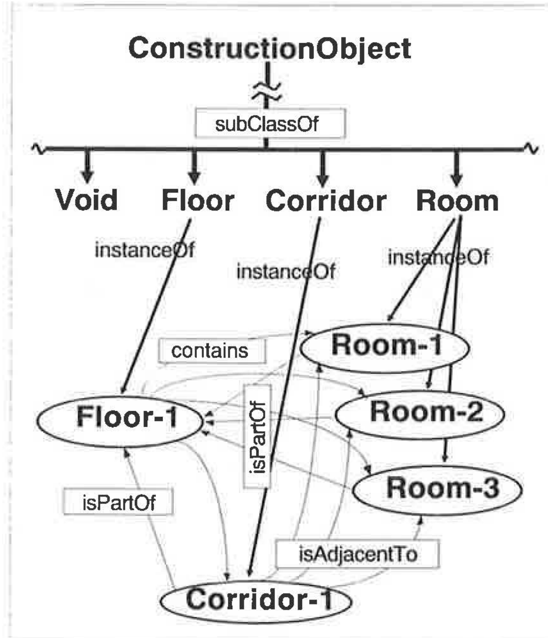

FIGURE 1 Example of aggregation and association.

BASIC ELEMENTS OF OODBMS

The basic elements of an OODBMS, which are commonly recognized within the context of object-oriented paradigm, should be reviewed before ODEPSI is described. The following definitions are based on the Smalltalk-80tm–style object-oriented programming (OOP) concept.

Object

An object is a sole data type (class) supported by OODBMS. Other data types may only be defined as a subclass of object. An object-oriented system, therefore, consists of objects of various kinds (including class objects and instance objects) responsible for features that are analogous to an operating system, program, function, or data element of conventional systems. No clear distinction exists for an application program and a data management system. An object is by itself a package of data and related procedures that belong together. In OOP, procedures are sometimes also called methods (*6*).

An object, being a combination of data and methods, allows information hiding and data encapsulation. Information hiding is a concept that simplifies programming tasks by providing minimum components necessary. It is accomplished by surrounding data and methods within the object. Data encapsulation is a concept that increases programming integrity by restricting objects' values accessed or modified only through their methods. This idea in turn allows reduction of the effect of any one software module's change on others. Put another way, information hiding and data encapsulation concepts ensure reliability, modifiability, and safety of software systems by reducing interdependencies between software components (*7–9*).

In OODBMS, each and every object is unique; a hidden, permanent, unique identifier is assigned to each object. This identifier is only visible to system components. If an object is to be queried directly from objects of the same class without comparing their values, an object's identity should be saved explicitly as a global variable. Thus, if a variable is known that references the object to be queried, there is no need to query the object. It is already accessed through the variable that references it.

For this reason, objects can be queried not only by their values but also by their identities. Some researchers of conventional RDBMSs have independently arrived at the importance of objects (*10–12*). They have expanded the traditional value-based approach of the RDBMS by using the object concept of OOP.

Classes and Instances

A class is an object that describes behaviors of similar objects (instances) in terms of class variables, instance variables, and methods. Class definitions are analogous to schemes of traditional DBMS (and instance variables are used similarly to the fields of relational tables), but classes differ from schemes in that they control their own behaviors. Unlike traditional DBMSs, which have separate programs for data manipulation, each OODBMS class is capable of controlling its behavior through the methods encapsulated within its structure. Manipulating data stored in class and instance variables, and performing system-level operations such as constraint enforcement or consistency maintenance, are possible.

Classes are organized hierarchically to facilitate system organization and development. A new class (data type) is created as a subclass of an existing class (which then becomes a superclass) and shares similarities. Methods and instance variables of the superclass are available for the subclass by means of inheritance. A class hierarchy represents a semantic relationship of generalization and specialization. A superclass is a generalization of its subclasses, and a subclass is a specialization of a superclass. These two semantic relationships are complementary to each other and are sometimes described as an is-a relationship.

Instances themselves become objects, each representing a specific case of a class. They are analogous to tuples (records or rows) of relational tables. An instance object's state is captured in its instance variables, and behaviors of an instance are controlled by the methods of its class. To be precise, those methods that control an instance's behaviors are called instance methods. There are other methods, namely class methods, that only respond to classes. Each class has its class methods and typically uses them for the creation of its instances.

Figure 2 shows an example of structural component classes and their instances. The objects in grey bubbles are classes, and the objects in white bubbles are instances.

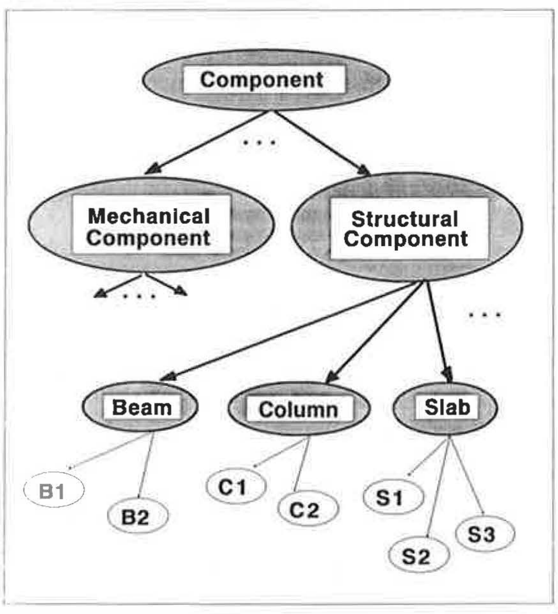

FIGURE 2 Classes and instances.

ODEPSI

Design Philosophy: Data Base plus Programming

The formula data base plus programming sums the objective of much current OODBMS research (*13–17*). This formula is also the design objective of ODEPSI. The OODBMS design philosophy resolves the impedance mismatch problem through the tight interaction of data base concepts within the OOP environment (*18*). Impedance mismatch is a term describing the language difference between a DBMS and related applications programs. Interfaces, like SQL, are widely used to resolve this language difference.

Traditional language and data base interfaces depend on a set of function calls or a separate data manipulation language that has little or no interaction with other language features. Traditional approaches burden the application program during data base interfacing and suffer more when they try to interpret data semantics. If the data base system is developed within the same programming environment as the application development, a programming environment that addresses these interfacing problems can be provided.

System Requirements

The primary requirement for ODEPSI is to provide a formal data (object) definition language (DDL) and data manipulation language (DML). ODEPSI provides extensive data manipulation capabilities, including most DBMS-like data access and query. This permits a focus on those operations that are application-specific. For a construction application, the user of ODEPSI needs to concentrate on the algorithmic behaviors of data objects like CPM processing or Earned Value generation. Access and query behaviors of each object are provided by the system.

ODEPSI also emphasizes automatic constraint management and semantic modeling. The system provides facilities for defining constraints on property values, object types, and object relationships. These constraints are then abided by the system and enforced to the data base users. Semantics of the real-world subject are modeled through the use of object properties, constraints, and interobject relationships. These capabilities are essential features for realizing the integrated data base of design and construction.

Objects live beyond the user sessions in which they are created. Each object is given a unique address by the system. As a concept-proving tool of the proposed research, ODEPSI needs only to be a single-user, virtual-memory–based, single-processor system. Other important data base issues, such as security control, concurrency control, and query optimization, are beyond the scope of this work.

System Development

Overall Architecture

ODEPSI is a set of new methods and classes added to the existing Smalltalk-80™ system. ODEPSI uses existing data types, variables, and control structures of Smalltalk-80tm. The specific additions made to Smalltalk-80tm were DDL and DML modules and a utility module for general system support, including automatic constraint enforcement and semantic modeling.

As schematically shown in Figure 3, ODEPSI and its generated data objects reside within the Smalltalk-80tm image. By combining all elements of program, DBMS, and data base within one homogeneous environment, the level of automation and integration necessary for integrated design and construction data management can be achieved.

ODEPSI Classes

ODEPSI is built around existing Smalltalk-80tm classes. As shown in Figure 4, most ODEPSI methods are created as a part of the Object and Behavior classes of Smalltalk-80tm. Because these two classes are well structured and readily provide a rich set of generic methods for object programming, it is appropriate to develop and include ODEPSI's methods for object management within the same classes.

To handle methods that have no clear links to the Smalltalk-80tm classes, two new classes, Property and Db, were created. They were designed as subclasses of Object to inherit its generic behaviors. Property creates instances for class properties, and Db organizes all the classes and their instances created by ODEPSI. The Db class is the root node of all classes created by ODEPSI. Db also provides methods for maintaining and providing unique symbolic identifiers for the objects created.

Concepts

Defining Object Values Through Properties An object is a multivalued entity. Its values are explicitly defined and stored in terms of properties. Each property identifies and stores one or more values of the object. Each property of an object has six generic slots. The actual storage for these slots is produced by generating an instance of Property class.

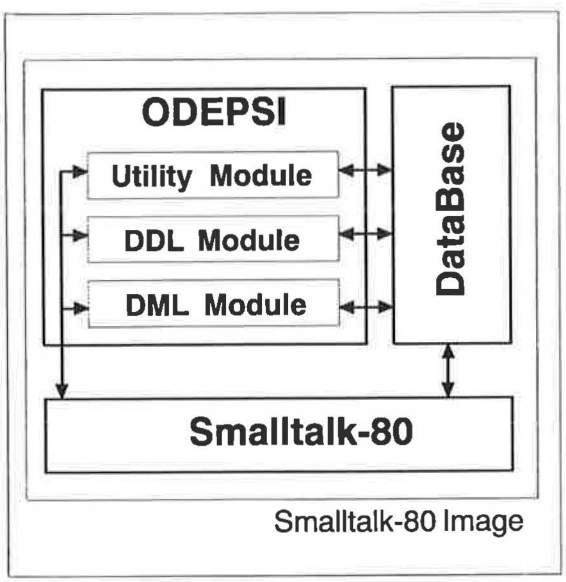

FIGURE 3 ODEPSI architecture.

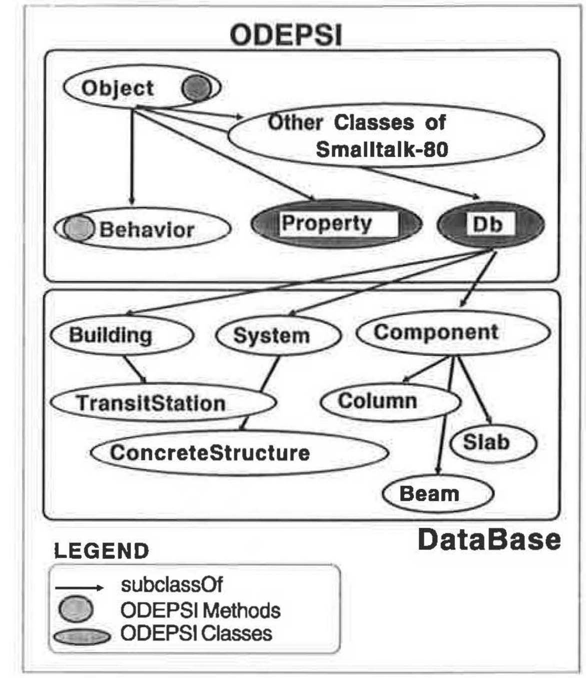

FIGURE 4 ODEPSI class structure.

Slots of a property are used to include the property's identification, value, allowable domain (type), constraint, trigger, and an indication of a single or multiple values. These slots are represented with the following Property instance variables: propName, propValue, propDomain, propConst, propTrig, and isSet (see Figure 5).

A class property in ODEPSI is internally represented with two items: an instance variable named after it and an instance of the Property class (see Figure 5). An instance variable is created to provide each instance a memory space to store its property value. A Property instance is created and allocated to the class to store the six generic contents of the property. A class object stores its Property instances into a class variable whose symbol is made by combining the class name and a word "PropertySet." As shown in Figure 5, Column's class variable is named "ColumnPropertySet."

Controlling Properties With Constraints A constraint can be defined for each property. A constraint of a property is used to check input data validity. A property's constraint is initially defined by the user with a list of procedural code. The constraint is compiled and stored in the constraint slot of the property. This list is evaluated when the property receives input data.

Figure 6 shows an example of this property constraint. A constraint is defined for the location property of Column class. A column instance, *C1*, is also created. The constraint of location property is evaluated when *C1* receives a value for its location property. As shown in the example, when a proper value, (1 2 1), is entered for *C1*'s location, the system accepts

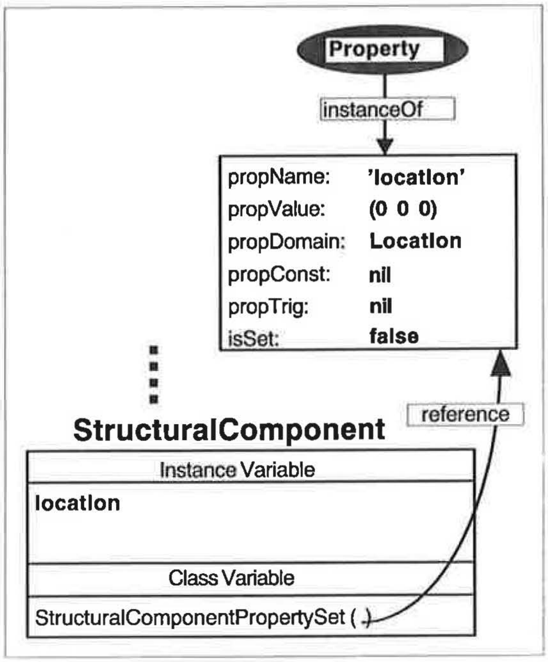

FIGURE 5 Defining a property location.

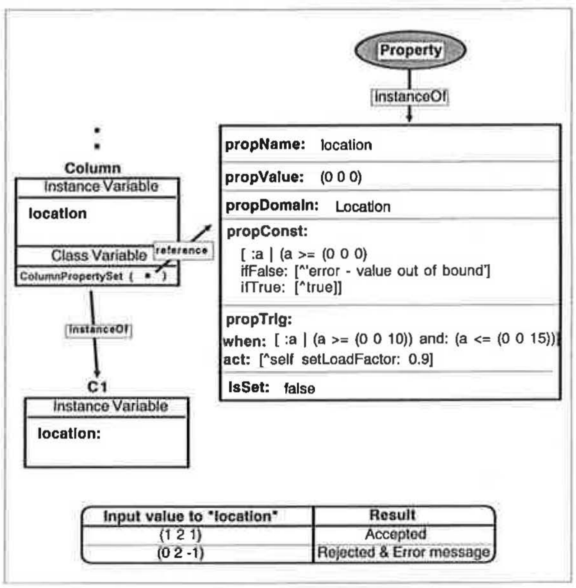

FIGURE 6 Property constraint and trigger.

the value—meaning that the constraint is satisfied. However, when an out-of-bound value, e.g., (0 2 −1), is entered, an error message is issued and the system rejects the value.

This property constraint maintenance mechanism is an important tool for enhancing the integrity and consistency of a data base. As stated earlier in the design philosophy, in this way the user of the data base can concentrate on more important aspects of problem solving. The data base supports the user in maintaining data integrity and consistency.

Controlling Properties With Triggers Another tool that can enhance the integrity and consistency of a data base is the trigger mechanism. A trigger can be defined for each property. A trigger is defined by the user like the constraint mechanism and stored in the trigger slot of a property. The purpose of the trigger mechanism is slightly different from that of the constraint. A property's trigger is activated only when its predefined condition is satisfied by the input data.

Figure 6 also provides an example of trigger operation. Here, a trigger is defined for the location property of Column class. The code defined in the "when:" part is the trigger's condition. If a column is between the 10th and the 15th floor, then the load factor of the Column is reset as 0.9. Neither of the two location values tried earlier satisfies this condition. Thus, this trigger is not activated.

Property Value Inheritance A class's properties are inherited to its subclasses. As shown in Figure 7, a StructuralComponent's subclass, Column, inherits all the properties defined for the StructuralComponent. When it is necessary for the Column class to change or customize the inherited properties, the user redefines any of the property content; value, domain, constraint, or trigger. This operation is supported by the system in the following manner: when the system faces this situation, it copies the property and assigns it to the class variable of the subclass, where the modification attempt is made.

For instance, Column's inherited property, load, is modified by overwriting its contents onto the copy of the property. The original load property object of the StructuralComponent is intact. Once the change is made, the Column class and its instances reference their own copy of the load property, not the property of the StructuralComponent class. This ability to customize the inherited properties is essential to efficiently delegate and control object behavior.

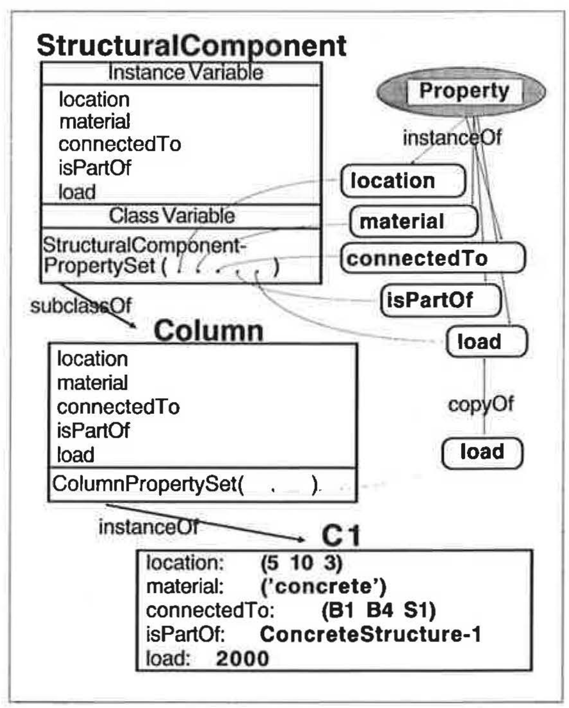

FIGURE 7 Property inheritance.

The benefit of this approach is shown in Figure 7. The Column class modified its inherited load property to define a constraint and a trigger. The constraint and trigger conditions required by Column may be different from those of the Beam or Slab classes. This approach allows individualized class behavior control. A Column instance, C1's load value is then affected by the constraint and trigger newly defined for its class.

SYSTEM USE

Creation of a Class Object

To create a new class object, a class method called defSubclass: is sent to an existing class from which the new classes are to be created. The following sequence of statement creates the initial definition of a new class, StructuralComponent, and its subclass, Column:

1. Component defSubclass: 'StructuralComponent'.
2. Structural Component defProperty: 'location';
   - defProperty: 'material';
   - defProperty: 'connectedTo';
   - defProperty: 'isPartOf';
   - defProperty: 'load'.
3. StructuralComponent defSubclass: 'Column'.

Component is the superclass of StructuralComponent. Once a new class named "StructuralComponent" is created, properties of the class are defined with the method defProperty: Another class, Column, is created as a subclass of the StructuralComponent class. As shown in Figure 7, the Column class inherits the properties of its superclass StructuralComponent. Each class in ODEPSI describes its status in terms of properties. This concept of property is similar to that of the attribute of a frame-based Artificial Intelligence (AI) knowledge representation concept.

Defining Set Properties

A property may have multiple values. For instance, a load-bearing wall may be built with several materials, such as concrete and rebar. To allow a property to have multiple values, it must be declared a set property using the class method defPropertyAsSet: The following statement defines the material and connectedTo properties of Column as set properties:

StructuralComponent defPropertyAsSet: 'material';
  defPropertyAsSet: 'connectedTo'.

The set property concept is an essential requirement for capturing interobject relationships, especially those of aggregation and association. For example, the Column instance is associated with several objects, such as Beam and Slab, and it is built with several materials (e.g., concrete and rebar). The Column is a subclass of the StructuralComponent class and, therefore, any decisions made on the Structural-Component's properties are also inherited by the Column.

Defining Property Domain

A property value can be restricted to a certain class domain in ODEPSI. This ability allows tighter control of data semantics. The domain concept is similar to the type constraint of conventional data bases, but domain is used optionally when necessary. Unless defined, the system provides a default domain value of nil. The following statement defines the domain of the property isPartOf to the class ConcreteStructure:

StructuralComponent defPropertyDomain: 'isPartOf' with:
  ConcreteStructure.

This statement restricts the value of the isPartOf property of StructuralComponent to be a ConcreteStructure instance or that of its subclasses. Once a property is given a domain, the system automatically checks for the validity of its value whenever the property receives a value. Any Smalltalk-80^{tm} class or user-defined class can be a domain value.

Defining Default Property Value

As an example, consider adding a new property to the Column class. This operation can be done by evaluating the following statement:

Column defProperty: 'width'.

A default value of the width property can also be defined. The following statement sets the property width of the class Column to 12:

Column defPropertyValue: 'width' with: 12.

Default property values of a class are automatically propagated to the class instances. For example, all instances of Column will have their width properties valued as 12. If some instance object's width is not 12, then the user must input the new value explicitly. This default concept is useful for engineering applications in which many standard values are repeatedly applied. Default values of class properties are also inherited to subclasses.

Defining and Triggering a Property Constraint

Knowledge necessary to propagate design changes to construction data can be captured through the use of these mechanisms. For example, if the user wants to limit allowable load values of the Column between 2,000 and 2,500 kg/in.^{2}, the user would define this constraint using the class method defPropertyConst::

Column defPropertyConst: 'load'
  with: '[:*a* | (*a* =< 2500 and: *a* => 2000)
  ifFalse: [^''error:
  load value should be between 2000 and 2500'']]'

In ODEPSI, both constraints and triggers are represented and stored within the Block object shown. An instance of Block is surrounded by square brackets and evaluated when it receives the message value. A detailed explanation of the operational aspects of Block and ifFalse: is available from Goldberg and Robson (*19*).

Continuing with the example, suppose the load value is increased from 2,000 to 2,500 kg/in.^{2}. A likely scenario is that the material estimate, especially rebar and perhaps formwork, may have to be adjusted, because the increased load-bearing capacity may demand less structural reinforcement. Users can prepare for this situation using triggers. The following trigger is defined to update the effect of the load-bearing capacity increase:

```
Column defPropertyTrigger: 'load'
  when: '[ :a | (a > 2000)
  act: [^self updateMaterialEstimate "rebar" with: a]]'
```

The preceding message defines the property load trigger and stores it in the class Column. Whenever the property load value of any one class instance is updated and meets the condition (defined in the when: message), its trigger (defined in the act: message) is fired.

SEMANTIC MODELING

Four basic relationships—generalization, specialization, aggregation, and association—are represented and supported in ODEPSI. The ability to handle these basic relationships is what distinguishes OODBMS from other data base concepts. To avoid redundancy, specialization and association are not explicitly discussed here. Their meaning can be inferred by understanding the generalization and aggregation concepts.

Generalization

A generalization relationship permits the grouping of similar objects into a single unit; in other words, generalization describes objects in an abstracted form. For example, the class Column may be a generalization of more detailed classes such as ConcreteColumn, SteelColumn, or WoodColumn. The class Column may store generic properties common to all columns. All columns have properties, such as columnDimension, columnLocation, or supportingComponents. The class Column is an ideal candidate for accommodating all these properties. Then more detailed properties can be assigned to the specialized classes. For example, the class WoodColumn may include a property like woodType to describe the wood material that it is made of. The following statements describe these generalization relationships with ODEPSI syntax:

```
StructuralComponent defSubclass: 'Column'.
Column defProperty: 'columnDimension';
  defProperty: 'columnLocation';
  defProperty: 'supportingComponents'.
Column defSubclass: 'ConcreteColumn'.
Column defSubclass: 'SteelColumn'.
Column defSubclass: 'WoodColumn'.
WoodColumn defProperty: 'woodType'.
```

One important aspect of the generalization relationship is its implication for inheritance. In ODEPSI, properties and their values of superclasses are inherited to their subclasses. In the previous example, those classes specialized from Column inherit all the properties of Column. In addition, those classes inherit all properties of the superclasses of Column (e.g., StructuralComponent). Like the WoodColumn class, they can add properties to themselves.

Aggregation

An aggregation is a form of an isPartOf relationship. With aggregation, one class that contains or aggregates others assumes the role of assembly, and the other classes assume the role of components. Aggregation relationships are especially abundant in engineering applications. A building's structural frame is an aggregation of basic structural components, such as columns, beams, slabs, or stairs.

An aggregation relationship between Floor-1 and the floor components (Room-1, Room-2, Room-3, and Corridor-1) is established as follows:

```
Floor-1 setPropertyValue: 'contains' with: Room-1;
  setPropertyValue: 'contains' with: Room-2;
  setPropertyValue: 'contains' with: Room-1;
  setPropertyValue: 'contains' with: Corridor-1.
```

CONCLUSIONS AND RECOMMENDATIONS

An experimental OODBMS, ODEPSI, has been presented. Basic concepts of object-oriented data representation and management have been discussed. Issues involved in ODEPSI's implementation and its basic syntax have been presented. The approach to achieving property value inheritance and semantic modeling has also been discussed.

The initial results of this work indicate that a homogeneous environment for programming and data base is advantageous for representing the semantics of a construction project. A project model that integrates design objects with construction planning and control objects is being developed with the current version of ODEPSI. This exercise will further clarify directions to take in developing an object-oriented project planning and control system.

The ODEPSI experience indicates several improvements to its functions. At the top of the list is a composite object function. To better represent functional and spatial constraints of a facility, an explicit mechanism is needed to handle those objects that are highly aggregated (i.e., assembly-type objects).

Another useful function being worked on is object versioning combined with the time concept. In the reality of design and construction, many objects are changing property values while design and construction are in progress. On many occasions, histories of those objects must be maintained to resolve conflicts or to capture valuable engineering or construction information that would otherwise be lost.

In the future, composite object management capability will be added to ODEPSI. As more of these enhancements are developed, transportation managers and engineers will see the appearance of object-oriented data bases in day-to-day practice.

ACKNOWLEDGMENT

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Cost-Time Bidding Concept: An Innovative Approach

RALPH D. ELLIS, JR. AND ZOHAR J. HERBSMAN

Details are presented of a relatively new and innovative approach for determining the low bidder on highway construction contracts. In the cost and time method, each bidder proposes both a time duration for the project and traditional unit prices for the work items. A road user cost (RUC) is applied to the proposed contract times. The low bidder is determined as the proposal that provides the lowest combination of bid cost and total RUC. Several state transportation authorities have experimented with this system. Results of these trial cases and the comments of the participants provide interesting indications as to the merits of this new system. Data acquired from 16 case studies are analyzed. Using the conclusions, an innovative bidding procedure is developed for application in public and private sectors.

Awarding construction contracts to the responsible bidder submitting the lowest price is the accepted standard procurement method in the United States. Certainly, with only a few exceptions, all public construction work is procured in this manner.

However, in spite of the universal acceptance of the low-bid system, many participants in the highway construction industry are frustrated with the current system. Owners and construction managers often find that awarding the contract to the low bidder may not achieve overall objectives in terms of total cost, timeliness, and project quality (*1*). Many contractors are also disappointed by a system in which low price is the sole criteria for awarding jobs. A commitment to providing quality in the constructed product may preclude a contractor from being a low bidder.

A review is presented of an innovative and relatively new bidding system for highway construction contracts. This new approach is called bidding on cost and time, which is also sometimes called the *AB* bidding system (*2*). In the cost-time method, each bidder submits a proposed contract time along with the traditional unit prices for the work items. Award is made on the basis of the lowest total cost. Total cost consists of a combination of the contractor's bid price *A* and time value *B*. Time value is derived by multiplying the time proposed by the contractor by a predetermined rate value, which is commonly called the road user cost (RUC) (*3*). In effect, the bidders are required to compete on the basis of both time and price.

Recently, TRB formed a task force to investigate and review innovative contracting practices. One of the objectives of this task force was to identify contracting methods that might prove to be of benefit to the highway construction industry. Much of the research done by the task force will be presented here.

Department of Civil Engineering, University of Florida, Gainesville, Fla. 32611.

EVALUATION OF THE CURRENT LOW-BID SYSTEM

Historical Background

The competitive bidding system appears to have been a part of the American construction process since its beginnings. For example, in New York competitive bidding statutes go back to the Canal Law enacted in 1847 (*1*). In fact, the competitive bidding concept seems to be rooted in America's fundamental belief in a free enterprise system.

Furthermore, the competitive bidding system evolved to provide specific public policy objectives. The first objective was to guard against corruption and mismanagement by public officials. Bidding was also supposed to provide the taxpayer with constructed projects at the lowest possible price obtainable through competition. A comprehensive coverage of the subject of the evolution of the competitive bidding system is presented by Cohen (*4*) and Netherton (*5*).

Today, the policy objectives of the low-bid system remain essentially unchanged. Protection from collusion and corruption are still valid objectives. Additionally, obtaining construction at the lowest competitive price is universally an important goal.

Weaknesses of the Low-Bid System

The current low-bid system is inefficient in several important ways. First, an award on the basis only of the lowest price is likely to produce an environment in which quality control problems may develop. Low price and high quality are, more often than not, contradictory terms. Low price also frequently means timeliness problems. Finally, awarding to the lowest price can lead to claim situations that, in fact, are really generated by an original bid based on unrealistic cost estimates. A low-bid project that has quality problems and that is not completed on time will cost the owner more overall.

Low bidders frequently do not produce the most desirable combination of contract cost, product quality, and project duration. Ideally, award criteria should include evaluation of the contractor's ability and commitment to providing project quality and minimum project duration as well as low-bid cost.

Certainly, there are many difficulties involved in modifying award criteria to include factors other than bid price. The choice of appropriate criteria and fairness are major concerns. Bidding on cost and time does not solve all of the problems of the low-bid system. However, inclusion of project duration as a factor can be a significant improvement to the traditional low-bid award criteria.

THE COST-TIME BIDDING SYSTEM

Project Duration

The task of determining appropriate contract durations is particularly important for transportation construction authorities. Accurate estimates of required construction time are essential (*6*). Unrealistic contract times may result in higher bid cost and increase the possibility of disputes between the contractor and the contracting authority (*7,8*). Also, Thomas (*9*) has emphasized the importance of reasonable time estimates with regard to claims management.

In spite of the importance of setting correct contract times, calculating the required time can be difficult for the transportation authority. The rate at which a project will be performed varies greatly from one contractor to another. Only the specific contractor knows what resources will be committed to the project. During the prebid determination of contract time, the construction authority can only make general assumptions about average production rates. Depending on which contractor is awarded the project, the assumptions may or may not be valid.

Another important point to be considered about project duration is direct effect on overall project cost. The contractor's bid price is in fact only part of the overall construction cost. Two other cost categories contribute significantly to the total construction cost of a project. First, the owner's cost of administering and managing the construction contract, and second, RUC must be taken into account. RUC is incurred by the public as a result of the construction. RUC typically includes delay cost, additional gasoline cost, and other indirect costs experienced by the motorist as a result of the road construction project. Both the administrative cost and the RUC depend on the project's duration.

THE COST-TIME CONCEPT

The cost-time bid concept is a new concept that is a modification of the current low-bid system. In the cost-time bidding system, the element of time is added to the system. Contractors must submit a proposed contract time with their price bid. The low bidder is determined as the bidder providing the lowest total cost combination of both bid price and project time.

Calculation of the total project cost is based on the following equation:

$$CT = C + R * T \tag{1}$$

where

CT = total combined project bid price,
C = contractor's bid price,
R = time value (RUC), and
T = contractor's time bid.

Application of this method can be illustrated with the following example. Given a specific highway project, the contracting authority determines the daily cost to the public resulting from the construction project. This cost will include the cost of administrating the construction project and the cost to the public as a consequence of the uncompleted work. This daily RUC is disclosed as a part of the bid documentation. Bidders are then required to submit both a price and a time proposal. Consider the time-cost bid data presented in Table 1.

Bid results presented in Table 1 indicate an award to bidder D, who has submitted the lowest total combination of both time and price. Contract price would be the bid price submitted by the contractor ($3,882,781.75) and the specified duration would be the contractor's time proposal. Once awarded, the contract is administered in the same manner that a typical low-bid contract is administered. The only difference is in the criteria used to determine the successful bidder.

DETERMINING DAILY RUC

The principal factor in the cost-time bidding system is the determination of the appropriate daily RUC.

Uncompleted projects represent a time-dependent cost for the owner. Certainly, this cannot be disputed. Ongoing construction projects require daily inspection and administration. Additionally, road users are typically inconvenienced by detours or lane closures, which result in longer trip times. Road users are being denied the benefit of the completed project that presumably will provide a safer and more efficient transportation system.

TABLE 1 EXAMPLE OF COST-TIME BID DATA

Bidder	Total of Bid Items (C)	Calendar Days Road Closure/ Contract Time (T)	Road User Cost at $5,000/ Calendar Day (R)	Grand Total for Bid Comparison (C_T)
A	$3,734,211.22	200	$1,000,000.00	$4,734,211.22
B	4,250,125.11	180	900,000.00	5,150,125.11
C	3,689,100.28	220	1,100,000.00	4,789,100.28
D	3,882,781.75	150	750,000.00	4,632,781.75*

* Lowest Combined Bidder Cost.

However, equating these various results of construction to a hard daily cost can be challenging. Careful consideration must be given to the selection of cost factors and the cost values assigned. Nevertheless, determination of an accurate and appropriate RUC value can be done. Figure 1 shows an example of the details of an RUC calculation performed by the Kentucky Department of Transportation for highway projects in Kentucky (*2*).

RUC for bid selection should also include a daily cost for inspection and administration in addition to the motorist cost developed in the Kentucky example (*10*).

Many departments of transportation routinely calculate RUC for various planning and administrative purposes. Determining an appropriate daily cost for the uncompleted project to be used in a cost-time bid system should not be particularly difficult for those agencies.

CASE STUDIES

Although the concept of bidding on cost and time is not totally new, the method has been tested in only a few states. These trials were conducted largely on an experimental basis. With the help of the FHWA most of the trial projects have been found. Results of these trial projects provide a useful indication of how the cost-time system really works. Although the projects were different in scope and location, most were successful in reducing project time as originally estimated by the owner.

The following three case study summaries shown in Figures 2–4 are representative of the general results found. Each figure shows the location and the type of the project. Also included (when available) is the engineer's prebid estimate of the time and cost as well as the RUC value. The figures show that in the final bid tabulation in Cases 2 and 3 (Figures 3 and 4, respectively) the lowest combined bidder was not the lowest on a cost basis.

Table 2 is a summary of the data from a number of case studies that have been conducted in various states in the last few years. In all of the projects, the concept of bidding on time and cost was used; however, the projects differ in details. A significant comparison can be made of the owner's original estimate and actual results. Columns 3 and 5 are the engineer's original estimate in dollars and calendar days, respectively. Columns 4 and 6 are the actual bid results of the lowest combined bidder in dollars and calendar days, respectively.

HIGHWAY ROAD USER COST (RUC) FORMULA (Kentucky 1983)

HUC = (Gasoline Consumption X $1.50/gallon) + (VMT X $0.17/mile vehicle)
+ (0.90 VHT X $0.50/vehicle/hour) + (0.10 VHT X $7.00/vehicle/hour)

where:

1. Gasoline consumption is shown in previous table.
2. $1.50/gallon estimated price for 1985.
3. VMT is vehicle mile of travel as shown in previous table.
4. $0.17/mile/vehicle is vehicle operating cost excluding price of gasoline, taxes, tolls and parking.
5. 0.9 VHT is vehicle miles of travel attributed to passenger vehicles.
6. $0.50/vehicle/hour is updated value of non-commercial or non-business auto trip time.
7. 0.10 VHT is vehicle miles of travel attributed to commercial vehicles (trucks).
8. $7.00 vehicle/hour is updated value of commercial truck trip time.

Without Sections 2A and 2B:

HUC = (1,292,000 gallons X $1.50/gallon) + (16,336,490 vehicle miles
X $0.17/mile/vehicle) + (0.90 X 708,897 vehicle hours
X $0.50/vehicle hour + (0.10 X 708,897 vehicle hours
X $7.00/vehicle/hour)

HUC = $1,938,000 + $2,777,203 + $319,004 + $496,226 = $5,530,435

With Section 2A and 2B:

HUC = (1,291,000 gallons X $1.50/gallon) + (16,358,302 vehicle miles
X $0.17/mile/vehicle) + (0.90 X 702,296 vehicle hours
X $0.50/vehicle/hour) + (0.10 X 702,296 vehicle hours
X $7.00/vehicle/hour)

HUC = $1,936,500 = $2,780,911 = $316,033 + $491,607 = $5,525,051

Daily Road User Benefit (DRUB):

DRUB = $5,530,435 – $5,525,051 = $5,384

From these calculations the Daily Road User Benefit to the motoring public from the construction of Section 2A and 2B of the Jefferson Freeway is $5,400 per day.

FIGURE 1 Calculation of daily RUC.

CASE STUDY # 1

STATE: Delaware
TYPE OR LOCATION: Adding a lane 5.6 miles along SR-1
ORIGINAL ESTIMATE BY DELAWARE DOT:
Cost: $2,904,811.10
Time: 170 calendar days

TIME VALUE (Road User Cost): $5000/day

BID RESULTS

Bidder #	Bid Cost Base	Days Bid	Time Value	Total Amount
1	$3,034,765	120	$600,000	$3,634,765*
2	$3,160,284	120	$825,000	$3,985,284
3	$3,562,980	470	$2,350,000	$5,912,980

*the lowest combined bidder

ACTUAL TIME RESULTS: 125 days

COMMENTS: The state of Delaware (the public) obtained use of the project 50 days earlier than the original estimate. The cost difference was $130,000. The savings for the state based on $5000/day is: 50 x $5000 – $130,000 = $120,000. If we consider all the indirect benefits, such as the DOT overhead, inspection, and less danger of traffic accidents, then the savings are much higher.

FIGURE 2 Bidding on time and cost—Case Study 1.

CASE STUDY # 2

STATE: Kentucky
TYPE OR LOCATION: Four-lane divided highway 5.1 miles long
ORIGINAL ESTIMATE:
Cost: (not released by the department)
Time: 729 calendar days

TIME VALUE (Road User Cost): $5000/day

BID RESULTS

Bidder #	Bid Cost Base	Days Bid	Time Value	Total Amount
1	$15,636, 180.56	450	$2,250,000.00	$17,886,180.56*
2	$16,070,558.46	426	$2,130,000.00	$18,200,558.46
3	$15,628,815.06	523	$2,615,000.00	$18,243,815.06
4	$16,231,527.80	646	$3,230,000.00	$19,461,527.80
5	$15,835,768.22	780	$3,900,000.00	$19,735,768.22

*the lowest combined bidder

ACTUAL TIME RESULTS: 406 days

COMMENTS: The contractor did not employ any outstanding construction techniques, only an aggressive work schedule (on workdays and weekends). A few bidders thought that the $5000/day was too low. No contractors were found who declined to bid on the project.

FIGURE 3 Bidding on time and cost—Case Study 2.

CASE STUDY # 3

STATE: Mississippi
TYPE OR LOCATION: Jones County I-59-2 Project
ORIGINAL ESTIMATE:
Cost: (Not available)
Time: 200 calendar days

TIME VALUE (Road User Cost): $7000/day

BID RESULTS

Bidder	Bid Cost Base	Days Bid	Time Value	Total Amount
1	$4,721,599	151	$1,057,000	$5,778,599*
2	$4,544,930	250	$1,750,000	$6,294,930
3	$5,271,196	212	$1,484,000	$6,755,196
4	$5,215,617	266	$1,862,000	$7,077,617

* the lowest combined bidder

ACTUAL TIME RESULTS: (Not Available)

COMMENTS: Within the department, there are those of the opinion that the road user cost should have been applied only to the time of the lane closure and the time that traffic was maintained on the two-way facility.

FIGURE 4 Bidding on time and cost—Case Study 3.

Differences between the engineer's original estimate and the contractor's bid are divided into project cost and time. Columns 7 and 8 show the difference in dollar value and percentage of cost, respectively, and Columns 9 and 10 show the difference in calendar days and percentage of time, respectively. Column 11 is the volume of RUC in dollars per day.

ANALYSIS OF CASE STUDY RESULTS

Analysis of the data in Table 2 indicates that, in general, significant reductions in project duration can be obtained from using a bidding procedure encompassing both time and price proposals. Although awarded contract prices were typically somewhat higher than the low-bid price, savings derived from reduced project time more than offset the additional base cost. On average, cost-time projects were acquired with a time 108 days less than the project time that would have been set by the owner. The net result of this time reduction was an average savings to the public of approximately $500,000 per project.

The savings to the public by using the bidding on the cost-time system can be calculated using the following formula:

$$S_p = (T_E - T_C)(R) - (C_B - C_L) \quad (2)$$

where

S_p = savings to the public (dollars),
T_E = contract time determined by the engineer (days),
T_C = time bid by contractor (days),
R = RUC (dollars/day),
C_B = bid price of successful or best bidder (dollars), and
C_L = bid price of low bidder (dollars).

Savings are computed as the total savings in daily RUC less any additional direct contract costs occurring when the successful combined-cost bidder is not the lowest base-cost bidder. Although not completely accurate, this formula does give good indication of potential savings to the public.

For example, in Case 2 the parameters were as follows:

T_E = 729 days,
T_C = 450 days,
R = $5,000/day,
C_B = $15,636,180, and
C_L = $15,628,815.

For these values, S_p = (729 days − 450 days) ($5000/day) − ($15,636,180 − $15,628,815) = $1,387,635. Table 3 presents a summary list of the estimated savings to the public achieved by the projects in the case studies. In 11 of 14 projects studied, a significant reduction in project time and a corresponding cost savings to the public were evident.

Additionally, the comments of the participants, both owners and contractors, were found to be supportive of the new system. An FHWA report states that using RUC in low-bid determination does have merit (*11*). However, FHWA also recommends limited use until more experience is gained. A participating state department of transportation concluded that the preliminary results were encouraging—proposed costs were not significantly higher and times were reasonable and desirable. The final FHWA report of a series of test projects in Kentucky found that projects, as advertised and awarded, attracted bidders who practiced efficient construction and engineering management with enough supervisory control to keep a large project on schedule (*10*).

One particularly insightful observation was obtained from a participating contractor. This contractor believed that the most valuable benefit of cost-time bidding is that the system challenges the contractor at the onset to become involved in developing a plan and schedule for performing the project.

VARIATIONS IN DETAILS OF THE COST-TIME METHOD

Although the basic bidding concept is relatively simple, many technical details exist that must be determined. In many cases, legitimate arguments can be made concerning which is the right solution. A few issues discussed are:

- Financial Incentives. A dilemma exists over whether or not to add a money incentive if the lowest bidder finishes the job ahead of the time bidded. For example, in one project in Missouri the original time estimate was 65 days, the contractor bid 45 days, completed it in 36 days, and received an incentive for 9 days. Other states like Texas (*3*) and Kentucky (*10*) used similar methods. Experts suggest that it may not make sense to let the contractor get a bonus related to his own bidding time. The real incentive for the contractor is to be the successful bidder.
- Maximum Time Limits. Whether or not to limit the maximum time of the project by adding a restriction not to exceed a certain number of days is another issue raised. On the one hand, this practice has merits in allowing the owner to set a maximum time limit. On the other hand, many contractors will use this number as their projected time instead of figuring their true optimal time. For a project in Maryland, the state set 800 days as a maximum time. Three of the four contractors bid exactly 800 days. For this reason, some experts think that it is better not to use a maximum time limit.

TABLE 2 SUMMARY OF CASE STUDY RESULTS

Case Study No. (1)	State (2)	Comparison of Engineer's Estimate to Contractor's Proposal: Engineer's Cost Est. ($) (3)	Contractor's Low Combined Bid Price ($) (4)	Engineer's Time Est. (days) (5)	Contractor's Time Bid (days) (6)	Difference Between Engineer's Est. & Contractor's Cost-Time Bid: Project Cost ($) (7)	Project Cost (%) (8)	Project Time days (9)	Project Time (%) (10)	Project Time ($/day) (11)
1.	Delaware	2,904,811	3,034,765	170	120	129,954	4.5	(50)	(29.4)	5000
2.	Delaware	2,158,900	2,306,380	125	160	147,480	6.8	(35)	(28.0)	5000
3.	Mississippi	**	4,721,599	200	151	**	**	(49)	(24.5)	7000
4.	Kentucky	*	16,329,262	1094	517	*	*	(577)	(52.7)	5000
5.	Kentucky	*	12,583,349	153	90	*	*	(63)	(41.2)	5000
6.	Kentucky	*	9,186,877	474	150	*	*	(324)	(68.4)	5000
7.	Kentucky	*	18,554,123	643	500	*	*	(143)	(22.2)	5000
8.	Kentucky	*	15,636,180	729	450	*	*	(219)	(38.3)	5000
9.	Maryland	31,956,630	35,087,606	571	571	3,130,976	9.8	0	0.0	3200
10.	Missouri	1,715,733	1,637,015	30	53	(78,719)	(4.6)	23	76.7	20000
11.	Georgia	1,020,900	1,361,009	90	111	340,109	33.3	21	23.3	7000
12.	Texas	31,120,038	39,833,648	1040	1010	8,713,610	28.0	(30)	(2.9)	5000
13.	Texas	31,824,897	39,781,121	960	900	7,956,224	25.0	(60)	(6.3)	5000
14.	Texas	14,969,654	15,867,833	75	61	898,179	6.0	(11)	(14.7)	5000
15.	Texas	8,893,709	8,200,000	**	360	(693,709)	(7.8)	**	**	5000
16.	Texas	39,743,590	43,400,000	**	750	3,656,410	9.2	**	**	5000

* Not released by the department ** Data not available

TABLE 3 ESTIMATED SAVINGS TO THE PUBLIC USING COST-TIME BIDDING SYSTEM

CASE STUDY # (1)	CASE STUDY STATE (2)	SAVINGS $ (3)	SAVINGS % (4)
1	Delaware	250,000	8.6
2	Kentucky	1,387,635	8.9
3	Mississippi	166,331	7.3
4	Kentucky	2,885,000	17.7
5	Kentucky	315,000	2.5
6	Kentucky	1,620,000	17.6
7	Kentucky	715,000	3.9
8	Delaware	175,000	8.1
9	Maryland	0	0.0
10	Missouri	(460,000)	(26.8)
11	Georgia	(147,000)	(14.4)
12	Texas	150,000	0.5
13	Texas	300,000	0.9
14	Texas	55,000	0.4

• RUCs. What to include in the time value figures for RUC needs to be considered. Some states include only direct costs (fuel, safety elements, etc.), whereas others contend that RUC also needs to include indirect costs, such as inspections, resident engineer, etc. Differences between these two approaches can be substantial. For example, in one Texas case the department of transportation tentatively calculated RUC at $60,000/day; however, after consideration the department revised the amount to only $10,000/day.

• Other Issues. A few more issues are being discussed, such as use of a penalty clause (disincentive), types of project to include in this system, etc. However, any organization can develop methods suited to its particular needs.

SUMMARY AND CONCLUSION

Using both time and base-bid cost as criteria for determining the low bidder on highway construction projects has been shown to be a successful innovation. This system does not change the fundamental concepts of the low-bidder system, but does incorporate an additional element (time) in the low-bidder selection criteria. Although the number of trial projects is relatively small, results indicate that substantial savings in project time can be obtained without significant increases in basic construction cost. Total net savings to the public, calculated by including the daily RUC, make an impressive argument for use of the cost-time bidding system. The basic principles of the concept are simple; however, many technical and legal details need to be determined. Analysis of case studies shows that the cost-time bidding system has an enormous potential for application in the public sector. In the long run, cost-time bidding may also be applied to the private sector.

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Variables Affected by Nighttime Construction Projects

JIMMIE HINZE AND DANA LYNN CARLISLE

An increasing number of roadway rehabilitation projects are using nighttime construction schedules. Factors involved in the decisions to employ nighttime construction schedules were examined in a study of state highway agencies. The study consisted of a survey concerning the various parameters that were considered when nighttime construction schedules were required in construction contracts. State personnel familiar with nighttime construction projects were asked to describe particular aspects of projects that were different as a result of the night schedule. In addition, a survey was conducted with construction contractors who had experience with nighttime construction projects. Results show that the overriding factor for a nighttime schedule that is contractually required is the unacceptably high traffic congestion that would result from daytime roadway work. On nighttime projects, of particular concern is safety and traffic control. Although worker scheduling is made more difficult at night, several contractors indicated that they elected to use nighttime schedules even when not contractually required to do so. These contractors noted that task scheduling is easier at night and that material deliveries are more readily made at night. However, for all parties involved in the process, safety tends to be the overriding concern.

Because roadway surfaces in this country are deteriorating, traffic congestion in metropolitan areas is becoming a greater problem. A paradox is presented because the congestion will increase as a direct result of the efforts taken to rehabilitate the roadways. The added congestion caused by the rehabilitation efforts is often so severe as to preclude work from being performed during regular working hours, particularly during times that coincide with the local rush hour or peak traffic periods. As a result, more work on roadways is taking place at night when the impact on the traveling public is minimized.

Undoubtedly, numerous factors exist that are affected by nighttime construction projects. Reduction in daytime traffic congestion that would result from a conventional construction schedule is one obvious benefit derived from nighttime construction work. However, various negative aspects are introduced as well. For example, construction workers are accustomed to working during the daylight hours. To what extent is labor scheduling a problem on nighttime construction projects? Is worker morale and productivity reduced on nighttime construction projects? Is worker absenteeism increased when work is performed at night? To what extent is supervision impaired by night work? Similar questions could be asked about:

- Availability of materials after hours and any associated premiums that must be paid,
- Impact of noise from nighttime construction projects on residential communities,
- Added risks to drivers imposed by nighttime construction work,
- Added risks for construction workers,
- Inconvenience imposed by nontypical working hours for inspectors and other representatives of the owner,
- Relative quality of nighttime construction caused by reduced visibility,
- Response time of answers to questions when few individuals with authority are available, and
- Impact of the change in relative humidity and temperature on material quality.

Many of these questions go unanswered. However, these are important issues to consider because nighttime construction schedules for rehabilitation of major metropolitan roadways is likely to increase in the coming years in an effort to resolve the problem of deteriorating roadways. For this reason, a research study was conducted to investigate, in greater detail, the decision-making process concerning nighttime construction. The primary objective was to determine which factors are considered most important when construction work needs to be done at night. The study was also designed to examine some of the principal variables that are of particular concern after a nighttime construction schedule has been established. Input was received from state highway agencies and from contractors experienced in nighttime work.

REVIEW OF RELATED LITERATURE

The literature review consisted of an examination of a variety of published materials on road rehabilitation and on road maintenance activities. Examination of these materials was further restricted to activities on major metropolitan highways or arterials that were performed, at least in part, at night. This search of the literature was conducted to identify the primary factors that are considered when a decision is made to use a nighttime construction schedule. A secondary objective was to identify unique aspects of nighttime construction.

From the literature, decisions regarding the use of a nighttime schedule are based on information similar to that used when making determinations of traffic capacity. Texas has devised a means of determining traffic capacity for various roadway configurations when varying numbers of lanes are closed (*1*). Although this technique may be valuable in forecasting traffic levels, this planning tool does not specifically address nighttime lane closures. Another method is the *Highway Capacity Manual*'s (*2*) empirical formula used to estimate

J. Hinze, Department of Civil Engineering, University of Washington, Seattle, Wash. 98195. D. L. Carlisle, Earth Consultants, Inc., 1805 136th Place, N.E., Suite 101, Bellevue, Wash. 98005.

roadway capacity. Some suggest that this formula be modified to reflect also the influence of the duration of the lane closure (*3*). Thus, various methods exist whereby predictions of traffic capacity can be made. Because actual traffic volume varies with the time of day, it is possible to determine those time periods during which lane closures would cause the roadway capacity to be exceeded. The next step would be to determine if the queue formed and the resulting delay are acceptable. When daytime traffic delays are unacceptable, nighttime work schedules are justified. A general consensus exists that backups can be expected when traffic congestion is around 1,500 vehicles per hour per lane (vphpl) (*4*). These approaches are relatively objective because they do not consider subjective variables such as the needs of the work site, the duration of construction, the effect on motorists, or the reaction of the public. Mathematical models have been developed to describe the characteristics of the vehicle queue (*5*).

From the previous discussion, objective means exist by which to measure the impact of roadway constrictions imposed by construction work. In specific instances, nighttime work provided cost savings as a result of reduced costs of traffic control and increased efficiency of operations (*4*). Methods for calculating user costs have been developed (*1,6,7*) although user costs are not directly paid by the owner agency and, as a result, may not be dominant factors in traffic decisions.

Although several studies have examined the effectiveness of different traffic control strategies, little has been concluded in terms of night work. Nighttime traffic control is made more complex because of reduced visibility and the possible presence of a greater number of intoxicated or inattentive drivers. Traffic control studies have evaluated the effectiveness of changeable message signs (*8*), appropriate wording on signs (*9*), effectiveness of flags and flashing warning lights (*4*), use of sequential arrow boards and reflectorized cones (*8*), value of using drums and barricades (*4*), merits of employing a uniformed traffic officer (*10*), usefulness of flaggers (*8*), need for a traffic control plan, and use of total road closures (*4*).

Reduced visibility on nighttime projects increases the inherent hazards of construction. Safety has also been addressed by various studies. One study disclosed an increased number of accidents on highway projects, but did not develop any conclusions regarding night work (*11*). Other studies have shown a marked increase in accident frequency on night projects (*12,13*).

Nighttime construction requires the use of artificial lighting in the work place. The lighting must be sufficient to permit clear visibility without creating glare. Some operations are best performed with the use of supplementary hand-held lights (*8*), portable floodlights (*4*), or spot lighting (*12*).

Poor visibility associated with night construction must be addressed to maintain an acceptable level of work quality. Acceptable surface finishes have been achieved through nighttime operations (*10*). Cooler nighttime temperatures offer the best opportunity for placing some types of portland cement concrete during the summer months. For similar reasons, a nighttime schedule is commonly used when using latex-modified concrete on bridge decks.

Noise from a nighttime construction project may be of particular concern if noise ordinances are more stringent at night. A contract may include an incentive provision for the contractor to reduce the noise level (*14*). In addition, various measures can be taken to reduce equipment noise (*15*).

Most construction workers are accustomed to working during the day. A nighttime project may be new to many workers. Nighttime projects incur a larger turnover of workers on the night shift, in spite of the pay differential that compensates the workers for the nighttime inconvenience (*4*).

Productivity is also affected by a nighttime schedule. This effect can generally be minimized with good planning and by staffing the project with a person who has the authority to make decisions for the company (*4*).

Various factors have been identified in the literature as being affected, either favorably or unfavorably, by a nighttime schedule. However, no study was identified that incorporated all of these variables and enumerated the relative importance of each in the decision-making process concerning nighttime construction.

RESEARCH METHODOLOGY

A two-part questionnaire was distributed to state highway agencies. Part I was designed to be completed by someone familiar with the decision-making process when a nighttime construction schedule was contractually required, such as a state transportation planner or a district construction engineer. This portion of the survey included questions on the amount of nighttime construction that is contractually required on state roadway projects, future projections of the amount of nighttime construction to be used in the state, a ranking of the most important variables to be considered when deciding on nighttime construction, the nature of the analysis to assess the merits of a nighttime construction schedule, and the cost impact of a nighttime construction schedule.

Part II did not address the decision-making process regarding nighttime construction but rather the various factors that are involved once a nighttime construction schedule is used on a project. Ideally, Part II was assumed to be completed by individuals with field experience involving nighttime construction, such as project engineers and resident engineers. These individuals have direct experience with planning and monitoring construction activities. This portion of the survey included questions on the ranking of the most important factors affected by nighttime construction schedules, including such variables as traffic control, safety, noise, productivity, project costs, and community impacts. Information was also requested on the most effective means to ensure safety for workers and nighttime drivers. Additional questions addressed unique problems encountered by nighttime construction schedules, the presence of any advantages of nighttime work, the relative quality of paving work performed at night, and specific cost differentials for performing certain tasks at night. The last question of the survey asked for the name of a contractor known to have experience with nighttime construction work.

The survey was sent to the 50 state highway agencies. In total, 30 responses from 21 states were received for Part I and 36 responses from 19 states were received for Part II. Twenty-three states were represented by the survey respondents because some states only responded to one portion of the survey. Multiple responses were received from three states.

Before compiling the responses, an examination was made to determine if multiple responses from states should be treated differently from single ones. This analysis revealed that multiple responses from states were not different from the single responses received from the state agencies. For this reason, the final analysis did not consider the differences between states, but treated all responses as being unique responses, regardless of respondent location.

Another survey was prepared to obtain information from contractors about nighttime construction. The contractors included in this study were those who were identified as being experienced with nighttime construction by respondents to Part II. Because no known study on nighttime construction had included contractors, a less formal format was seen as being most effective. Thus, the contractor portion of the study was conducted on the telephone. Questions were asked about contractor rationale for electing to perform nighttime construction work, types of projects best suited for nighttime scheduling, most effective traffic control devices, cost differential for various types of work performed at night, and particular problems encountered with night work. Eighteen contractors from 11 different states were interviewed.

DECISION TO REQUIRE NIGHTTIME WORK

Responses to Part I reflect the experience of 30 respondents each of whom award, on the average, more than 150 contracts per year. Each respondent awards about three contracts per year requiring work to be performed at night. The nighttime construction contracts, totaling 173 for the respondents, were valued at more than $1.5 billion, with the average per-contract value being just under $9 million. An additional three night projects per respondent were performed each year, although the nighttime schedule was not contractually required. Over the past 5 years, 408 projects, valued at nearly $3 billion, were awarded in which nighttime construction was contractually required and another 364 projects were completed in which the contractors elected to use a nighttime schedule. The total number of nighttime construction projects has remained relatively constant over the past 5 years. However, the respondents projected that approximately 195 nighttime construction contracts would be awarded in the next 2 years, indicating an increase in the use of contractually required nighttime construction work (see Table 1).

Several respondents pointed out that the distinction between contractually required nighttime projects and projects in which the contractors elected to perform the work at night may be subtle. For example, a contract may not specifically state that nighttime work must be performed, but instead may stipulate that lane closures cannot occur during certain peak traffic hours. The result is that the contractor is essentially bound to perform the work at night in response to the lane closure limitations. The completion date, coupled with the unique traffic handling requirements, can be construed to dictate a nighttime work schedule.

Because nighttime construction work was contractually required by most (approximately 70 percent) of the respondents, questions were asked to ascertain the relative importance of the different factors considered during the decision-making process. Twelve factors meriting consideration were identified through the literature review and personal interviews. These factors were then rated on a scale of 7 by each of the respondents. A rating value of 1 indicated a low importance for the factor, whereas a value of 7 represented the highest level of importance that could be assigned. The summary of this factor rating is presented in Table 2.

TABLE 1 GENERAL DESCRIPTION OF THE RESPONDENTS TO PART I OF THE STATE HIGHWAY AGENCY SURVEY

Total Number of Respondents	30
Number of States Represented by the Respondents	21
Number of Construction Contracts Awarded per Respondent	161 Contracts/Year
Number of Construction Contracts Requiring Nighttime Work Awarded in the Past Two Years per Respondent	3.1 Contracts/Year
Average Value per Nighttime Contract	$8.9 Million
Number of Construction Projects on Which the Contractor Elected to Work at Night in the Past 2 Years	3.0 Projects/Year
Number of Construction Contracts Requiring Nighttime Work Awarded in the Past 5 Years per Respondent	3.1 Contracts/Year
Average Value per Nighttime Contract Awarded in the Past 5 Years	$7.1 Million
Number of Construction Projects on Which the Contractor Elected to Work at Night in the Past 5 Years	3.3 Projects/Year
Number of Construction Contracts Requiring Nighttime Work Expected to be Awarded in the Next 2 Years per Respondent	4.2 Contracts/Year
Average Value per Nighttime Contract to be Awarded in the Next 2 Years	$6.5 Million

From the rating values provided, traffic congestion receives the most important consideration. Safety is also considered to be important, whereas the cost of the nighttime project to the owner is of relatively low importance in the decision-making process. One respondent indicated that safety was a serious concern because reduced traffic congestion associated with a nighttime project caused driver speeds to be higher. This condition is aggravated by the presence of greater numbers of intoxicated or inattentive drivers.

Because the importance of congestion to the decision-making process was anticipated, questions were asked about the type of analyses conducted when the decision is made on whether nighttime construction work will be required. An objective analysis, similar to that described earlier, is employed by four of the respondents. When compared with other respondents, these four gave higher ratings to the importance of safety, productivity, and noise. However, the importance of user costs was rated lower by those performing a detailed analysis.

Empirical formulas and agency experience are two means of estimating roadway capacity. Fourteen of the respondents use both approaches, eight make exclusive use of the *Highway Capacity Manual* (*2*) equation, and five indicated that they relied solely on experience (three provided no response). One

TABLE 2 RATINGS OF THE IMPORTANCE OF FACTORS TO CONSIDER WHEN DECIDING ON NIGHTTIME CONSTRUCTION CONTRACTS

Factor	Average Rating
Congestion	6.72
Safety	5.93
Noise	5.31
Work Time Available	5.21
User Cost	5.14
Quality	4.93
Light Glare	4.66
Productivity	4.29
Agency Experience	3.79
Contractor Experience	3.43
Temperature	3.38
Owner Cost	3.07

TABLE 3 TYPES OF INFORMATION USED TO QUANTIFY TRAFFIC CONGESTION

Type of Information	Number of Respondents[1]
Current Traffic Volume Counts	23[2]
Past Experience	20
Queue Theory	9[3]

[1]A total of 27 responses were received to this question
[2]Five respondents use this method exclusively
[3]Traffic counts and past experience are also used.

TABLE 4 RATING OF CONCERNS WHEN DECIDING ON PARTIAL VERSUS TOTAL ROAD CLOSURES

Factor	Average Rating
Congestion	6.79
Detour Routes Available	6.24
Duration of Project	5.69
User Cost	4.55
Owner Cost	4.38

respondent also mentioned that actual counts by time of day were useful for determining highway capacity.

Level of congestion can be quantified objectively or subjectively. Objective analysis includes use of either current traffic volume counts or queuing theory. Use of past experience or judgment is a subjective approach. The 27 respondents to this question indicate using volume counts is the most common, with 23 (85 percent) using volume counts and 5 (18 percent) of the respondents making exclusive use of this method. Queuing theory is used by 9 (33 percent) of the respondents, whereas 20 (74 percent) use past experience. All three approaches are used by 32 percent of the respondents (see Table 3). Multiple responses from some states were not the same for all respondents within each state, implying that differences exist between the districts within the states.

Regardless of the means used to predict level of congestion, several respondents were reasonably consistent in considering nighttime construction to be appropriate when congestion approached 1,500 vphpl. Consistent with the literature, 10 of the respondents consider night work to be justified when congestion is in the range of 1,200 to 1,600 vphpl. The actual range of this value for all respondents was from 800 to 1,900 vphpl. Experience is also used in this determination without specifying the actual level of congestion.

A question was asked about determining whether construction should be done with total or partial road closure. Factors generally considered include anticipated level of traffic congestion, duration of the project, existence of a feasible detour route, cost of the project to the owner, and cost impact on road users. Respondents were asked to rate the relative importance of these factors on a 7-point scale. As presented in Table 4, level of congestion and availability of a detour are the most important considerations. For respondents conducting detailed analysis, importance of the level of congestion is rated even higher. In fact, respondents performing detailed analyses rated all of these factors as being more important.

One question concerned the cost of nighttime construction projects. In order to quantify costs, identifying specific cost items affected by the nighttime schedule would be necessary. When asked if any specific pay items for nighttime work existed in the pay schedules, only three respondents gave affirmative answers. Thus, costs of specific items of work required by a night schedule cannot be readily retrieved by most state highway agencies because the costs of nighttime construction are generally absorbed in other pay items. The lack of this type of information is consistent with the low level of importance placed on owner costs.

Safety of roadway construction depends to some extent on the public information system used to notify motorists, police, fire, and ambulance services about upcoming road and lane closures. This information permits users to identify alternate routes, reschedule their travel accordingly, or expect delays. The notification system varies by state. The most common means of disseminating information is through newspapers, radio, and television. Many agencies have a public information division or press office that is responsible for notifying the media concerning construction that will affect the public. In some cases, contractors are required to notify the public information office or the media directly. A time frame of from 24 to 72 hr before a road or street closure is usually specified. In some cases, notification 1 to 2 weeks before closure of a street or lane may be necessary. In several states, personnel with emergency units are invited to a preconstruction meeting in which they are fully apprised of the impact of the anticipated work. One state agency writes a weekly newspaper column, which publicizes information about road and lane closures. Information is also spread with the use of changeable message signs, billboards, or special road signs.

FACTORS AFFECTED BY A NIGHTTIME CONSTRUCTION SCHEDULE

Part II of the state highway agency survey focused on primary concerns of using a nighttime construction schedule. A wide variety of projects were selected on which the experience of the respondents was based, including new construction with asphalt and portland cement concretes, bridge deck overlays or replacements, asphalt concrete overlays, portland cement concrete removal and replacement, roadway maintenance activities, and various combinations. Paving with an asphalt concrete overlay was the most common activity performed at night.

Respondents were asked to provide answers to a variety of questions pertaining to projects with which they were personally familiar. Respondents were asked to rate the relative importance of a number of planning concerns when nighttime road closure projects were being scheduled (see Table 5). Traffic control was given the highest rating, followed by concern for worker and driver safety. With partial closure, the same three factors were seen as dominant concerns. However, concern for driver safety was rated the most important. Other concerns about night work specifically mentioned included amount of work, time frame of road or lane closure, public notification, quality of workmanship, and weather.

Road closures, whether total or partial, require that careful attention be paid to the means used to control the traffic. Various types of traffic control devices and methods of providing for worker safety are available. Respondents were asked to provide their perception of the relative effectiveness of various selected methods on a 7-point scale (see Table 6). For total and partial road closures, lighting was considered to be the most effective means of providing for the safety of workers and drivers. Signs and sequential arrow boards were also given high ratings for total and partial road closures. Use of safety vests and physical barriers was rated as having greater importance on partial road closure projects. Other suggested methods include the use of reflective channel devices, reflectorized barrels, and flashing lights. Some respondents mentioned that the effectiveness of some methods may be considerably enhanced when one of the other approaches is also used, presenting difficulties in judging a single approach.

In situations where changeable message signs were used, respondents were asked to indicate the most effective wording they had observed. Approximately two-thirds of the respondents preferred wording such as Right (Left) Lane Closed, Merge Left (Right) over less specific wording such as Night Work Ahead and Construction Ahead. Speed reduction signs and specific instructions for drivers such as Exit Available to Avoid Construction were mentioned as helpful in controlling traffic.

TABLE 5 RATING OF VARIABLES TO CONSIDER WHEN PLANNING NIGHTTIME WORK

PLANNING CONCERN	TOTAL ROAD CLOSURE	PARTIAL ROAD CLOSURE
Traffic Control	6.76	6.61
Safety of Workers	6.53	6.57
Safety of Drivers	6.47	6.66
Community Impact	5.79	5.33
Productivity	4.94	4.89
Project Cost	4.44	4.33
Noise Level	3.85	3.89

TABLE 6 RATING OF TRAFFIC CONTROL DEVICES ON NIGHTTIME CONSTRUCTION PROJECTS

Traffic Control Device	Total Road Closure	Partial Road Closure
Lighting	6.39	6.59
Signs	6.24	6.41
Sequential Arrow Board	6.06	6.41
Physical Barriers	5.94	6.09
Changing Message Sign	5.76	5.91
Safety Vests	5.48	6.32
Reflectorized Cones	5.31	5.91
Police Patrol	5.06	5.65
Flaggers	3.94	4.91

Aside from traffic control, nighttime construction projects present new challenges for these projects. Several questions were asked to determine the extent that nighttime construction altered or affected the quality of the work performed. One question concerned the quality of asphalt concrete paving when performed at night. Seventy-six percent of the respondents felt that the quality of asphalt concrete paving work performed at night was lower than the quality of similar work performed during the day. Only one respondent felt that the quality of night work was better. A typical response was that "it is very hard to control the placing and finishing of asphalt and get a good job." Two respondents indicated that, in their respective states, the wearing course was never placed at night.

When asked about the principal problems encountered with the nighttime placement of asphalt concrete (AC) paving, "unevenness of the paving surface" was the dominant (79 percent) response. Inconsistency in the mix, poor compaction, and cold joints were mentioned as problems by about 20 percent of the respondents. Most of these problems stem from the difficulty in providing sufficient lighting for the site. Inspection of the work is a problem for a similar reason. Other difficulties mentioned include shadows, less control of tack spread, a longer waiting period for the tack to break, asphalt dropping from trucks onto the pavement that cannot be readily seen at night, alignment, repairing roller marks, and scheduling work forces.

Similar questions were asked about the night placement of portland cement concrete (PCC). Most respondents (63 percent) indicated that the quality of PCC placed at night was below that of materials placed during the day. The overriding problem, noted by 85 percent of the respondents, was the difficulty of obtaining a good finish. Problems of cold joints and mix inconsistencies were noted by only two respondents. The root source of the problems with placing PCC at night is the issue of insufficient lighting. In addition, cooler nighttime temperatures limit PCC placement time in some areas. However, during summer months nighttime work offers more comfortable working temperatures, better workability and curing of concrete, and easier material delivery with the absence of daytime traffic congestion.

Other typical road work activities were identified and the respondents were asked to rate, on the 7-point scale, the relative quality that could be expected on those work items if performed at night (see Table 7). The best quality was apparently achieved in the compaction of the subgrade (rated 4.88), whereas the lowest quality attained occurred with crack sealing on AC pavement (rated 4.10) and crack sealing on PCC pavement (rated 4.19).

Apparently, nighttime construction is different principally because of the absence of natural lighting by which to observe the work activities being performed. Because some artificial lighting must be provided in order for night work to proceed the survey asked whether lighting intensity to be provided was stipulated in the contract documents. Eighty-two percent of the respondents did not specify a minimum level of lighting intensity. Respondents who did have specifications for minimum acceptable lighting intensity also rated lighting as being a particularly important safety measure. Although lighting intensity was specified by some respondents, foot-candle intensities specified were too varied to provide any meaningful guidance. These respondents also rated barriers as effective safety devices on road closures. Incidentally, these were the same respondents who indicated that better quality performance was realized with subgrade compaction, asphalt concrete surface compaction, AC pavement crack seals, and PCC pavement crack seals.

Various disadvantages of performing construction work at night have been presented. Respondents were also asked to identify tasks for which performance was improved at night (Table 8). Although no single task was agreed on by a majority of the respondents, some tasks were mentioned more frequently than others, notably AC paving, bridge deck rehabilitation, pavement marking, and PCC paving. In most cases, the advantage of a night project tends to be associated with cooler temperatures and more efficient delivery of materials because of reduced traffic.

Regardless of the quality of work achieved, cost differentials exist for work performed at night. Respondents were asked to estimate cost differentials for a selected number of items (see Table 9). Although Table 9 is interesting, the information must be recognized as being variable. Most respondents gave estimates for various cost differentials; however, each estimate was based on a specific project. Accurate estimates are difficult to ascertain when portions of a project are performed during the day and when added costs of night construction are buried in other pay items. Cost values provided should be useful to provide an order of magnitude estimate of the added costs of night construction, recognizing that the costs may vary considerably with the type of project and the locality. The overall cost differential for total project costs is 9 percent. This increase may be attributed, in large part, to the other cost items noted in the table. One respondent stated that night construction was more costly because night work increased project duration, which contributed to a 30 percent cost increase in administration costs.

TABLE 7 RATING OF QUALITY ACHIEVED IN SPECIFIED OPERATIONS PERFORMED AT NIGHT

Operation Performed at Night	Quality Rating
Compaction of Subgrade	4.88
Surface Compaction of Asphalt Concrete	4.77
Installing Guard Rails	4.73
Placing PCC Pavement	4.54
Pavement Marking	4.36
Crack Sealing on PCC Pavement	4.19
Crack Sealing on AC Pavement	4.10

TABLE 8 HIGHWAY CONSTRUCTION TASKS PERFORMED BETTER AT NIGHT

Construction Task	Number of Respondents Naming the Task
Asphalt Concrete Paving	6
Bridge Deck Rehabilitation	5
Pavement Marking	5
PCC Paving	4
Demolition	3
Setting Girders	2
Concrete Joint Repairs	2
PCC Pavement Spall Repair	2
PCC Pavement Slab Repair	2
Latex Modified Concrete	2
Grooving PCC Pavement	1
Large Concrete Pours	1
Asphalt Concrete Removal	1
Loading and Hauling Dirt	1
Preparing for Structural Pours	1
Backfilling Structures	1
Shifting Traffic	1
Installing Guardrails	1
Signals	1
Street Lighting	1
Grading/Crushing Aggregate	1

TABLE 9 ESTIMATED INCREASE IN THE COST DIFFERENTIAL OF PERFORMING WORK AT NIGHT

Project Cost Item	Increase in Cost
Lighting	63%
Traffic Control	28%
Engineering Inspection	22%
Labor (Shift Premiums)	18%
Overtime (Agency Personnel)	16%
Material Costs	5%
Total Contract Amount	9%

RESULTS OF CONTRACTOR INTERVIEWS

Information obtained from the contractors was less extensive than that received through the state highway agency surveys. However, some interesting comments were given. The focus of the interviews was on issues of safety, innovations, and problems associated with nighttime construction.

The contractors represented a diverse group, ranging from firms involved solely with projects performed at night to those who performed only 5 percent of their work at night. For the sample, the average contractor performed about 50 percent of the work at night. From these statistics and supported by their comments, some contractors prefer nighttime construction projects, whereas others prefer daytime projects.

Nighttime projects ranged in duration from 2 months to as long as 3 years. Longer projects reported about 25 percent nighttime work, whereas shorter projects had a much greater extent, about 80 percent. Contract values ranged from $300,000 to more than $200 million. On these projects, about 40 percent of the contractors made the decision to work at night, about 40 percent were contractually obligated to perform the work at night, and the remainder made a joint decision with the state highway agency on the nighttime schedules. Some key factors in deciding on a nighttime schedule included less traffic, better production, more work-time available, early completion incentives, and greater ease of getting materials delivered.

Contractors were asked to describe the types of work performed at night (see Table 10). The most frequently mentioned projects involved AC paving, PCC paving, and bridge decks. Contractors indicated that on about 70 percent of the projects the traffic control plan used was specified by the state highway agency. On 20 percent of the projects, contractors worked with the state agency to develop an acceptable traffic control plan. For the remaining projects, development of the traffic control plan was the sole responsibility of the contractor. In establishing a traffic control plan, contractors indicated that the most effective traffic control devices were arrow boards (see Table 11). Other effective means less frequently mentioned included lighting, police patrol cars, and changeable message signs. Contractors mentioned problems with some of the devices. Some stated that arrow boards were generally not placed for the best effect and become visible only after the driver was in the work zone. Others stated that drivers often failed to read the messages regarding traffic control.

TABLE 10 TYPES OF WORK PERFORMED AT NIGHT AS REPORTED BY CONTRACTORS

Type of Work	Number of Responses
AC Paving	8
Bridge Decks	7
PCC Paving	6
Hauling	3
Structures	3
PCC Pavement Repair	2
Pile Driving	2
Milling	2
Grading	1
Drainage	1
Excavation	1
Utilities	1

TABLE 11 EFFECTIVE TRAFFIC CONTROL DEVICES AS REPORTED BY CONTRACTORS

Traffic Control Device	Number of Responses
Arrow Boards	12
Lighting	7
Police Patrol Cars	6
Changeable Message Signs	5
Physical Barriers	3
Reflectorized Barrels and Drums	3
A-Frame Barricades	2

Most contractors stated that nighttime work was dangerous. In order to counteract the dangers, workers must wear reflective clothing and the site must be well lighted. For worker safety, the workers should be separated from traffic by physical barriers. Several contractors stated that contract provisions may limit lane closures to the one lane on which work is being performed. For improved safety, these contractors would like to be permitted to close an additional lane. Another common problem is that drivers fail to adequately reduce their speeds consistent with the needs of the project. More than 75 percent of the contractors stated that worker morale is not a problem. One contractor stated that wage premiums, more time off, short-duration projects, and a regular night schedule were important means by which worker morale and productivity could be maintained. This comment implies that worker morale might become a problem if specific measures are not taken.

Contractors were asked to estimate the cost differential for performing construction work at night. Average overall added cost for nighttime construction projects was estimated to be about 10 percent, a value consistent with the estimate provided by the state agency respondents. Most of the added costs were attributed to the premium wages paid for shift work. Materials were an added cost only for those few contractors who did not have their own batch plants. Material costs could rise significantly if more expensive materials, such as a fast-setting patching product, were required as a result of a night schedule. Traffic control was not generally mentioned as being more expensive at night. One contractor stated that lighting (an expenditure not encountered on daytime projects) could constitute a 1 percent cost increase on the total contract.

Contractors were also asked to comment on any problems that were specific to nighttime construction. Typical comments listed quality control problems stemming from poor visibility, shadows, material mix inconsistencies, and cold temperatures. The danger posed by intoxicated or inattentive drivers was a cause of common concern for most contractors. Lane closure schedules pose unique problems because workers can only access the site after the closure and because the work site must be vacated when the roadway or lane is reopened to traffic.

CONCLUSION

Information has been added to the existing literature on nighttime construction. Some of the results support various findings of past research studies, whereas other results are unique. The literature describes several types of detailed analyses that can be used to evaluate nighttime construction alternatives; however, the findings show that these approaches are used by only a small number of state agencies. Decision making tends to focus on traffic congestion as the most important factor, but the anticipated level of congestion is largely based on experience and judgment. Little use is made of objective analysis, which indicates that state agencies assume that a good intuitive understanding of congestion exists.

Consistent with the literature, congestion and safety were found to be the primary factors for deciding on a nighttime construction project. When the impact is considerable, either factor may dictate that nighttime construction is appropriate. Cost of the project to the owner is regarded as being of little importance, whereas user costs, which are directly related to congestion, are given careful consideration. Noise was also found to be a relatively important factor in the decision-making process. In fact, where specific nighttime noise levels are prescribed by local governments, nighttime construction may be precluded unless remedial actions to reduce the noise levels are undertaken.

When contractors elect to perform work at night, their decision is based primarily on production and scheduling. As noted in the literature, no detailed methods exist by which scheduling problems can be mitigated. Difficulties in scheduling are presented by the time required to set up traffic control devices and to remove them before reopening the traffic lanes. Scheduling becomes more severe when the rush hour or peak levels of traffic develop early in the day and extend into the night hours. Night hours must also be coordinated with state personnel, for example, inspectors and project engineers.

Costs associated with nighttime construction were examined. State agency estimates indicated that total project costs could be expected to increase by about 10 percent when nighttime construction is performed. Interestingly, contractors provided a similar estimate of the cost of nighttime construction. This cost is lower than most cost estimates of nighttime construction noted in the literature. Perhaps these values are reasonably accurate and indicate why owners do not regard owner cost as being important.

The sources of the added costs of construction were examined. One cost that tends to be unique to nighttime construction is the need for artificial lighting. In order to meet lighting requirements, contractors can add lights to their equipment or obtain a variety of other types of lights that can be placed on the job near the primary work activity. Equipment modified with lights provides the added benefit that the setup cost is incurred only one time. Unfortunately, little information is available in the literature regarding cost-efficient, safe, and effective means of equipping paving machinery with special lighting.

State agency personnel estimated that material costs were increased by 5 percent when nighttime construction took place. Contractors, on the other hand, did not give estimates that were consistent with these values. Most contractors stated that material prices did not increase when work was performed at night. Cost of materials are not increased because most contractors have their own batch plants and thereby establish greater control over the material. Because streets are less congested at night, the actual cost of material delivery may be reduced on night projects.

The literature suggests that more traffic control should be used for nighttime road construction than for daytime work. State personnel and some contractors estimated that the added costs of traffic control on a nighttime project were about 25 percent higher. However, many contractors indicated that they emphasized traffic to an equal extent for daytime and nighttime construction. Traffic control is important regardless of the time of day. Reduced speeds through the construction work zone and a well-lighted site can add considerably to the safety of construction workers and drivers. The issue on which there is disagreement between state personnel and the contractors is whether costs are increased by day or by night to get drivers to observe precautions.

The literature indicates that changeable message signs are effective for traffic control in a nighttime construction work zone. Respondents do not fully agree. The state agency respondents indicated a preference for changeable message signs with simple messages, such as Night Work Ahead. Use of such signs by the contractors is relatively low. Perhaps the information about these signs is not well disseminated among the firms that could benefit from them.

Flashing warning lights are another traffic control mechanism to get the attention of drivers. Survey respondents and the literature agree on the value of flashing warning lights. One particular type of warning light, found on police patrol cars, was noted in the literature as being effective for traffic control. However, survey respondents did not evaluate this as an effective means. Perhaps the respondents had little experience with the use of police patrols on construction sites.

The majority of the state highway agencies agree that the difficulty of providing adequate lighting has an adverse impact on quality. Although the quality may meet established standards, defects in the finished surface for both asphalt and PCC pavement are apparent. Defects appear to be a tradeoff that agencies are willing to accept in exchange for reduced congestion.

From the literature, public information seems to be an important means of reducing problems of nighttime construction. Respondents also agreed that public information is helpful. Various means of disseminating information such as newspapers, radio, and signs may all be used to reach different audiences.

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Current Survey of Computer Status in the U.S. Construction Industry

Fazil T. Najafi

Computer technology changed drastically during the 1980s. Microcomputers now perform functions that only large, expensive mainframe computers performed a decade ago. Computers are used by a rapidly growing majority of construction firms. A study was conducted to investigate computer applications in various aspects of the construction industry, such as planning, surveying, design, graphics, prebidding, budgeting, scheduling, quality and cost controls, and equipment management. A questionnaire was sent to various companies in the construction industry concerning (a) the type of construction, (b) the annual dollar volume of business, (c) the types of computers used, (d) the types of software used, (e) the percent of time a computer is used to perform various construction tasks, and (f) the effect of computers on the advancement of the construction industry. The results indicated that minicomputers are used predominantly in all types of construction tasks. Commercial software is more commonly used and requires constant revision and improvement. Software problems are the major cause of user dissatisfaction. In general, the computer has benefited the construction industry, and the number of users is increasing. The computerization of scheduling techniques and construction material codings has helped improve the speed and accuracy of computer data entry. The construction industry needs to enhance user capabilities to save costs.

The main objective of this study was to identify the extent to which the computer has enhanced business in the construction industry. Additional objectives were to identify (a) reasons why some businesses do not use computers, (b) the percent of computer use in various aspects of construction activities, (c) the types of computers and software being used, and (d) problems with available software.

The construction industry is fragmented, sensitive to economic cycles, and highly competitive. A contractor is far more at risk in this industry than in almost any other (*1*). The number of yearly business failures in the construction industry increased by 484 percent between 1978 and 1986 (*2*). Because of the competitiveness that exists in the construction environment and the concern for time and cost savings, computers are being used by a rapidly increasing majority of construction firms.

The availability of microcomputers with increased capacity, low cost, user-friendliness, and a menu-driven nature encourages those in the construction industry to use computers in various activities of construction projects. Costs range from under $1,000 for an XT-compatible machine to $2,000 for an AT, and up to $10,000 for a 386-based machine. Price depends on the computer's standard features, such as the central processing unit (CPU), size of the random access memory (RAM), and disk operating system (DOS). The applications of a personal computer (PC) range from word processing to running simple design programs. A PC system is expandable and can thus meet the changing needs of its user.

A literature search was conducted, along with a telephone interview, to identify computer applications in performing different construction tasks. A questionnaire was then prepared and mailed to various U.S. construction firms.

The questionnaire results indicate that construction firms predominantly use minicomputers. The commercial software commonly used requires constant revision and improvement. Software problems are the major cause of user dissatisfaction. The computer has helped the construction industry, and the number of users is increasing. Computer training is essential for users in the construction industry to optimize their computer skills.

LITERATURE SEARCH

The purpose of the literature search was to identify computer applications in various aspects of construction activities. In the past, engineering and construction applications of the computer were adaptations of manufacturing and business management programs. These programs did not take into account the unique site-specific nature of design and construction. As a result, commercial software never fully met the needs of the construction industry.

With sales of computer-aided design and drafting (CADD) applications reaching $120 million annually, vendors of hardware and software systems have recognized the potential in other areas of construction (*3*). Bookkeeping was one of the earliest applications because it is a basic business need. But with growing demand, more versatile programs have become available, especially for the small computer. A major difficulty in buying a software package for a particular need is finding out what programs are available. User groups, newsletters, and current reference books are excellent sources of information.

A variety of computer applications for the construction industry are described in the following subsections. Commercial software is not yet available for some of these applications (*3*).

Planning

In the construction industry, computers are used for a number of planning activities, including financial feasibility analysis, traffic flow studies, optimizing land or structure use, space

Department of Civil Engineering, University of Florida, 345 Weil Hall, Gainesville, Fla. 32611.

requirement projections, mathematical modeling, and geophysical and seismic data analysis.

Construction Management

A major area in construction applications is construction and project management. Construction management software is available through the University of Florida's McTrans Center, a software distributor and user support center for FHWA. Some examples of the construction-related programs at this center are Quality Level Analysis (QLA), Highway Design and Maintenance (HDM), and Project Tracking System (PTS). Construction scheduling software include Primavera, Harvard Total Project Manager, Microtrak, Timeline, Superproject Plus, Pertmaster Advance, and Quicknet Professional. Each of these software packages allows importing, exporting, and data modification in a dBase or LOTUS 1–2–3 format. This capability allows the owner to perform additional analysis or modification to the project data and then import this information back into the source scheduling software. These software packages can handle 1,000 to 10,000 different activities and all of them can generate bar charts.

For instance, the QLA program statistically estimates the degree of conformity of construction materials to specification requirements. Quality control test results are entered as well as specification requirements and tolerance levels. QLA can predict whether construction materials and methods will meet owner specifications.

An example of quality assurance is the production of concrete through automated concrete-batch plants, which enable the operator to request any of several predefined mixes. The computer operates the plant until the correct mix is discharged into a waiting concrete truck. Batch information is then printed out, and copies are given to the truck driver to take to the point of delivery for an inspector's approval before the concrete goes into the pourer. Copies are attached to the samples made at the pour site. The loop is closed after testing, when sample results are logged and sent back to the quality assurance department. Similar applications are gaining acceptance in such areas as welding and asphalt paving.

For high-quality concrete in batch plants with higher volume, such as nuclear power stations, a computer controls the selection, transport, weighing, charging, and mixing of cement, sand, aggregates, water, and admixtures for batches that must meet specified design criteria for a specified structural component. The computer simultaneously handles administrative reporting for delivery, quality, and cost control.

Another area in construction management is cost estimating. The difficulty in this application is establishing and maintaining an accurate data base of construction materials, labor, and equipment cost (or in-place cost). Software is available to allow some sophisticated applications for material take-off. One estimating system automates the drawing of take-offs through the use of a sonic digitizer, which replaces conventional scales, tape measures, and measuring wheels. The digitizer calculates coordinates from the sound impulses of a stylus drawn across the drawing, feeding the data directly into the computer. Once the drawing is digitized, the cut-and-fill volumes and costs, site-preparation costs, and preliminary building estimates are automatically calculated (*3*).

A similar system solicits information from the estimator by asking questions to be answered from construction drawings. Questions are grouped around the pertinent sets of dimensions to save computer memory. The system lists possible answers for each question, with the most likely choice starred; the answer is usually entered by typing a single digit. Once the cost estimate is produced, bid solicitation postcards are printed automatically for each phase of the project using the system's file of eligible subcontractors. Other programs allow firms to work totally with their drawings on the screen for material take-offs (*4*). Cost-estimating programs range in a broad spectrum of capability and are available for all sizes of computer systems.

Job-site cost control is another area of concern in construction management. Small, on-site computers can organize a job's lost-time information on Friday afternoon, allowing the project managers and supervisors to determine problem areas before quitting time. Appropriate information is produced for each level of management, summarizing the week's assessment on a single-page printout (*4*).

A wide range of bar code applications in construction has recently been developed by member firms of the Construction Industry Institute. Specific areas of applications are quantity take-off, field material control, warehouse inventory and maintenance, tool and consumable material issue, time keeping and cost engineering, purchasing and accounting, document control, and office operations (*5*).

Other computer applications for construction management include budgeting and scheduling, CPM/PERT schedules and charts, manpower schedules, progress payment requisitions, change orders, color selection devices for matching building painters' materials inventories, manpower and machinery allocations, operation and phase scheduling, record keeping, and progress reports.

Equipment Management

Computer applications for equipment management include equipment scheduling, replacement allocation, service and repair scheduling, ordering and inventory of replacement parts, and small tools and hardware distribution.

Most major construction equipment manufacturers are experimenting with, and even producing, machines that include on-board microprocessors for monitoring performance, maximizing engine power and fuel economy, optimizing gear shifts, and keeping loads within safe tolerances.

Surveying

Computer programs are used for such surveying activities as distance and bearing traverse measurements, coordinate geometry, radial distance calculations, contouring, cross-sectioning and profiling, and cut-and-fill calculations.

Survey data gathered by mechanical means or aerial photographs can be entered into a program with subdivision designs. Area maps can then be generated, corrected, and computer drafted.

Automated excavation grade control is now possible through the use of laser surveying equipment combined with electrohydraulic feedback control systems mounted on bulldozers, motor graders, scrapers, and so forth. In applications such as highway grading, constructing large parking lots, and constructing canals, these techniques have reduced costs (in some cases by over 80 percent) and improved quality (*6*).

Office Administration

In office administration, computers help with general accounting, payroll, record keeping, progress payments, financial statements, progressive cost analysis, cash flow control (cumulative labor and materials costs, time and materials costs, profitability reports, expense and budget monitoring, actual versus estimated cost assessment, and project status), materials and supplies purchasing, delivery scheduling, and inventory control.

An example of computers' estimating expenditures during construction is a program based on double-entry accounting, with the added feature of tracking retainage cash flow. This program provides a monthly balance sheet, a profit-and-loss statement, a list of charges to each account (established by the user), and a list of checks written and journal entries made each month. By interactively entering payroll data (question-and-answer entry), a record of costs for the particular job is automatically created, along with an automatic backup record of the postings on a separate floppy disk to prevent loss of data. Accounts payable and payroll transactions are automatically posted to the job cost and general ledgers. Reports track labor, material, and subcontractor costs by category and by job. One report shows the status of every unpaid charge, not just the current month's transactions. Social security and unemployment taxes, both state and federal, are produced in a report and can be automatically charged to the accounts defined by the user. The program also produces a report for each job that itemizes income and expenditures. It indicates the amount that can be billed for work accomplished and the amount in retainage, and can flag any items over the designated budget (*7*). These are the types of features most contractors need in a business application. This particular program is designed for a small computer; however, the same type of software can be found in larger systems.

Another example is a time-sharing system, in which multiple users rent computer time and the user's microcomputer is tied into a distant, larger computer by telephone and modem. This system provides expanded memory capacity, enabling customers to use the service for many jobs. It is especially advantageous for large jobs so that the job's complete financial history can be followed. Some smaller computers simply do not have the necessary memory.

Design and Graphics

In the construction industry, design applications include analysis for shear, moment distribution, axial force, and sidesway; selection of structural members (reinforced and prestressed concrete, steel, or composite sections); design of beams, columns, slabs, frames, arches, foundations, footings, formwork, and shoring; load analysis for conveyors, chutes, piping, scaffolding, and other construction support systems; design of heating, ventilating, and air conditioning systems; and pipe network balancing. Computer graphics programs are used for such applications as plot maps, topographic maps, project layouts, as-built drawings, project plans, schematics, detail drawings, shop drawings, and materials tables.

For example, a computer-aided design (CAD) system allows a drafter to develop drawings through the use of commands. Results are displayed on the terminal screen and can be printed for a hard copy.

Through the interactive control of the user, CAD provides the means to compose original maps and designs, encode existing base maps and drawings, and store, retrieve, and modify maps and drawings. The graphics workstation consists of a digitizer combined with a graphics display (cathode ray) screen, a command menu and input devices, a table, and a keyboard. A number of engineers and technicians can work simultaneously at various terminals, all sharing the minicomputer's memory, storage, and processing capabilities, which include plan preparation, photogrammetric mapping, bridge and roadway design, and other drafting (*8*). CAD produces plans at a faster rate than manually, with higher quality and lower construction costs due to improved and more accurate design (*9*).

It has been claimed that CAD saves as much as 90 percent in conceptual design time, 25 percent in design cost, 30 percent in bidding time, 15 percent in construction cost, and 40 percent in construction loan interest expense (*10*). A CAD program can plan a highway profile by use of topographic data along a right-of-way. Once complete, a graphic view is presented of what the final road will look like when traveling on it.

The New Hampshire Department of Transportation has divided its 16,000-mi highway system into more than 100,000 segments and is entering the data into a Graphics Design System (GDS). These data include hundreds of common attributes, such as pavement type and conditions, traffic volume, accident statistics, maintenance records, geometry, and physical features. The GDS has cut manual drafting time in half, and many jobs are now handled in-house instead of using an outside consultant (*11*).

Word Processing

A final application in construction is word processing, which takes advantage of the computer's ability to manipulate words and characters. The computer can store text, allow editing, and print quality letters, forms, and reports. Word processing programs are ideal for retaining specifications and bid documents, which can be edited later to conform to a new job.

SURVEY ON COMPUTER USE

In an attempt to understand the extent to which the use of computers has enhanced business in the construction industry, a set of questions was prepared to survey computer users within various states (see Figure 1). The literature search helped in the development of the questionnaire by focusing on those construction-related areas in which computers have been used extensively.

QUESTIONNAIRE

1. Please mark the following areas in which your business construction activities are closely related.

   a. Building Construction

      1) residential _____ 2) commercial _____

   b. Highway Construction

      1) local roads, _____ 2) collector roads, _____

      3) arterial roads, _____ 4) interstate, _____

      5) other _____

   c. Heavy Construction

      1) bridges, _____ 2) tunnels, _____

      3) dams _____ 4) other _____

   d. Industrial Construction

      1) light industry _____ 2) heavy industry _____

   e. Municipal Construction _____

   f. Railroad Construction _____

2. Are you a foreign construction firm? a. Yes ___ b. No ___

3. Do you use your own computer? a. Yes ___ b. No ___

4. Are you a builder only? a. Yes ___ b. No ___

5. Are you a design firm only? a. Yes ___ b. No ___

6. Are you both a design firm and builder? a. Yes ___ b. No ___

7. Are you a construction management firm only? a. Yes ___ b. No ___

8. Are you going to purchase another computer? a. Yes ___ b. No ___

9. Do you currently rent a computer? a. Yes ___ b. No ___

10. Are you using a computer? If so, please indicate the percentage of computer application in the following areas of your business.

Computer Application	Percent of Use
Bookkeeping	%
CPM/PERT (Scheduling)	%
Cost Estimating	%
Bid Computation	%
Budget Tracking	%
Designing	%
Surveying Calculations	%
Personnel Listings	%
Equipment Management	%
Project Material Status	%
Project Material Inventory	%
Graphics	%
Job site cost control and time-lost information	%
Pre-bidding	%
Quality Control	%
Quality Assurance	%
Construction Material Mixing	%
Construction Material Coding	%
Grading (e.g., computer mounted on construction equipment)	%
Other, please state _______________	%

11. Indicate the extent to which you are dissatisfied with computer application.

Reasons	Percent of Users Not Satisfied
Software Problems	%
Hardware Problems	%
Lack of Programmer	%
Poorly Trained Personnel	%
Other Reasons	%

12. If computer utilization has advanced your construction business activities, please complete the following:

Size of Company* (please circle)	Answers by Percentage		
	Yes	No	Partially
Small			
Medium			
Large			

* Circle appropriate company size which is based on the following annual $ volume of business:

<u>Small</u> refers to business volume of $0 - $5M/year

<u>Medium</u> refers to business volume of $5M - $50M/year

<u>Large</u> refers to business volume of over $50M/year

13. Indicate the reason and extent you are <u>not</u> using a computer in your business.

Reason	Percent of Non-Users
Expense	%
Lack of Workload	%
Inadequate Training	%
Employees Adequately Handle Workload	%
Other Reasons	%

14. Do you use any of the following computer software?

   a. Primavera®
   b. MicroTrak
   c. Harvard Total Project Manager II®
   d. TIMELINE 3.0®
   e. SUPPERPROJECT PLUS®
   f. PERTMASTER ADVANCE®
   g. QWICKNET® Professional
   h. Survey 3.0
   i. ROAD RUNNER
   j. SURFER
   k. Draftsman
   l. LOTUS 1-2-3®
   m. VisiCalc
   n. QLA
   o. HDM
   p. PTS
   r. Other (please state):

      _______________, _______________,
      _______________, _______________,
      _______________, _______________.

15. What type of computer(s) do you have?

16. Are you satisfied with your hardware? a. Yes ___ b. No ___

17. Have you changed your hardware? a. Yes ___ b. No ___

18. If yes to #17, please state the reason you changed your hardware.

FIGURE 1 Construction industry questionnaire.

The questionnaire identified the respondents by their type of business, their dollar volume of business, and the percent of time their computer performed different tasks. Table 1 presents a breakdown of the responding firms by their business classification.

The survey achieved a 48 percent response, of which 83 percent used computers and only 13 percent rented a computer through time-sharing.

Twenty-two percent of the respondents had annual sales of under $5 million, classifying them as small. Forty-seven percent did $5 to $50 million in annual sales and were classified as medium-sized, whereas 31 percent had annual sales of over $50 million and were classified as large-sized.

Ninety-nine percent of the respondents performed construction activities, which averaged 91 percent of their business workload. Eleven percent conducted design activities, averaging 6 percent of their business workload. Twenty-six percent were involved with construction management, which averaged 30 percent of their business workload, and 7 percent conducted other types of business, averaging 23 percent of their workload.

Thirty-eight percent of the small businesses did not use computers, 13 percent used time-sharing, and 50 percent owned computers. Of those small businesses owning computers, 75 percent had small computers, averaging one per firm. No small businesses owned large computers.

Fifteen percent of the medium-sized businesses did not use computers, 15 percent used time-sharing, and 70 percent owned computers. Of the medium-sized businesses owning computers, 24 percent owned small computers, averaging two per firm. Seventy-six percent of those owning computers had minicomputers, averaging one per firm. No medium-sized businesses owned large computers.

Only 5 percent of the large-sized businesses did not use computers, 2 percent used time-sharing, and 94 percent owned computers. Of the large businesses owning computers, 41 percent owned small computers, averaging eight per firm, 73 percent owned minicomputers, averaging eight per firm, and 32 percent owned large computers, averaging two per firm.

Sixty-three percent of the small businesses, 85 percent of the medium-sized businesses, and 96 percent of the large businesses used computers. Seventy percent of the computer users had purchased commercial software, with 30 percent writing their own programs. Table 2 presents the percentage of use for various computer applications. Only 6 percent of the users indicated that their computer was not being used for its original purpose.

Fifty-eight percent of the users reported that their computer had met their expectations, 33 percent said it had only partially met expectations, and 8 percent said it had not met expectations. Table 3 presents the reasons why users were dissatisfied with their computer. Some of the other reasons for dissatisfaction that were cited included (a) the lack of requirements identification in selecting a turnkey solution,

TABLE 1 BUSINESS CLASSIFICATION OF RESPONDING FIRMS

		Business Category* Sub-Totals						
Location	Members Surveyed	B	H	HV	I	MU	RR	F
Northeast FL	19	15	2	7	5	6	0	1
South FL	20	17	3	5	7	3	0	2
Mid-FL	24	16	5	12	7	10	0	0
Northwest FL	18	10	5	6	6	7	0	0
East Coast FL	29	16	9	9	5	14	0	1
Georgia	16	10	5	6	6	7	2	0
Alabama	24	14	10	16	13	12	5	0
TOTAL	150	98	39	61	49	59	7	4
Percent of TOTAL		65	26	41	33	39	5	3

* Business Category abbreviations:

B - Building construction
H - Highway construction
HV - Heavy construction
I - Industrial construction
MU - Municipal construction
RR - Railroad construction
F - Foreign construction firms

TABLE 2 COMPUTER APPLICATIONS BY USERS

Application	Percent of Use
Bookkeeping	90
CPM/PERT (Scheduling)	30
Cost Estimating	42
Bid Computation	13
Budget Tracking	63
Design	7
Surveying Calculations	3
Personnel Listings	38
Equipment Status	35
Project Material/Status	20
Project Material Inventory	22
Other tasks	13

TABLE 3 REASONS FOR USER DISSATISFACTION WITH COMPUTERS

Reason	Percent of Users Not Satisfied
Software Problems	38
Hardware Problems	15
Lack of Programmer	17
Poorly Trained Personnel	10
Other Reasons	20

(b) incorrect cost information provided by superintendents on each phase of the work, (c) a lack of appreciation of the capability of computers, (d) the limited flexibility of software, (e) the amount of lead time needed to get answers and the restrictive format of answers, (f) project managers' lack of knowledge concerning computers and accounting procedures, and (g) software delivery delays.

Forty-eight percent of the computer users indicated that they would purchase another computer in the future to expand their capabilities, and 52 percent of the users said they would not buy another computer in the future. Eighty-three percent of the computer nonusers indicated that they would purchase a computer in the future, and 17 percent said they did not plan to buy a computer. Table 4 presents various reasons for computer nonuse.

Table 5 presents the respondents' views concerning the computer's role in the advancement of the construction industry. Sixty-nine percent believed that the computer had advanced their business, 4 percent believed it had not, and 26 percent said it had a partial effect.

Sixty-five percent of the respondents commented on the type of software programs they would like to see developed. Some comments clearly came from inexperienced users, whereas other respondents were intimately familiar with computer applications. The major theme in these responses was for better accounting programs, that is, programs that are not as rigid in format and content. Many users asked for basic programs in accounting, cost estimating, and cost scheduling.

Users also indicated the need for software that could handle such functions as (a) accessing historical data, including lost bid comparisons; (b) cost estimating for heavy and highway construction; (c) printing end-of-year government returns; (d) making minor payroll deductions without modifying the program; (e) tracking cost and maintenance scheduling of heavy equipment; (f) tracking work backlog status; and (g) preventing double billing.

Sixty-nine percent of the respondents attributed better project cost control to the ability of the computer to quickly condense large volumes of information, allowing constructive management action. Some respondents indicated that CADD systems have improved working efficiency. Apparently, the computer's major benefit is its ability to provide better job control.

TABLE 4 REASONS FOR COMPUTER NONUSE

Reason	Percent of Non-Users
Expense	0
Lack of Workload	31
Inadequate Training	8
Employees Adequately Handle Workload	46
Other Reasons	15

TABLE 5 EFFECT OF COMPUTERS ON ADVANCEMENT OF CONSTRUCTION INDUSTRY

Size of Company	Answers (By Percentage)		
	Positive Effect	No Effect	Partial Effect
Small	94	0	6
Medium	62	3	35
Large	64	9	27

FUTURE USE OF THE COMPUTER IN CONSTRUCTION

New technology is already emerging with magnetic bubble memory, which is envisioned as plug-in modules with far greater capacity and faster access than traditional floppy and hard disks (secondary memory). Primary memory will grow, eventually requiring that only data be stored in secondary memory. This feature is becoming possible with the emerging 256K RAM chip (*3*). User-oriented features will continue to be added to simplify understanding and operation, with voice and handwritten input being implemented routinely (*3*).

As a result of these technology advances, small computers will be increasingly available and in greater demand, especially by the construction industry. Integrated data bases will become possible on small computers, allowing design, construction, and accounting from a single source. Computers at the job site will likely become commonplace in the construction industry, which is especially significant considering the advancements being made with CAD and the new technology of integrating various software programs. Satellite-linked networks for international firms will become more advanced. There will be no need for hard-copy drawings because field terminals or systems will provide the necessary information. Engineering and construction students will routinely be trained in the computer's use and conversant in its applications.

The trend toward lowering cost may gradually be reversed because of increased sophistication and capabilities (*12*,p.108). The cost for software programming will increase. It will eventually be possible to place 20,000 activity networks on a small computer, with mainframe computer scheduling programs becoming obsolete (*12*,p.108). Programs will proliferate in the industry for every phase and aspect of construction, with construction management receiving more and better tools to increase productivity.

CONCLUSION

The majority of construction firms use computers, and this trend is increasing. Small computers and minicomputers predominate in this industry. Clearly, computer use increases with the dollar volume of business. Commercial software is more commonly used. Bookkeeping is the most predominantly used function of computers, followed by budget tracking, cost estimating, personnel listing, equipment status, and project scheduling.

The majority of users reported that the computer had met their expectations, with very few reporting that it had not. For those not completely satisfied with their computer, software problems were the major cause. The majority of computer users indicated that they would not buy a computer in the future, but nonusers clearly planned to do so. The majority of respondents stated that the computer has benefited construction by providing financial information faster, allowing better cost management. A few added that CAD systems had further improved the industry.

The major reason cited for the nonuse of computers was that employees adequately handled the work load, with lack of work load being second. This response indicates either a lack of knowledge about the computer's ability to increase productivity or a fear of the unknown.

The 1980s were considered by many to be the decade of the computer. A firm's ability to function effectively in the future will greatly depend on its ability to harness computer technology.

The low cost of PCs (approximately $500) is attractive to minority contractors and small businesses. These firms are slowly moving toward the use of computers in their businesses.

In general, contractors must first decide the types of programs they need. A professional can be helpful in configuring a system that meets those needs.

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Fatigue Cracking in Welded Steel Bridges

JOHN W. FISHER AND CRAIG C. MENZEMER

A number of localized failures have developed in welded steel bridge components because of fatigue crack propagation, which in some instances has led to brittle fracture. Increases in the number of trucks and allowable weights has led to a rapid accumulation of loading cycles. A large number of structures experiencing cracking have welded details that have been identified as susceptible to fatigue crack propagation only after they were built. Oversimplification of member interactions and connection behavior allowed by design codes has resulted in a large number of cases of distortion-induced fatigue cracking. Many of the structurally deficient bridges are over 50 years old. Environmental corrosion has built up over time and has caused increasing amounts of damage. As the infrastructure ages and the costs for new construction escalate, maintenance and rehabilitation are becoming increasingly important to the continued operation of the transportation system. Existing bridge structures have been fabricated under a wide variety of practices and operate in a full spectrum of service conditions. As such, no single set of guidelines can adequately ensure the safety and reliability of all the existing structures. Periodic inspection for accumulated damage and deterioration is critical to acceptable operation of the nation's bridges. Continued technology transfer and education of engineers and qualified inspectors will increase the effectiveness of the inspection and maintenance process. Typical types of fatigue damage found in welded steel bridges are reviewed, specific examples are cited, common retrofit procedures are examined, and proper investigation practices are outlined.

Since the early 1960s, a number of localized failures have developed in steel bridge components because of fatigue crack growth that in some instances resulted in brittle fractures. Several hundred bridges are known to have developed one or more types of cracking. Often several types of fatigue cracking developed in a single bridge because different details existed on several types of structures (*1–3*).

The largest category of cracking is a result of unequal out-of-plane displacements, usually across a small unstiffened segment of girder web. Large numbers of distortion-induced cracks of this kind may form nearly simultaneously in a structural system because the cyclic stress is high and the number of cycles needed to produce cracking is relatively small. The problem of fatigue cracking induced by distortion has developed in a wide variety of bridge structures, including suspension bridges, two-girder floorbeam bridges, multiple-girder bridges, tied-arch bridges, and box girder bridges. In general, the cracks formed parallel to the primary tension from the applied loading and were not detrimental to the performance of the structure, provided they were discovered and retrofitted before turning perpendicular to the main stress field.

Another class of fatigue cracks are those related to connection restraint. Use of coped members such as stringers, floor beams, and diaphragms is common in bridge structures in which members frame into one another. When rolled sections are used, these copes are often flame cut, resulting in residual tensile stresses along the cut edge. Often, the residual tensile stresses from the cutting operation approach the yield point of the parent material. With welded built-up members, terminating the flange outside the end connection is not unusual. Increasingly, cracks are being observed at these types of connections (*1,3*). A related type of cracking develops in the end connection angles. End rotation deforms the connection angle out-of-plane. This condition results in cracking of the angle, often at the fillet or the bolt-rivet restraint line. In cases where the angle is relatively thick, rivet or bolt heads may crack off.

Most of the remaining cracks resulted from details that were not known to have such a low fatigue resistance at the time of the original design, such as cover-plated beams, welded flange attachments, and web gusset plates.

LOW-FATIGUE-STRENGTH DETAILS

The possibility of fatigue cracks forming at the ends of welded cover plates was demonstrated at the AASHO road test in the 1960s (*4*). Multibeam bridges subjected to relatively high stress range cycles (12 kg/in.^{2}) under controlled truck traffic experienced cracking after 500,000 vehicle crossings. In general, few crack details were known to exist until inspections revealed a cracked beam in Span 11 of the Yellow Mill Pond bridge in 1970 (Figure 1). Between 1970 and 1981, the Yellow Mill Pond multibeam structures in Bridgeport, Connecticut,

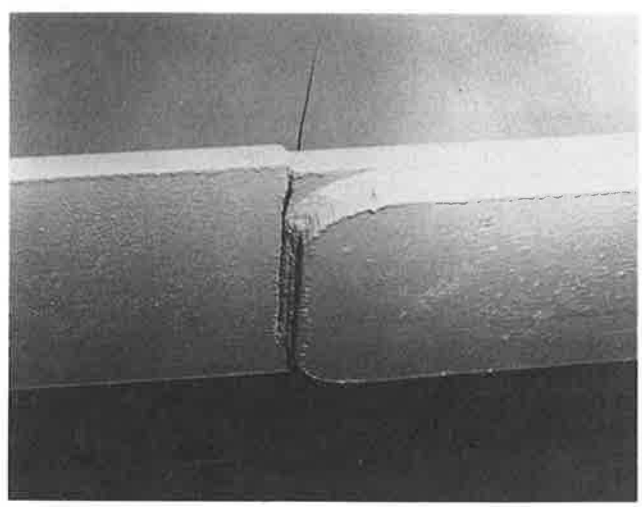

FIGURE 1 Fatigue crack at end of coverplate on Yellow Mill Pond bridge.

ATLSS Engineering Research Center, 117 ATLSS Drive, Lehigh University, Bethlehem, Pa., 18015.

developed numerous fatigue cracks at the ends of cover plates (*1*). These cracks resulted from the high volume of truck traffic and the unanticipated low fatigue resistance of the large cover-plated beams (*5*).

Another low-fatigue-strength detail is the welded web gusset plate shown in Figure 2. These plates are particularly susceptible to crack growth when adjacent, but not attached, to transverse stiffeners and connection plates. A detrimental combination of cyclic stresses results from the expected in-plane deformation of the main girder and from unexpected out-of-plane web gap stress. This latter stress develops from the lateral forces that cause the gusset plate to twist and deform the unstiffened segment of the web between the gusset and transverse stiffener or connection plate.

Low-fatigue-strength details, such as cover-plated beams and welded web and flange gusset plates, should be avoided on new structures that will experience large numbers of stress cycles during their design life.

LARGE INITIAL DEFECTS

Large initial defects and cracks make up the next category of cracked members and components. In several cases, the defect resulted from poor quality welds that were produced before nondestructive test methods were well established. Many of these cracks occurred because the groove-welded component was considered a secondary member or attachment and no weld quality criteria were used nor were nondestructive test requirements imposed. Splices in longitudinal stiffeners frequently fall into this category.

In November 1973, an inspection revealed a large crack in the south fascia girder of the suspended span of the I-91 bridge over the Quinnipiac River (*6*). The bridge was approximately 9 years old when the crack was discovered. Figure 3 shows that the crack had propagated to the middepth of the web and had extended into the bottom flange at the time it was discovered. Detailed examinations of the fracture surface conducted during the course of the investigation revealed that the fracture began at the unfused butt weld in the longitudinal stiffener splice.

A similar condition has occurred when backing bars were used to make a groove weld between transverse stiffeners and

FIGURE 2 Fatigue crack at web gusset weld termination.

FIGURE 3 Crack originating from groove weld in longitudinal stiffener.

a lateral gusset plate. Lack of fusion usually exists adjacent to the girder web in the transverse groove welds. When the detail is such that the transverse welds intersect the longitudinal welds, a continuous path is created for the crack to enter the girder web. Longitudinal stiffeners on beams or girders of new structures should be fabricated with fully fused welds whose quality is verified by nondestructive examination.

Cracks that have developed in the web at lateral connection plates have generally started at intersecting welds. Horizontal gusset plates used to connect diaphragms and lateral bracing members to a longitudinal girder are often slotted around transverse stiffeners. The Lafayette Street bridge over the Mississippi River at St. Paul, Minnesota, was one of the first known structures to exhibit this type of cracking (*7*). The primary problem was a large defect in the weld attaching the lateral connection plate to the transverse stiffener. Because this weld was perpendicular to the primary stress field and also intersected the vertical welds attaching the stiffener to the web and the longitudinal welds attaching the gusset to the web, a path was provided into the girder web. Detailed studies indicated that the crack originated at a large lack-of-fusion discontinuity in the weld between the gusset plate and the transverse stiffener. Intersecting welds at the corner permitted the transverse crack to penetrate into the girder web.

Development of these cracks indicates that considerable care must be exercised when web gusset plates are used for bracing or lateral systems.

Intersecting welds should be retrofitted and the local area carefully inspected for cracks. A related lack-of-fusion type defect and cracking, which has been among the most severe encountered, occurred at details where a plate component was inserted through an opening cut into a girder web. The resulting detail was usually welded into place with either fillet or groove welds. In either case, large cracks in the girder web resulted at the edge of the flange plate where short vertical weld lengths contained large unfused areas. This feature was illustrated by a large crack in one of the steel box bents supporting the elevated track of Chicago's mass transit Dan Ryan line. Discovered in January 1978, subsequent inspection showed that box adjacent bents were also cracked (*8*). Initial field examination of the fractures indicated that all of the cracks started at the junction of the plate girder flange tip to the box side plate, as shown in Figure 4. All three cracks completely severed the bottom flange of the box girders and the webs.

Extreme care must be exercised when flange, web, or diaphragm plates pass through the web of an intersecting member. Because it is nearly impossible to provide full fusion welds at the penetrating flange tips, installation of open holes are desirable at the ends of the slots to ensure adequate fatigue resistance. Figure 5 shows a retrofitted detail.

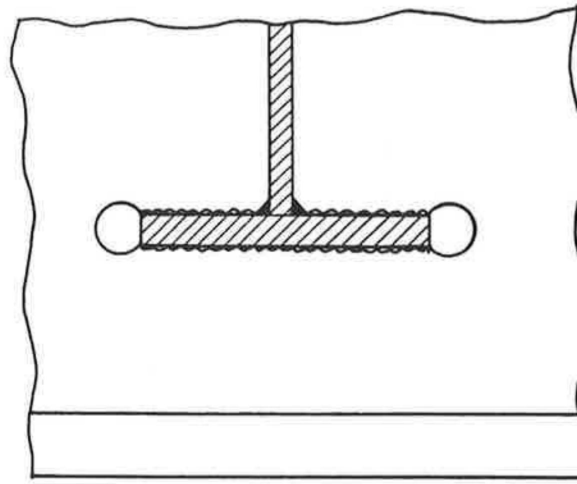

FIGURE 5 Flange plate passing though intersecting web with holes at flange tip.

OUT-OF-PLANE DISTORTION

Several hundred bridges have developed fatigue cracks as a result of out-of-plane distortion in the small, unstiffened segment of a girder web (*1–3*). When distortion-induced cracking develops in a bridge, usually large numbers of cracks form before correction action is taken because the cyclic stresses are usually high. As a result, many cracks form nearly simultaneously in the structural system. Early detection of this condition would permit other potential crack locations to be identified and retrofitted before significant damage develops elsewhere.

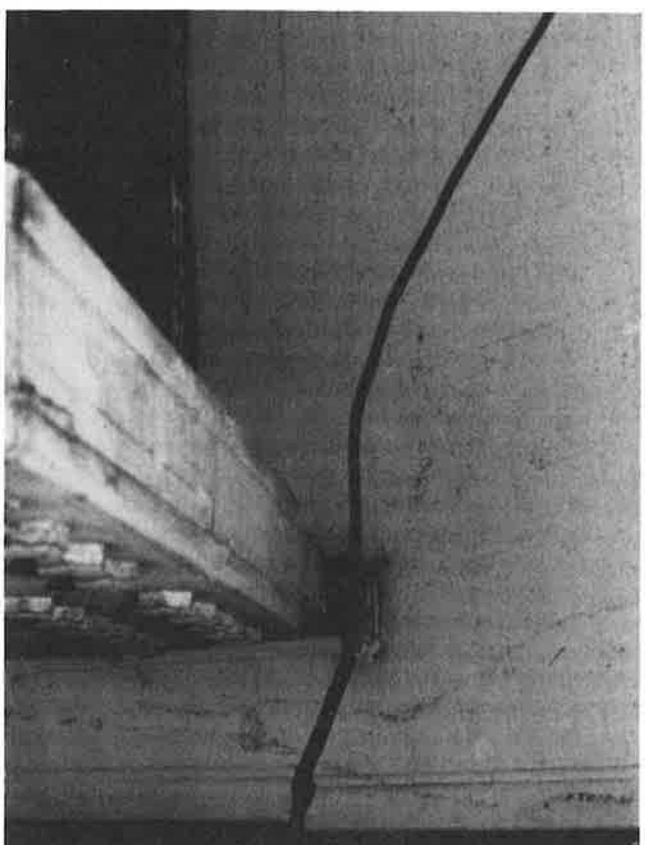

FIGURE 4 Crack in box girder web at intersecting flange tip.

The problem of displacement-induced fatigue cracking has developed in many types of bridges, including truss, suspension, two-girder floorbeam, multibeam, tied-arch, and box girder bridges. Cracks have usually formed in planes parallel to the stresses from loading and have not been detrimental to the performance of the structure providing they were discovered and retrofitted before turning perpendicular to the applied stresses from loads. In some structures, the cracks stopped in low-stress areas and thus have served to relieve local restraint conditions.

Conditions that favor the formation of web gap cracks have most often developed because of the desire to avoid welding transverse connection plates to the tension flange. Figure 6 shows a typical condition that exists in floor beam girder bridges. As the floor beam rotates under traffic loading, the segment of girder web is pulled out-of-plane, producing a large stress gradient in the web gap. Such large cyclic stresses will result in fatigue cracking in a relatively small number of load cycles.

Other examples of web gap fatigue cracking abound. Examples include diaphragm connection plates in multibeam bridges, internal diaphragms in box and tie girder structures, and lateral connection plates that are welded to girder webs but cut around transverse stiffeners.

Current AASHTO specifications require new designs to provide a positive attachment between transverse connection plates for diaphragms and X frames and both girder flanges (*9*). This attachment decreases the web gap distortion to acceptable levels, providing web gaps at the copes are at least 2 in. or four times the web plate thickness, whichever is larger. Structures without positive attachment will eventually have to be retrofitted by providing this corrective measure.

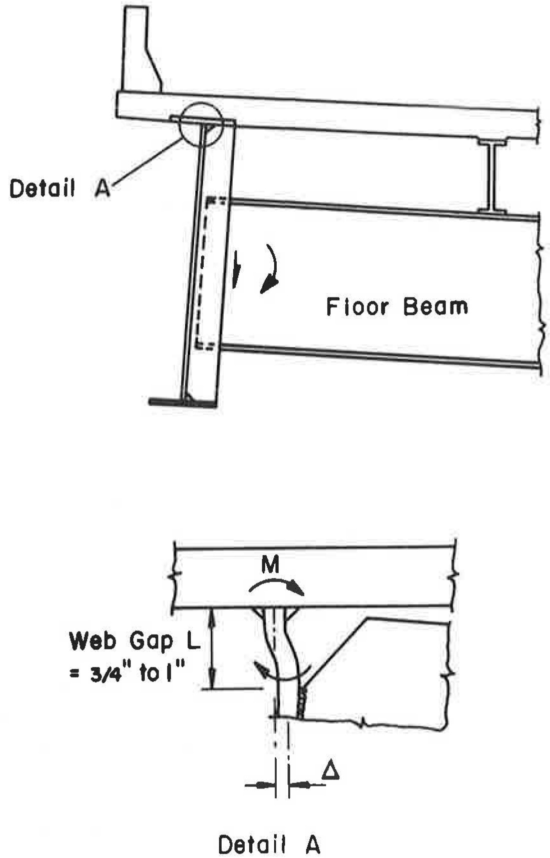

FIGURE 6 Schematic of distortion in web gap at end of transverse connection plate.

RESTRAINT, COPES, AND FLANGE TERMINATIONS

Increasingly, cracks are developing at coped flanges and welded flange terminations. All simple connections provide some degree of end restraint (*1–3*). When rolled sections are coped by flame cutting, the burned edge has tensile residual stresses that approach the yield point. Because one or both flanges are removed, the web plate has a low section modulus compared with the member section. This condition can increase the bending stress in the web by 200 to 300 percent.

A similar condition results when the flanges of welded beams are terminated short of their end connections. This practice is typical in many floor beam girder bridges. Figure 7 shows a crack that developed at the end of a floor beam flange of the Woodrow Wilson bridge over the Potomac River. The fatigue crack at the cope of the floor beam resulted from a reduction of in-plane bending resistance, low-fatigue-strength detail created by termination of the flange plate welded to the web, and end restraint of the shear connection. Cracks can form at the top and bottom flanges because of the residual stresses at the flange-to-web weld termination and because of construction-induced stresses.

Coped flanges are also susceptible to distortion in certain types of applications. For example, at expansion joints it is not unusual to cope the top flange to accommodate the expansion joint. Lateral movement may develop between adjacent spans and produce large out-of-plane web bending stresses at the cope. Coped members need continued observation for cracking.

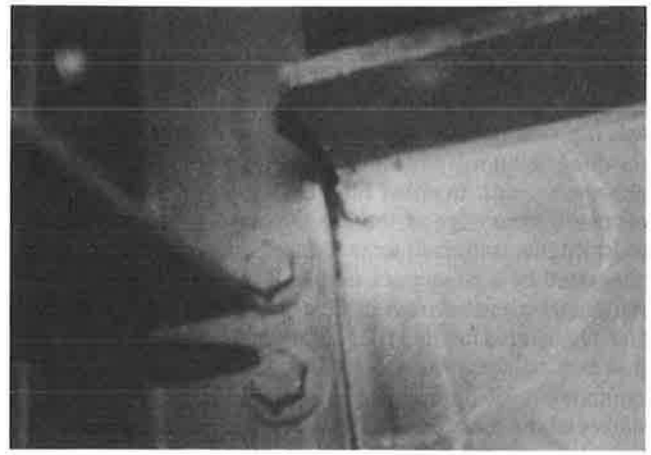

FIGURE 7 Fatigue crack in floorbeam web at end of welded flange.

GUIDELINES FOR INVESTIGATING THE CAUSES OF CRACKING

If a crack is detected in a bridge component, the type of structure, crack location, and characteristics of the crack must be considered before steps can be taken to evaluate the causes of the cracking.

Of primary importance is the significance of the crack for the load-carrying capacity of the bridge. If the crack is moving perpendicularly to the main tension from the applied loads, holes should be drilled at the crack ends so as to blunt the tips and arrest growth. Installation of either a ¾- or ⅞-in. bolt should be accommodated by the hole. Valuable information can be obtained from the region and the holes should be made with a hole saw so that the crack tips are available for subsequent study. After the holes are drilled, the hole surface should be checked by dye penetrant or some equivalent method to ensure that the crack tip has been removed. Usually, the drilling of holes can be considered only a temporary retrofit pending permanent repair. In addition, field personnel should spray-coat the crack surfaces with a clear acrylic lacquer so that the surface features can be preserved.

Removal of Crack Segment

In order to investigate the cause of the crack, part of the cracked segment is usually removed. However, before removal, data need to be acquired on size, location, and orientation of the crack. These data should be gathered by

- Detailed sketches showing crack location, dimensions of the crack, and orientation with regard to the primary stresses in the member, and
- Photographs showing visible crack conditions and location of the crack relative to the detail at which it is formed.

After documentation, a portion of the crack can be removed to permit evaluating the causes of the cracking. Generally, two types of samples can be removed. If material characteristics need to be determined, then a larger portion can be removed. This method will allow for composition and material property determination. Often, only one crack surface needs to be removed, although removal of both surfaces is preferable.

During removal, steps should be taken to ensure that crack surfaces do not come into contact; otherwise, damage to surface features is likely and important information may be destroyed.

In a number of cases, much smaller cracks exist and removal of large pieces is not necessary. Core samples can be used to remove all or a major part of the crack. Figure 8 shows the polished surface of a core sample removed from a groove weld. The core was positioned near the end of the crack to see if fatigue crack growth had occurred. Figure 9 shows a core removed from a cover-plate termination where a partly through crack was found at a weld toe.

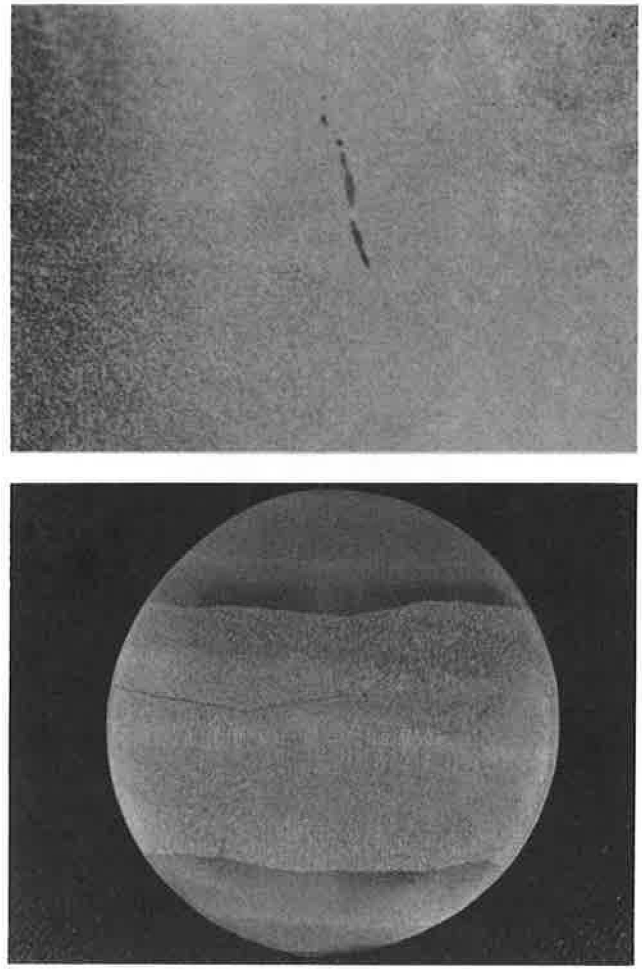

FIGURE 8 Crack in groove weld (*top*) and core sample removed from groove weld (*bottom*).

Data Gathering

In order to assist with the investigation, evaluation, and retrofitting of cracked components and members, the following information should be assembled and documented:

1. Date the crack was first detected;
2. Design stress conditions normal to the crack or detail, as applicable;
3. Average daily truck traffic (ADTT) estimate from the most recent traffic survey and where possible, an estimate of the ADTT for the period of time the bridge has been in service;
4. Estimated frequency and magnitude of any overloads, by permit or other basis;
5. An indication of the approach and deck conditions to permit an assessment of impact;
6. Yield point of the cracked plate or rolled section, as provided by mill reports or tests;
7. Charpy V-notch impact values and test temperatures from mill reports or other sources;
8. Minimum temperature that the structure has experienced during service life;
9. Minimum temperature that the structure experienced during the year preceding discovery of the crack;
10. An indication as to whether the cracked member was struck by a vehicle;
11. If the crack formed at a groove weld, determination of how these welds were inspected when the member was fabricated and all available inspection reports and a radiograph, if possible;
12. If the crack formed at a welded detail, identification of the applicable fatigue design classification according to AASHTO; and
13. Details of weld procedures, qualification tests, and fabrication procedures for the failed component.

Material Tests

Material in the cracked component will usually have documentation available on chemical composition and mechanical properties. However, fracture toughness values will often be unavailable. If material is available, extensive supplemental tests can be conducted. These tests can include standard Charpy V-notch tests. From 12 to 18 specimens should be prepared and tested at several temperature increments. At least three specimens should be tested at each temperature. Where limited amounts of material are available, tests should be carried out at AASHTO specified temperature.

Other tests, such as mechanical and chemical properties, may be useful but are not as essential because mill reports are often available.

For segments of rapid fractures, an estimate of the expected toughness can sometimes be made if the stress, crack size, and geometric conditions are available. This information will permit use of simple fracture mechanics models and allow an estimate of the stress intensity factor.

Metallographic Examination

Determination of weld profile and plate microstructure is often desirable. Investigation of a segment not in the proximity of the crack can reveal profile, heat-affected zone, and weld passes. In addition, information on plate microstructure can be obtained.

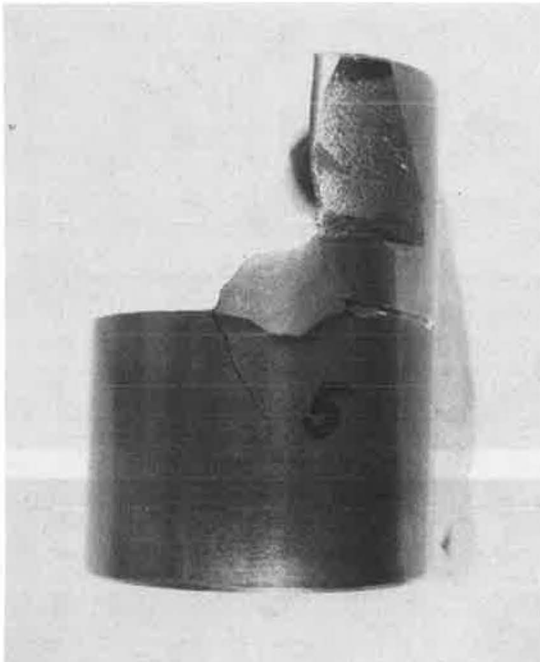

FIGURE 9 Core sample from end of coverplate showing crack at fillet weld toe.

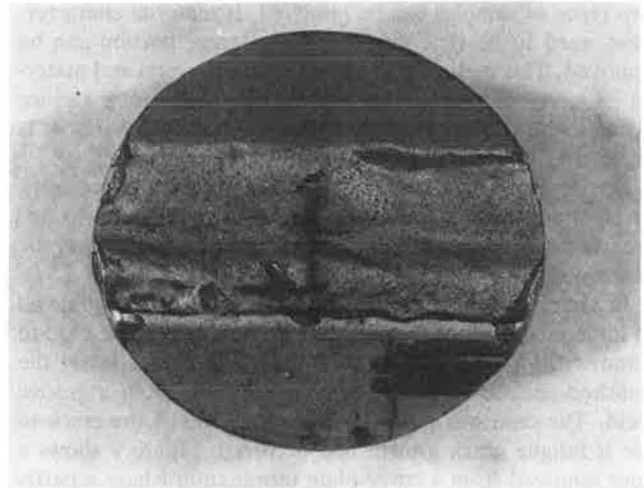

FIGURE 10 Core containing crack from box corner weld.

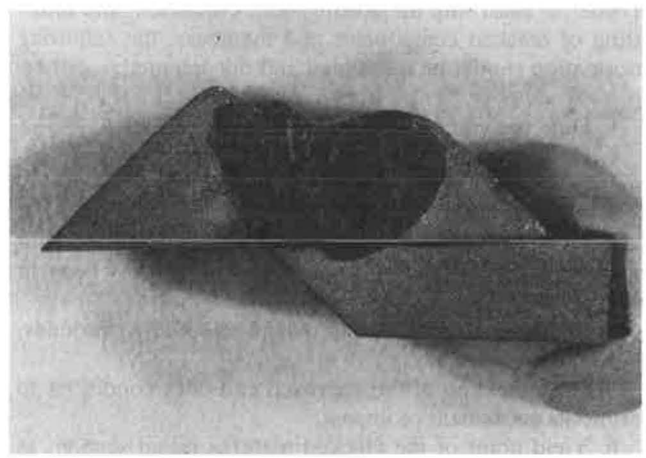

FIGURE 11 Exposed crack surface from box corner weld.

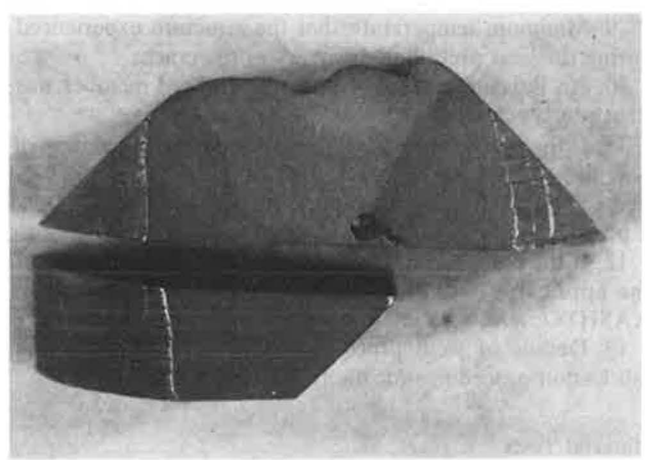

FIGURE 12 Polished and etched plate behind crack surface.

If a crack is associated with a weld and a core is used to obtain a segment of the crack and adjacent material, a detailed examination should be carried out on the surfaces of the core before it is broken apart and cut into segments. Surfaces of the core should be polished and etched to reveal the location of the crack and nature of the surrounding material. Before polishing and etching, the crack should be sealed with an inert wax on the core surface to prevent damage to the crack surface.

Generally, microscopic examinations of core or plate surfaces are made at magnifications between 1 and 100 times normal. Suitable photographs of the exposed features should be obtained. If crack surfaces are not exposed during fracture, cutting the core or sample into segments will be necessary. Surfaces should be exposed only after the piece has been cooled in a liquid nitrogen bath, so the sample can be readily broken apart. Care should be exercised in exposing the crack so that saw cuts do not destroy the crack tips.

Careful attention to the saw cut surfaces, which can be polished and etched, can also reveal weld repair passes, method of fabrication, and other characteristics that may be important in assessing the fracture. Figure 10 shows a crack in a longitudinal box corner weld that has been removed with a hole saw. When the crack surface was exposed, the crack shown in Figure 11 was revealed.

When crack surfaces are exposed, polishing and etching a saw-cut plane parallel to the crack surface will generally be desirable. This plane should be relatively close to the crack surface, normally within ½ to 1 in. Weld repairs, secondary cracking, and other features may be observed. Figure 12 shows the polished and etched surface of a plane parallel to the crack surface shown in Figure 11.

Crack Surface Examination

Where possible, a direct visual examination of the fracture surface is often beneficial in defining the cause of cracking. However, severe corrosion can destroy fatigue and fracture surface markings, as shown in Figure 11. Adequate photographic documentation should be obtained before cutting the surface into segments or cleaning off corrosion.

Normally, fatigue crack growth surfaces will be flat and smooth. Brittle fracture surfaces do not often exhibit the flat, smooth characteristics of fatigue cracks. Chevron-type markings are often apparent on the surfaces of bridge steel that has rapidly fractured. These chevrons point back to the origin of the fracture. Often, shear lips will be apparent on the surfaces of a brittle fracture, examination of which can assist in the estimation of material toughness.

Once the as-received surfaces are examined and photographed, the clear lacquer protective coating and loose corrosion can be removed with an organic solvent such as acetone. This process will often reveal surface features that were obscured. A soft toothbrush can be used to remove the coating and any loose corrosion.

Considerable success has been achieved in crack surface examinations by stripping the areas clean using solvent-softened replica tape. The tape is pressed against the surface and removed after hardening. Oxides and other corrosion will be stripped away. If this procedure is repeated, most of the debris can be eventually cleaned from the crack surface. Final replicas can be examined with a transmission electron microscope because the replica will carry a reverse impression of the crack surface.

Areas of fatigue crack growth may exhibit striations on the replica surface (*8*). These marks are a series of lines or bands that fatigue crack growth exhibits as the crack advances. Other modes can also be verified as cleavage facets may be apparent. Obviously, use of either a scanning or transmission electron microscope requires experienced personnel. These instruments are used to provide a microscopic examination of selected areas of the crack surface.

On the basis of visual observations of the crack surface, establishing whether or not there is evidence of a large initial flaw or crack may be possible. Of particular focus should be evidence of fatigue cracking. Such observations can be used to decide if the cracking can be understood in terms of final flaw size, stress, expected toughness, and so forth. Estimates of stress range and cumulative cycles can be used to decide whether or not the observed crack extension is consistent with flaw size and crack growth kinetic data.

CONCLUSION

Once fatigue cracks are found, investigating the cause of the failure and retrofitting the structure is important. A majority of cracks in welded steel bridges may be a result of low-strength details, large initial defects because of geometric conditions and fabrication, distortion, or restraint problems. Investigation may reveal a consistent and logical answer that can then be used to assess damage in other locations and to develop an economical and effective fix. If an understanding is not achieved, consideration should be given to seeking expert help.

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Method for Predicting the Disadvantaged Business Enterprise Capacity of the Texas Highway Construction Industry

DONN E. HANCHER, ZANE A. GOFF, AND DOCK BURKE

A method for predicting the annual work capacity of disadvantaged business enterprises (DBE), both individually and collectively, is presented that considers variables used to predict failure of small businesses.

The Surface Transportation Assistance Act (STAA) of 1982 specified that at least 10 percent of all federal funds authorized to be appropriated under this Act shall be expended with small business concerns owned and controlled by socially and economically disadvantaged individuals as defined by the Small Business Administration. These small business firms are more commonly referred to as disadvantaged business enterprises (DBEs) and the program implemented by FHWA to fulfill STAA requirements has become known as the DBE program.

Since passage of STAA, each state has established separate DBE programs and added staff to monitor, administer, and evaluate DBE requirements. Programs have been the center of many political debates and controversies, with great pressures placed on highway departments to meet their goals while trying to prevent the creation of illegal firms, or "fronts," which are really not independent companies.

In 1987, STAA was amended to allow the 10 percent goal to be met by an accumulation of the total volume of work performed by DBE and women business enterprises (WBES). This change resulted in many highway construction firms seeking WBE certification. However, many were not approved because their owners were not deemed qualified. As a result, controversy was added to the administrative chores of state agencies. In addition, DBE contractors complained about the fairness of allowing WBEs to be included in their 10 percent goals, feeling that WBEs should have separate goals.

Each state transportation agency can set goals for the DBE program and does not have to reach the 10 percent goal if a lower goal can be justified. However, few states have been able to obtain approval for lower goals. The major difficulty for most agencies is the ability to accurately assess the true capability of minority firms in their state to perform highway construction work. A set of analytical procedures was developed that will assist the Texas State Department of Highways and Public Transportation (TSDHPT) in assessing DBE work capacity.

D. E. Hancher, Department of Civil Engineering, Z. A. Goff and D. Burke, Texas Transportation Institute; Texas A&M University, College Station, Tex. 77843.

DEFINITION OF CAPACITY

The term "capacity" is used to describe the maximum amount of work per year, measured in dollars, that a group can perform at an acceptable level of quality without diminishing the future viability. This group can be an individual business firm or a collection of business firms such as found inside a governmental unit (i.e., a county, state, or highway district). Hence, the term "statewide capacity" refers to the collective capacity of designated individual firms within a state, whereas firm capacity pertains to a single business entity.

LITERATURE REVIEW

No empirically valid solution or procedure exists for predicting the capacity of a construction firm. However, several rule-of-thumb procedures do exist. For example, the surety industry sets bonding capacity at 10 times working capital (i.e., current assets less current liabilities) or 5 times net worth. Other factors, such as company age, size, project experience, financial position, etc., affect bonding capacity as well. In another example, bidding capacity of prequalified general contractors is defined by TSDHPT as 20 times working capital adjusted for various factors. In both examples, neither empirical tests nor financial theory entered into that capacity-setting process. These limits were set by years of experience and an engineering judgment for what is correct. However, a logical and empirically verifiable thesis can be presented that construction revenue is positively correlated, in a statistical sense, with financial resources. An additional thesis is that owners, insurers, and creditors of construction projects desire that contractors have the necessary economic viability to complete these projects. In other words, construction firms should be as far from the prospects of bankruptcy as feasible in an economically cyclical industry like construction.

Empirical financial research has demonstrated that financial ratios are statistically significant and reliable predictors of financial distress from 1 to 5 years before business failure (*1–8*). This relationship holds for large publicly capitalized and traded concerns to small businesses regardless of the type of venture.

Table 1 presents the ratios investigated along with the name of the researcher. A strong amount of agreement exists among these researchers. Altman (*1*) and Ohlson (*2*) agree that liquidity, leverage, and profitability ratios are important in

TABLE 1 IMPORTANT FINANCIAL RATIOS FOR PREDICTING BANKRUPTCY

	Researcher					
	Altman	Beaver	Blum	Dambolena	Ohlson	Vinso
Ratios	1968	1966	1975	1980	1980	1979
Liquidity ratios:						
Working capital/Total assets (1,2)	X		X		X	
Current assets/Current liabilities (1)					X	
Leverage ratios:						
Total assets/Total liabilities (1,2)	X	X			X	
Market value of equity/Book value of debt (1)	X					
Retained earnings / Total assets (1)	X					
Profitability :						
Cash flow / Total liabilities (1,2)		X	X		X	
Earnings /Total assets (1,2,3)	X	X			X	
Net loss last 2 years					X	
Net income /Equity (1)			X			
Other measures :						
High variability of income year-to-year						X
High variability of financial ratios	X			X		
Initial pool of funds (1,4)						X
Size (1)					X	

Notes :

(1) The lower the value the more likely the firm will become bankrupt

(2) Related measurements or concepts

(3) Earnings in this case pertains to either earnings before interest and taxes or net income

(4) The amount of long-term debt and equity it took to start the business

predicting bankruptcy. Blum (*3*) concurs with respect to liquidity and profitability, whereas Beaver (*4*) confirms the importance of leverage and profitability ratios. Dambolena (*5*) found evidence, as did Altman, that a high variability of financial ratios is a good predictor of bankruptcy. Finally, Vinso (*6*) found that high variability of income from year to year and the initial pool of funds (i.e., start-up capital) is important for predicting bankruptcy as well. If the Vinso study can be considered valid for DBEs in the highway construction industry, then this finding has ominous implications for DBEs because they are engaged in a cyclical industry (i.e., income varying from year to year depending on the economy) and many lack sufficient start-up capital. Overall, an inference can be made that unsuccessful firms are low in liquidity, highly leveraged, and unable to generate sufficient operating funds. The lack of working capital, net worth (i.e., higher net worth precludes higher leverage), and cash flow are important in the viability of a DBE.

The Edmister study (*7*) is particularly relevant to DBEs because this study concentrated on small business firms and found a relationship between undercapitalization and revenue. A net worth/revenue ratio of less than 7 percent was one indicator of future failure. The implication is that equity capital is insufficient to support revenue whenever this ratio reaches a level below 7 percent or its reciprocal becomes greater than 14 to 1.

BASIC FORECASTING MODEL

Statewide highway capacity (in dollars) of certified DBEs can be divided into two components:

1. Amount of revenue produced annually by DBEs, and
2. Proportion of DBE annual revenue derived from state highway work.

Therefore, the basic capacity forecasting model can be expressed as

$$S_t = 0.1584 \sum_i R_{ti} \qquad i = 1, 2, \ldots, n \tag{1}$$

where S_t equals the expected maximum amount in dollars contracted to DBEs in year t (i.e., statewide DBE highway capacity), and R_{ti} equals the annual dollar capacity of the ith DBE in year t.

The factor 0.1584 represents the average proportion of the revenue derived from Texas highway subcontracts to contracts for DBEs. This factor is the product of two proportions:

1. Proportion of certified DBEs winning state highway subcontracts (i.e., 24 percent); and
2. Proportion (i.e., 66 percent) of a DBE's revenue earned from state highway work, given that the DBE has won an award.

These two proportions were obtained by examining 1986 and 1987 TSDHPT DBE and WBE construction reports along with DBE financial statements and represent the average of the 2 years.

METHODS OF ESTIMATING CAPACITY R_{ti}

According to the financial literature, a viable firm (i.e., one that will be solvent) has adequate capital (*1,6,7*), can generate suitable profits or cash flow relative to debt (*1,3,8*), and has sufficient liquidity (*2,7,8*). On the basis of these aforementioned characteristics, five different methods can be used to estimate annual capacity:

- Net worth method,
- Revenue method,
- Regression method,
- Minimax method, and
- Working capital method.

Net Worth Method

In this method, capacity is a multiple of net worth, calculated as follows:

$$R_{ti} = 14.2NW_{(t-1)i} \tag{2}$$

where $NW_{(t-1)i}$ equals the net worth in year $t - 1$ of the ith DBE.

The rationale behind using net worth is that net worth is a permanent source of funding, provides a rough indication of the firm's profitability, and is an approximation of the amount of protection afforded creditors. In turn, the viability of a contractor depends on the ability to receive credit.

The multiplier 14.2 is a threshold number equal to the ratio of sales to net worth found by Edmister (7). In that study, small businesses that have ratios higher than this amount do not have adequate capital to support such sales.

Revenue Method

In this method, capacity is defined as

$$R_{ti} = 2R\max_{(t-1,\ldots)i} \tag{3}$$

where $R\max_{(t-1,\ldots)i}$ equals the highest revenue earned before year t for the ith DBE.

The multiplier 2 is the maximum that a contractor could reasonably expect to manage (9). The assumption behind this method is that past work-load experience, as expressed by revenue, influences future work-load performance.

Regression Method

In this method, capacity was predicted by multiple regression techniques using Texas DBEs as the data base. The type of data under consideration evaluated the DBE's financial resources (as derived from financial statements), bonding capacity, human resources (such as work and business experience), and other factors like geographic location, sex and race of owner, and work specialty. This information was obtained from the TSDHPT using DBE files from questionnaires sent to DBEs in Texas.

The regression model was based on lagged variables (i.e., independent variables for year $t - 1$ given that the response variable is for year t) as in the previous three models. Therefore, only DBEs that had reported complete financial statements for two consecutive years could be considered. Furthermore, other factors reduced the pool of DBEs that qualified for the multiple regression analysis. Firms that had a negative net worth (i.e., insolvent firms) or those that did not report financial statements reflecting adequate accounting procedures were omitted as well. From approximately 500 certified, highway-related DBEs (engineering firms and trucking firms not engaged in hauling were not considered), the number of firms meeting the criteria was reduced to 90.

Independent variables for this multiple regression model were selected using the stepwise computer selection procedure. This procedure starts with the best one-variable equation, but before adding subsequent variables, the statistics are examined for insignificance, and, if found, the variable is eliminated and another variable chosen. The procedure terminates for specified significance levels when variables can neither be added nor deleted (10). This procedure was modified slightly in that goodness-of-fit criteria were considered as well. This procedure led to a model in which one parameter was included that did not have the same level of statistical significance as the others selected, but did achieve a better fit.

From the collection of variables presented in Table 2, four statistically significant variables were selected that had the most explanatory power. These variables were total assets (TA), long-term debt (LTD), total assets times bonding capacity in dollars (TA * BOND), and working capital (WC).

Tables 3 and 4 present the various statistical tests of the regression model. Table 3 indicates that the regression model has adequate goodness-of-fit as shown by the adjusted coefficient of determination (adjusted R^2) of 0.698. Second, the analysis of variance test reveals that a statistically significant model ($\alpha \leq 0.0001$) was developed. In Table 4, the test of the individual parameter statistics (t values) reveals that the parameters TA, TA * BOND, and LTD are highly significant ($\alpha < 0.05$), whereas WC does not have this level of significance (i.e., the chances that the coefficient of WC is zero are almost 13 in 100). Furthermore, the sign of the coefficients indicates that total assets, bonding capacity, and working capital increase future revenue, whereas long-term debt has a tendency to reduce future revenue. Hence, large, unleveraged, liquid DBEs that are able to secure bonding should produce the greatest revenue.

Because capacity is the maximum annual revenue that a firm can manage consistent with its financial and human resources, the upper 99 percent confidence intervals for the

TABLE 2 VARIABLES CONSIDERED FOR SELECTION

Code	Description
NORG	Proprietorship (1) , partnership (2) ,corporation (3)
NBUSYRS	Number of years in business
BOND	Bonding capacity in $ (000's)
HIREV	Highest revenue in $ (000's) earned in one year
CA	Current assets in $ (000's)
TA**	Total assets in $(000's)
CL	Current assets in $(000's)
LTD **	Long-term debt in $(000's)
TLIA	Total liabilities in $(000's)
NW	Net worth in $(000's)
WC **	Working capital in $(000's)
LBOND	Whether or not firm is bondable (1=yes,0=no)
TCAP	NW + LTD
TA *BOND **	Total assets times bonding capacity
TREND	Trend in revenue (1=increase,0=same,-1=decrease)
LOC	Geographic location of business (code used)
DENS	Population density of location
ETHNIC	Ethnicity/gender of owner (1=Black male ,2= Hispanic male, etc.)

TABLE 3 ANOVA FOR REGRESSION MODEL

Source:	DF:	Sum Squares:	Mean Square:	F-test:
REGRESSION	4	240439428.7	60109857.2	52.5
RESIDUAL	85	97304276.2	1144756.2	p=.0001
TOTAL	89	337743704.9		
R:	R-squared:		Adj. R- squared:	
.844	.712		.698	

TABLE 4 BETA COEFFICIENT TABLE

Parameter:	Value :	Std. Err.	t-value :	Probability:
INTERCEPT	198.374			
TA	2.360	.263	8.97	.0001
LTD	- 2.392	.633	3.78	.0003
TA*BOND	1.723 E-4	4.920E-5	3.50	.0007
WC	.286	.186	1.54	.1270

four parameters are used as the coefficients for this model. Upper 100 (1 − α) percent confidence estimates $B_{(1-\alpha)j}$, are constructed from the following equation (*10*):

$$B_{(1-\alpha)j} = b_j + t_{(\alpha,q)}s(c_{jj})^{0.5} \quad (4)$$

where

b_j = *j*th parameter coefficient estimate,
s = estimate of standard error,
$t_{(\alpha,q)}$ = *t*-value for α and *q* degrees of freedom, and
c_{jj} = *j*th row and column element of the variance-covariance inverse matrix **C**.

From Table 5, DBE capacity is estimated using the following regression:

$$R_{ti} = 198.374 + 3.053\ (\mathrm{TA}) - 0.724\ (\mathrm{LTD}) + 0.000302\ (\mathrm{TA} * \mathrm{BOND}) + 0.775\ (\mathrm{WC}) \quad (5)$$

All quantities for these variables are expressed in thousands.

Minimax Method

The minimax method selects the minimum estimate of the previous three capacity (maximum revenue) methods applied to an individual DBE. The method can be expressed as

$$R_{ti} = \min\{R[1]_{ti}, R[2]_{ti}, R[3]_{ti}\} \quad (6)$$

The numbers in square brackets signify net worth, revenue, and regression methods, respectively. The working capital method was omitted because of possible accounting classification problems concerning current liabilities resulting in negative working capital, or some valid relationships between a DBE and prime contractor permitting assistance in bill-paying ability. Also, the regression method puts some weight on working capital.

The underlying concept of the minimax method is one of conservatism. Recall that the lack of net worth has been significantly associated with bankruptcy. Hence, some measure of an upper bound should be instituted.

TABLE 5 CONFIDENCE INTERVALS AND PARTIAL *F* TABLE

Parameter:	99% Lower:	99% Upper:	Partial F:
INTERCEPT			
TA	1.666	3.053	80.454
LTD	-4.059	-.724	14.285
TA*BOND	4.270E-5	3.020E-4	12.269
WC	-.203	.775	2.375

Working Capital Method

Working capital is defined as the excess of current assets over current liabilities. The ratio of revenues to working capital is used as a measure of adequate liquidity by Robert Morris Associates (*11*) and Dunn & Bradstreet credit rating agencies (*12*). In addition, lack of adequate working capital was found to be a significant factor leading to bankruptcy (*1*,*3*).

This method calculates capacity as

$$R_{ti} = 20\mathrm{WC}_{(t-1)i} \quad (7)$$

where $\mathrm{WC}_{(t-1)i}$ equals the working capital in year $t - 1$ for the *i*th DBE.

The multiplier 20 is essentially that used by the TSDHPT for evaluating the bidding capacity of prequalified prime contractors. Although there is nothing sacrosanct about the value 20, this figure does place DBEs on an equal evaluation basis with Texas prime contractors. In addition, this value is more than twice the median value for typical highway contractors as determined by Dunn & Bradstreet (*12*).

EVALUATION OF THE CAPACITY METHODS

Table 6 presents a comparison of the multiplier values used for the working capital, net worth, and revenue methods and the values from five highly capitalized general contractors—Blount, Morrison-Knudsen, Perini, Fluor, and the Slattery Group. Data were obtained from the Value Line Investment Survey (*13*) and represented the maximum values for each firm over the last 5 years. In each case, the DBE multiplier exceeds the median value for these large, publicly traded general contractors.

Both for the net worth and revenue methods, the DBE multipliers exceed the maximum values of these contractors. This is what one would desire for a number that purports to be a reasonable maximum. The working capital method does not achieve this, which suggests that this method may have too much variability to be a good predictor of capacity. Each multiplier for these methods exceeds the median value of several highly successful construction management firms in the heavy construction industry.

However, some caveat regarding the use of a sample of five largely capitalized general contractors for valid compar-

TABLE 6 COMPARISON OF MULTIPLIER VALUES

Method	DBE Multiplier	Large Capitalized General Contractors (n=5) Minimum	Median	Maximum
Working Capital	20.0	5.4	16.9	130.0
Net Worth	14.2	3.0	5.7	8.7
Revenue	2.0	1.0	1.3	1.5

ative purposes is in order. First, the comparison of multipliers in Table 6 is used only for estimating purposes. Second, the five large general contractors used may not be representative of typical highway construction firms. Last, if the study team would have been able to obtain the multipliers for Texas prime contractors, these data would have been preferred. However, TSDHPT does not require all of the financial information necessary for this type of analysis from their prime contractors. Notwithstanding these drawbacks, the DBE multipliers are adequate maximums for the following reasons:

1. The multiplier of 14.2 is the threshold value cited in an empirically valid study (7);
2. The revenue multiplier of 2.0 implies workload growth of 100 percent per year, an enviable rate for any firm; and
3. As confirmed by Table 6, each of these multipliers exceeds the maximums of large, well-organized, and established construction firms.

Table 7 presents advantages and disadvantages of each of the methods. Each of the three multiplier methods has a drawback by basing capacity on a single factor. Net worth and revenue methods have the advantage of being consistent when compared with the large capitalized contractor values presented in Table 6. The working capital method does not achieve this consistency, but is widely used in the bonding industry. With respect to DBEs, this method may present accounting classification problems. The regression method overcomes the disadvantages of the other three in that several factors are considered. However, the regression model does not entirely explain DBE revenue-producing process. The minimax method has the advantage of using all of the methods previously discussed (except working capital), provides a conservative estimate, and seeks to place an upper bound on the estimates that is consistent with the financial research on bankruptcy. All methods have merit; however, the minimax method is judged to be the most reasonable method to use.

TABLE 7 EVALUATION OF CAPACITY ESTIMATION METHODS

Method	Advantages	Disadvantages
Working Capital	• emphasizes importance of liquidity • wide acceptance in bonding industry	• values too volatile for good predictor • account problems • considers only one factor • can not measure explanatory power of model
Net Worth	• emphasizes importance of retained earnings & start up capital • fairly consistent predictor* • valid predictor of business failure	• considers only one factor • can not measure explanatory power of model
Revenue	• most consistent predictor * • considers past experience	• considers only one factor • can not measure explanatory power
Regression	• several factors considered • procedure can measure explanatory power of model	• explanatory power of model adequate, but not exceptional
Minimax	• considers 3 methods • conservative estimates • provides upper bounds	

* as compared to Table 6 Comparison of Multiplier Values

APPLICATION: ESTIMATION OF STATEWIDE CAPACITY

Texas DBE capacity is estimated by Equation 1, in which the values of R_{ti} are computed using the minimax method as expressed by Equation 6. Table 8 presents example calculations for Texas DBE capacity.

First, the three maximum methods designated $R[1]$ to $R[3]$ on Table 8 are applied to each firm. Second, for each firm the minimum value designated $R[4]$ is calculated. Third, the statewide capacity is found by summing the $R[4]$ column and multiplying by a factor of 0.1584 as indicated by Equation 1. For firms that do not provide essential pieces of financial information required by this procedure, the average DBE capacity (i.e., $0.1584\Sigma R[4]/N$) is used. In Table 9, the value of N is 140.

Table 9 presents 1988 DBE capacity estimates for the western, central, and northern regions of Texas, as well as those derived from DBEs headquartered out-of-state. For the state, work totaling an estimated $81 million can be performed by DBEs working in Texas. More than 53 percent is expected to come from the central portion of the state, 26.5 percent from the northern districts, and approximately 9 percent from the western region of Texas. According to this model, Texas would obtain 11 percent of its expected capacity from out-of-state sources.

The capacity figures for each region were calculated in the following manner:

$$\text{Total region capacity} = 0.1584\Sigma_i R[4]_i + (\text{Average firm capacity} \times K) \quad (8)$$

where i equals $1, \ldots, m$, the number of firms in the region that have the essential pieces of financial information, and K equals the number of certified DBE firms in the region not having the required financial information.

The average firm capacity is $0.1584\ \Sigma\ R[4]/140$.

The estimate of $81 million presented in Table 9 compares favorably with the $88 million that was actually achieved by Texas DBEs for the 1988 fiscal year.

MODEL LIMITATIONS

Limitations of the model are the following:

1. Possible lack of generalizability of the model to other transportation departments. Data presented are based on

TABLE 8 EXAMPLE CALCULATIONS FOR DBE CAPACITY

All values in $(000's)

	Required Financial Information							Capacity Estimation Method			
Firm	Revenue	TA	LTD	BOND	TA*BOND	WC	NW	R[1]	R[2]	R[3]	R[4]
1	46.0	42.0	0.0	0.0	0.0	2.9	33.4	474.3	**92.0**	328.8	**92.0**
2	1400.0	406.0	180.0	200.0	81200.0	145.3	204.0	2898.6	2800.0	**1444.7**	**1444.7**
.											
.											
.											
N	6200.0	3038.2	1549.4	0.0	0.0	964.6	697.4	9903.1	12400.0	**9099.8**	**9099.8**

Notes:

R[1] = 14.2 NW

R[2] = 2 REV

R[3] = 198.374 +3.053TA - 0.724LTD +0.000302TA*BOND +0.775WC

Total capacity of Texas DBE's= .1584Σ (R[4])

TABLE 9 ESTIMATION OF DBE CAPACITY FOR 1988

Region	**Estimated Capacity ($ thousands)**	
Out of State	8,900	11.0%
West Texas Districts: 3-8,23-25	7,600	9.4%
Central Texas Districts: 12-16,21	43,000	53.1%
North Texas Districts: 1,2,9-11,17-20	21,500	26.5%
Total	**81,000**	**100.0%**

financial information of acceptable quality from Texas DBEs. Few firms provided audited financial statements.

2. Stability of the multiplier 0.1584 in Equation 1. Recall that this multiplier is derived from (a) the proportion of certified Texas DBEs receiving a contract or subcontract, and (b) the proportion of revenue applicable to DBE highway work, which represents a 2-year average.

3. Adequacy of the regression model. Even though the adjusted coefficient of determination was 0.698, approximately 30 percent of the variation in the response variable is unexplained by the model. The literature reveals that profitability was a good predictor of bankruptcy. This factor may be the needed variable. However, TSDHPT does not require this information, which therefore was unobtainable.

CONCLUSION

The proposed minimax method for predicting firm capacity used in conjunction with Equation 1 appears to be a logical analytical procedure for estimating statewide capacity of DBEs. This methodology includes carefulness to not violate the empirical findings regarding the prediction of bankruptcy or firm failure.

However, the method presented is exploratory in nature and more data are needed to further adjust and validate the prediction model. An analysis of complete financial data for DBEs that have terminated their operations would help in this validation process. The problems are

1. Obtaining financial data of such distressed firms. The Small Business Administration, state highway departments, banks, and Dunn & Bradstreet would be likely sources. However, because of the sensitive and personal nature of this information, such organizations may be reluctant to release these data.

2. High likelihood that such DBE firms did not keep reliable financial statements that conform to acceptable quality standards. Lack of a proper record-keeping system contributes to a firm's downfall. The typical DBE in construction may be familiar with the construction aspects of a project, but not with the management aspects, such as budgeting, cost control, scheduling, and financial expertise.

In addition, regression parameters could be derived for nonminority subcontractors for comparative purposes. The problem is the cooperation needed from nonminority subcontractors. Experience indicates that the DBE program is such a controversial subject that the cooperation of nonminority subcontractors would be difficult to obtain. Because a substantial number of DBE and WBE firms have 5 years or less experience, a true comparison would require nonminority subcontractors to furnish financial information reflecting their status as contractors at the same level of experience. However, nonminority subcontractors may not have retained this type of information.

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Disadvantaged Business Enterprise Program: A State Perspective

LISA WORMINGTON

Arizona's Disadvantaged Business Enterprise (DBE) program is discussed from the experience and perspective of the Arizona Department of Transportation (ADOT). ADOT establishes goals for DBE participation on highway design and construction projects. Attainment of a goal of 10 percent for design jobs is part of a scoring schedule. Goals ranging from 0 to 15 percent for construction projects are set according to project size and location, DBE availability, and type of work involved. The DBE program has positive and negative aspects. On the positive side, the program has opened doors for small businesses to participate in the construction industry as evidenced by such firms' also working on projects without goals. The program has also enabled minorities and women to move from being employees to owning small businesses. Also, the increased competition among DBEs and other subcontractors appears to have resulted in lower construction costs. On the negative side, setting goals on construction projects can be difficult because of the nature of the work. The bidding process has become problematical. Further, the program has resulted in increased responsibilities in the field and has involved Arizona in the prime and subcontractor relationship. Finally, addressing problems arising when DBEs are unable or unwilling to perform is time consuming and controversial.

The Arizona Department of Transportation (ADOT) is charged with designing, constructing, and maintaining all federal and state highways and roads in Arizona. Currently, the majority of funds to perform these functions are allocated from the Arizona Highway User Revenue Fund. Approximately 17 to 18 percent of total funds expended is federal aid.

The 1987 Surface Transportation and Uniform Relocation Assistance Act requires recipients, in this case ADOT, to expend no less than 10 percent of the federal funds on firms owned and controlled by socially and economically disadvantaged individuals. ADOT has consistently exceeded this goal (see Table 1). In 1985, disadvantaged business enterprises (DBEs) received $30 million, or 18 percent of the federal dollars. In 1986, the figure increased to $36 million (14 percent), whereas in 1987, Arizona experienced a decrease in federal funds and the DBE share was $26.5 million (16.5 percent). In 1988, federal funding declined further and $22 million was expended with DBEs, or approximately 11.6 percent of the total federal dollars. Only federally assisted contracts have DBE requirements—no state law mandating DBE requirements on state-funded projects exists.

ADOT is proud of the success of the DBE program. The largest factor for this success is the strong support the program receives from the director, the state engineer, and the state transportation board. Other significant factors include support and guidance from FHWA and the efforts of field personnel.

The DBE program includes engineering, consulting, and construction contracts. Inclusion of engineering projects is a recent development. In 1986, engineering consultant services included points for use of DBEs as part of the overall scoring system. DBE participation counted for five points in a go or no-go system. However, the department found that this policy was not as effective as anticipated. DBE participation was low and most of the work was going to firms specializing in geotechnical work or surveying. A system was then developed in which prime consultants received an additional three bonus points for using a DBE firm specializing in design. Subsequently, the maximum number of points a prime consultant could receive is 103. Since this development, contract provisions were revised to include a 10 percent DBE participation requirement. Another development is that the affirmative action office now reviews, scores, and approves all statements of interest and technical proposals for DBE compliance. Since implementing these changes, a significant increase in DBE participation occurred, from 4.5 percent in 1986 to 6.0 percent in 1988. Also many smaller DBE design firms are being included in the prime consultants' teams. ADOT's goal in 1989 is 10 percent DBE participation in designing projects.

As with most states, the construction aspect of highway projects is where the DBE goal has the most impact. ADOT annually establishes a 10 percent DBE goal, which has always been exceeded. DBE construction goals are established on a project-by-project basis. Factors taken into consideration include type of work involved, availability of DBEs to perform the work, location of the project, duration of the project, and size of the project. DBE goals on ADOT projects range from 0 to 15 percent. The average goal is 7.8 percent and the median goal is 10 percent.

In the past, bidders were required to indicate with their bids which DBEs were being used to meet the goals. However, this requirement was changed approximately 2 years ago. The apparent low bidder is allowed five working days to submit this information. Three reasons brought about this change. First, contractors were simply not completing the affidavit properly, resulting in approximately one job per month being awarded to the second-low bidder. Second, bidders were alleging the DBEs were submitting their bids too late to be considered. This situation was confirmed. Many DBEs believed that the later bids are submitted, the lesser the chances were that their prices would be bid-shopped. Third, the extra time would permit contractors to better research a DBE's bonding and insurance capabilities and, subsequently, result in fewer requests to substitute DBEs for these reasons. In the

Arizona Department of Transportation, Affirmative Action Office, 206 South 17th Avenue, Room 154A, Phoenix, Ariz. 85007.

TABLE 1 SUMMARY OF ARIZONA DEPARTMENT OF TRANSPORTATION DBE GOAL ATTAINMENT, 1985–1989

	FY 1985 DOLLARS	FY 1985 PERCENT	FY 1986 DOLLARS	FY 1986 PERCENT	FY 1987 DOLLARS	FY 1987 PERCENT	FY 1988 DOLLARS	FY 1988 PERCENT	FY 1989 DOLLARS	FY 1989 PERCENT
Total Prime Contracts										
Number	132		94		91		116		76	
Total Dollars	$170,583,171.00		$257,825,216.00		$160,387,344.00		$180,919,375.00		$108,193,940.00	
DBE Prime Contracts	$6,254,110.00	3.67%	$400,968.00	0.16%	$1,304,392.00	6.07%	$527,967.00	3.16%	$629,264.00	6.19%
DBE Subcontracts Awarded	$21,850,955.00	12.81%	$27,612,479.00	10.71%	$17,106,194.00	10.67%	$14,984,476.00	8.28%	$7,851,905.00	7.26%
DBE Subcontract Commitments	$19,070,985.00	11.18%	$28,019,854.00	10.87%	$21,481,375.00	13.39%	$16,728,177.00	9.25%	$10,168,895.00	9.40%
WBE Prime Contracts	$2,175,082.00	1.28%	$810,917.00	0.31%	$1,098,140.00	0.68%	$0.00	0.00%	$375,157.00	0.35%
WBE Subcontracts Awarded	$3,341,203.00	1.96%	$7,550,822.00	2.93%	$2,659,872.00	1.66%	$3,445,218.00	1.90%	$2,188,358.00	2.02%
WBE Subcontract Commitments	$3,080,476.00	1.81%	$6,934,245.00	2.69%	$2,620,918.00	1.63%	$3,702,931.00	2.05%	$2,956,241.00	2.73%
DBE/WBE PARTICIPATION BY AWARD	$33,621,350.00	19.71%	$36,375,186.00	14.11%	$22,168,598.00	13.82%	$18,957,661.00	10.48%	$11,044,684.00	10.21%
DBE/WBE PARTICIPATION BY COMMITMENT	$30,580,653.00	17.93%	$36,165,984.00	14.03%	$26,504,825.00	16.53%	$20,959,075.00	11.58%	$14,129,557.00	13.06%
DBE Awarded Contracts by Race/Ethnic/Sex										
Blacks	$1,823,693.00	5.42%	$2,370,560.00	6.52%	$1,294,713.00	5.84%	$1,572,467.00	8.29%	$547,419.00	4.96%
Hispanics	$16,886,295.00	50.22%	$18,493,007.00	50.84%	$11,900,222.00	53.68%	$10,643,805.00	56.15%	$5,607,446.00	50.77%
Native Americans	$3,484,459.00	10.36%	$6,334,014.00	17.41%	$4,238,296.00	19.12%	$2,475,975.00	13.06%	$1,884,247.00	17.06%
Asian Indians	$5,910,618.00	17.58%	$509,850.00	1.40%	$392,099.00	1.77%	$62,689.00	0.33%	$0.00	0.00%
Asian Pacific Islanders	$0.00	0.00%	$246,600.00	0.68%	$538,121.00	2.43%	$757,507.00	4.00%	$442,057.00	4.00%
Other	$0.00	0.00%	$59,416.00	0.16%	$47,135.00	0.21%	$0.00	0.00%	$0.00	0.00%
Women	$5,516,285.00	16.41%	$8,361,739.00	22.99%	$3,758,012.00	16.95%	$3,445,218.00	18.17%	$2,563,555.00	23.21%
Total	$33,621,350.00	100.00%	$36,375,186.00	100.00%	$22,168,598.00	100.00%	$18,957,661.00	100.00%	$11,044,724.00	100.00%

past 2 years, only one project has been awarded to the second lower bidder because of DBE problems.

Once a project begins, the prime contractor is required to submit to the affirmative action office, through the project office, the fully executed agreements between the prime contractor and the DBEs. These contracts are used by the project office and the affirmative action office to ensure DBE compliance. Although field personnel have the day-to-day responsibility for monitoring compliance, affirmative action staff try to visit projects at least once. Projects with questionable prime contractors or DBEs are monitored more often.

At the completion of the project, the prime contractor is required to submit certification of payment affidavits. These affidavits certify what primes have paid or will pay DBEs for their work on projects. DBEs are required to confirm what the prime contractors submit. Retention money is held pending receipt of these affidavits. Staff in the affirmative action office compare the affidavits with the subcontracts to ensure that DBE goals were achieved. In addition, audit and analysis has included DBE compliance as a regular part of any audit.

During the past 4 years, increasing numbers of DBEs are working on projects that are state funded. These firms are competing in a market where they have no advantage. Their name, service, and reputation are getting the DBE work, not their certification status.

Many DBE owners worked previously as employees of non-DBE firms. They now are responsible for their own firms. In the past year, four DBEs have begun working as prime contractors on smaller jobs. Some of these jobs involve federal funds, whereas others are state funded.

From an informal comparison of the difference between the state's estimate and the low bid on federally assisted contracts and the state's estimate and the low bid on state funded contracts, the DBE program may not have higher costs. Because the state's estimate does not take into consideration DBE program requirements, increased competition may actually be lowering the cost of jobs.

The DBE program, despite all its successes, also has negative aspects. First, setting DBE goals on federally funded projects can be difficult. Until 1987, Interstate 10 through Phoenix was under original construction. This situation meant that there were large projects with a variety of work, such as reinforcing and structural steel; concrete curbs, gutter, noise walls, and paving; pipe; drilled shafts for cassions; signing; electrical; fence and guardrail; and trucking. Setting goals on these projects was easy. Now, with all of these projects under construction, setting goals is more difficult. Arizona has finally caught up with the rest of the country and must try to achieve 10 percent DBE participation on jobs that do not lend themselves to DBE goals. Landscaping projects now make up a large part of the federally funded work in Phoenix. However, the only available work for DBEs is supplying gravel, hauling, surveying, and perhaps, electrical. Work outside of the major metropolitan areas seems to be rebuilding bridges and milling, which have limited opportunities for the use of DBEs. Two DBE firms specializing in milling have decided not to work in Arizona in the future.

Despite these changes, the bidding process is still problematical. Even with the new procedure, one bidder failed to submit the 5-day information within the time frame stipulated in the contract. Another contractor indicated that specific DBEs would be used to meet the goal, only for the ADOT to discover the DBEs had not even submitted bids on the project. Another problem is that many new bidders are unaware of ADOT requirements. Consequently, after bids are opened, the affirmative action office must find them and explain the procedure to ensure that the required paperwork is completed properly and submitted on time. The 5-day period after bids are opened has also become an issue. Many DBEs have indicated that this 5-day period has led to intensified bid-shopping by the apparent low bidder. As a result, a committee has been established to discuss this matter and to develop recommendations addressing the situation.

During construction of a job, field personnel must monitor the DBEs. Often, the inspectors provide informal technical assistance to DBEs by showing certain aspects of the job. Field personnel must also determine when and if a DBE on a project has entered into an unauthorized second-tier subcontract with a non-DBE firm and whether this agreement exceeds the contract stipulated limit. In the recent past, lack of an approved agreement has led to a myriad of problems, such as unauthorized or unlicensed subcontractors on projects and the possibility of the prime contractor's not meeting DBE goals. Field personnel must also ensure that DBEs are submitting other required paperwork, such as Equal Employment Opportunity project workforce reports and certified payrolls.

With the DBE program, ADOT often finds itself in the middle of the relationship between prime and subcontractors. ADOT has held that its legal relationship is with the prime contractor or consultant on all projects, whereas the subcontractor or subconsultant has a relationship with the prime contractor. In the typical prime and subcontractor relationship, the prime is responsible for all work and notifies the subcontractor when changes are required. However, field personnel often find themselves communicating directly with the DBE subcontractors when problems arise with the DBE's work. Another issue that ADOT is often asked to mediate is scheduling. DBEs allege that prime contractors deliberately schedule their work so that they will experience difficulties, i.e., requiring the DBEs to report to projects but not being ready for them or requiring the DBEs to perform their subcontracts in small, distantly spaced pieces. Less-frequent problems arise from disputes over plan interpretations, personality conflicts, and scope of work of the subcontract.

The largest issue that ADOT finds itself in the middle of is prompt payment. When DBEs have not been paid when they believe they should have, they call the affirmative action office, field report staff, or the resident engineer. DBEs are told that ADOT cannot ensure payment, but instead tries to determine the reasons. Sometimes the problem results from something as simple as failure to submit required documents, such as certified payrolls or material certifications. Occasionally, prime contracts do not have the money even though ADOT has paid them. At other times, the problem is contractual. In these cases, ADOT advises the DBE to seek legal advice.

Because field report staff processes all paperwork for monthly progress pay estimates, DBEs often call to see what work ADOT is paying the prime contractor for that month. This action involves a great deal of work for field report staff. DBEs then base their invoices to the prime contractors on the quantities they were told. Occasionally, the quantities indicated by field report staff are different from the DBE's records, thus creating another dispute. When this situation occurs, the resident engineer is called in to resolve quantity disputes, expedite change orders or force account work, and special requests for early release of retentions. This action again requires the field personnel to circumvent traditional relationships with prime contractors.

A final problem arises when DBEs are unable or unwilling to perform their subcontracts. The construction industry holds that there are few DBEs qualified to do the work. However, when experienced and qualified DBEs submit bids that may be more realistic but higher, these bids are rejected. Subsequently, DBEs who submit lower unrealistic bids are used to meet the goals. These firms often experience difficulties performing their contracts, thereby perpetuating the stereotype.

When DBEs fail, ADOT must determine whether to hold the prime contractors to the DBE goals or to lower them. In all instances, prime contractors are required to submit evidence of their good-faith effort to meet the DBE goals. However, each project must be evaluated on its own merits when deciding to stay with or lower the DBE goal. Several factors then come into consideration. First, will the traveling public suffer if the project is delayed while the prime contractor seeks additional DBEs? Second, how is ADOT progressing toward its annual commitment? Third, what is the nature of the remaining work and are there DBEs who will do it? Fourth, how much time is remaining on the contract? Fifth, will the prime contractor file a claim for more money? Sixth, why did the DBE fail? All of these questions require a great deal of time and interaction between the affirmative action office, the resident engineer, and field personnel, field report staff, and FHWA.

In spite of these challenges, ADOT remains firmly committed to the DBE program. The department is proud of its successes and of those firms that have grown as a result of it.

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FHWA Viewpoint of the Disadvantaged Business Enterprise Program

DAVID S. GENDELL, DAVID C. GIBBS, AND CHARLES W. KLEMSTINE

The disadvantaged business enterprise (DBE) program and its predecessor, the minority business enterprise (MBE) program, have been a part of FHWA's federal-aid highway program since the early 1970s. The MBE program, early on, was integrated into the federal-aid highway program through each state's nondiscrimination affirmative action plans to ensure equal opportunity for individuals in federally assisted projects. The 1982 and 1987 federal-aid highway program reauthorization acts revamped the MBE program to the DBE program and established the clear statutory authority, which exists today. The 1982 and 1987 acts also implemented several key components of today's program, including a national 10 percent achievement goal, a precise eligibility definition, inclusion of women as a presumptive group, and uniform certification criteria. Each of these elements has significantly contributed to a program that has as its principal objective maximizing the opportunity of bona fide DBEs to participate in federally assisted contracts, while minimizing the potential of firms fraudulently trying to enter the program and failing to perform a commercially useful function. The program has shown considerable growth, especially since the enactment of a statutory goal in 1982. Since 1982, approximately $10.3 billion of federal-aid funds have been awarded to DBEs on federal-aid highway projects.

The promotion of increased participation by minority contractors in the highway construction industry is not a recent phenomenon. Several early legislative statutes and executive orders greatly shaped and defined FHWA's DBE program. Title VI of the Civil Rights Act of 1964 forbids discrimination in the provision of benefits, services, and participation in federally assisted programs. Executive Order 11625 (October 13, 1971) required that federal executive agencies develop comprehensive plans and programs to encourage minority business participation. Finally, President Carter's Urban Policy Statement of March 27, 1978, directed all federal agencies to triple federal contracting with minority businesses and to include minority business enterprise (MBE) goals in federal assistance programs.

As far back as 1975, when FHWA issued Order 4700.1 (August 28, 1975), the agency has had an affirmative action policy to provide minority businesses with the maximum practical opportunity to participate in the federal-aid highway construction program. Guidance set forth in the 1975 order represented the agency's first extensive attempt to foster a nationwide program to develop plans and program goals, establish performance monitoring and reporting systems, and evaluate the results.

D. S. Gendell, FHWA, 31 Hopkins Plaza, Room 1633, Baltimore, Md. 21201. D. C. Gibbs, FHWA, 105 Sixth Street, Ames, Iowa 50010. C. W. Klemstine, Contract Administration Branch, HHO-32, FHWA, 400 7th Street, S.W., Washington, D.C. 20590.

FHWA followed this order with implementing regulations under Title 23, Code of Federal Regulations (CFR), Part 230, on October 22, 1975. FHWA directed the state highway agencies' (SHAs') efforts in fulfilling the federally mandated minority business program. Several key elements of this new regulation were (a) the definition of the term minority business enterprise (MBE); (b) the requirement that SHAs take affirmative action to increase participation by minority businesses; (c) the requirement that prime contractors specify their intention to sublet any portion of the contract and, if so, take affirmative action to seek out and consider minority subcontractors; and (d) the incorporation of minority affirmative action clauses in the subcontract. The regulation defined an MBE as a business with at least 51 percent of the stock owned by minority group members.

Congress reinforced the concept of using federal financial assistance programs to promote minority businesses through enactment of the Public Works Employment Act of 1977. Section 103(f)(2) of that act contained a provision that 10 percent of the funds for local public works projects be expended with MBEs. This act, until passage of the 1982 Surface Transportation Assistance Act (STAA), was the only legislative statute mandating a specific minority business percent goal for a federally funded program.

The U.S. Department of Transportation's (DOT's) Order 4000.7A, issued on March 6, 1978, established the administrative framework for DOT's modal administrations MBE programs. DOT directed the establishment of strong affirmative action efforts to set and meet goals for increasing MBE involvement.

Recipients were required to present for approval an affirmative action program to promote MBE participation in federally assisted work. The term "good-faith effort" was first introduced in this order. Also, women were included in the definition of MBE.

DOT specified an 11-point affirmative action program for adoption by recipients. These 11 points were

1. A policy statement expressing a commitment to use MBEs in all aspects of procurement to the maximum extent feasible;
2. Appointment of a liaison officer and support staff to administer the program, noting authority, responsibility, and duties;
3. Percentage goals for the dollar value of work to be awarded to MBEs and reasonable written justification for these goals;
4. Procedures by which recipients seek affirmative action on MBE participation from major suppliers or contractors to the recipient;

5. Where allowable under local law and appropriate to meet MBE goals, procedures to carry out an MBE set-aside program;

6. Procedures to require MBEs to be identified by name when bids are submitted and to permit their legitimacy to be ascertained;

7. Procedures to ensure known MBEs are able to compete equally by arranging solicitations, time for presentation of bids, quantities, specifications, and delivery schedules;

8. Means of assistance in overcoming barriers to program participation such as through bonding, insurance, and technical assistance activities;

9. Information and communication programs for MBE opportunity awareness;

10. Opportunities for use of minority-owned banks; and

11. To the extent necessary, methods of requiring contractors and subcontractors to comply with provisions of the program outlined.

On March 31, 1980, DOT issued implementing regulations under Title 49, CFR, Part 23, for a uniform MBE program by which firms owned and controlled by minorities and women could participate in contracts let by recipients of DOT financial assistance.

This regulation superseded all MBE regulations, orders, directives, etc., previously issued and was the first in a series of DOT regulations addressing its preference programs for ethnic minorities and women, with others to follow in 1981, 1983, and 1987.

The 1980 regulation provided the backbone of FHWA's program as it exists today. Key elements of this regulation were that it (a) defined MBE program elements that had to be implemented by recipients as a condition of federal assistance, (b) required program implementation by the modal agencies, (c) permitted contract goals to be set by recipients, and (d) required certification of the eligibility of participating firms by recipients. The regulation also adopted Section 8(a) of the Small Business Act (SBA) that defined a minority under a number of specific ethnic and cultural races as well as a general definition that also covered members of other groups, or other individuals found to be economically and socially disadvantaged by the SBA.

This regulation delegates to each modal administration the responsibility to approve the methods for determining its overall program and contract MBE goals. All programs had to be approved by August 1, 1980. Content and format of these programs were to take basically the same shape as that prescribed by DOT's Order 4000.7A. For the first time, a requirement was in effect that directed establishment of distinct and separate goals for MBEs and what became commonly known as women business enterprises (WBEs).

These early goals were generally expressed in dollar amounts. In 1980, approximately 3.4 percent of federal-aid construction funds were awarded to MBE and WBE firms.

One provision of the 1980 regulation, later rescinded by a final rule published on April 27, 1981, was the adoption of an administrative finding called "conclusive presumption." If one bidder met the MBE contract goal and offered a reasonable price, then all bidders that did not meet the goal were conclusively presumed as not exerting sufficient reasonable efforts and therefore were ineligible to receive the contract. As noted, the conclusive presumption provision was rescinded in 1980 and replaced by one requiring contractors to make a good-faith effort.

The good-faith effort provision allowed the low bidder to receive the contract if meeting the MBE contract goal or satisfactorily demonstrating to the recipient that a good-faith effort had been made to do so. The final rule provided guidance criteria for determining what constituted good-faith effort.

In the evolution of FHWA's present DBE program, the major event was the passage of the 1982 STAA. This act was signed into law on January 6, 1983, and contained Section 105(f), which imposed the following requirement on federally assisted highway and transit projects:

> Except to the extent that the Secretary determines otherwise, not less than 10 percent of the amounts authorized to be appropriated under this Act shall be expended with small business concerns owned and controlled by socially and economically disadvantaged individuals as defined by section 8(d) of the Small Business Act . . . and relevant subcontracting regulations promulgated pursuant thereto.

Section 105(f) made three substantial changes to the broad parameters of the MBE program as conceived in the 1970s. First, passage of the act for the first time provided clear statutory authority for DOT's program. Second, the act established a 10 percent goal for program participation. Finally, the definition of eligible individuals and groups was adopted from Section 8(d) of SBA. Section 8(d) provides a broader ethnic base of eligible groups and individuals. By defining participation under Section 8(d) of SBA, the act shifted eligibility criteria from minority status to disadvantaged status. The term "disadvantaged business enterprise" became the name for DOT's program. The 10 percent participation requirement applied only to ethnic minorities. Nonminority WBEs were not included but continued as eligible participants under the voluntary language established in Title 49, CFR Part 23, in 1980.

With passage of the 1982 Act and resulting program changes, DOT published implementing regulations that added Subpart D to existing 49 CFR 23 on July 21, 1983. The new regulation dealt primarily with DOT's interpretation of the congressional intent in setting a goal of not less than 10 percent and the newly defined eligibility criteria. DOT went to great lengths to repudiate any thought that the 10 percent goal for the DBE program was to be handled as a fixed set-aside or firm-quota program. The legislative history of Section 8(d) indicated that members of presumptive groups were to be conclusively considered as socially and economically disadvantaged, regardless of their actual economic situation. However, DOT's regulation included a rebuttable presumption requirement. Simply, this requirement states that belonging to one of the presumptive groups does not conclusively render an individual socially and economically disadvantaged.

A challenge provision was included in the regulation whereby recipients were required to adopt a procedure to allow anyone to challenge the presumption of being disadvantaged.

CURRENT PROGRAM IMPLEMENTATION

As the previous discussion indicates, the DBE program has changed considerably during its history. Passage of the 1987 Surface Transportation and Uniform Relocation Assistance

Act (STURAA) brought even more changes to the program. STURAA was enacted on April 2, 1987. Under this law, Section 105(f) of the 1982 STAA was replaced by Section 106(c) as the statutory authority for the DBE program. Section 106(c) required four basic changes to the DBE program:

- Women are to be presumed to be socially and economically disadvantaged individuals and, therefore, separate goals for DBEs and WBEs would no longer be permitted;
- In order to be an eligible DBE, a firm or group of firms controlled by the same individual cannot have average annual gross receipts over the preceding three fiscal years in excess of $14 million, as adjusted by the Secretary of Transportation for inflation;
- DOT must establish uniform certification criteria for DBE firms; and
- Recipients are required to update their directories of eligible DBE firms annually, including the location of such firms.

On October 21, 1987, a final rule developed by the Office of the Secretary implementing Section 106(c) of the STURAA was published in the *Federal Register*. The final rule amended the existing DBE regulation (49 CFR 23) to include changes mandated by Section 106(c) and finalized several other changes that had been pending. Other changes included expanding the definition of "Hispanic" to include Portuguese and finalizing a change in the procedures for crediting the value of goods received from DBE suppliers toward DBE goals—raising the credit from 20 to 60 percent.

INCLUSION OF WBEs

With the addition of women as part of the presumptive group, retaining a two-goal system was no longer permissible. The legislative history of Section 106(c) clearly indicated that Congress intended DOT to adopt a one-goal system.

By memorandum dated August 26, 1987, the FHWA administrator informed each state that Section 106(c) did not allow separate DBE and WBE goals. Each state would be allowed to specify only a single DBE goal on future federal-aid contracts. This change became effective October 1, 1987.

A number of complaints have been received from WBEs and ethnic minorities claiming that the inclusion of WBEs in the definition of DBEs has resulted in one group receiving a disproportionate share of the work to the detriment of the other group. In several instances, inclusion of WBEs and designation of a single goal resulted in a disproportionate amount of work being obtained by WBEs. For example, on two large projects in Illinois (Dan Ryan expressway in Chicago and the Martin Luther King bridge in East St. Louis), WBEs accounted for approximately 65 percent of the DBE goal. This situation resulted in protests from the black contracting community and threats from the mayors of Chicago and East St. Louis to stop the projects unless black contractors received a larger share of the work.

On average, since implementation of Section 106(c), WBEs have gained an increasing share of the total DBE program.

Some states have attempted to offset any disproportionate effects of the single goal requirement by implementing some remedial actions. These actions have included (a) maintaining separate goals on 100 percent state-funded projects to compensate for the disparity on single-goal federal-aid projects, (b) establishing higher contract goals to provide additional work for competing ethnic minority firms and women-owned firms, and (c) increasing the number of DBE set-aside contracts to provide additional contracting opportunities.

CERTIFICATION

In continuing the DBE program under Section 106(c), effective enforcement of eligibility requirements is a key part of ensuring that the program achieves its purpose. Keeping ineligible so-called "front" firms out is essential to maintaining the program's integrity and credibility.

As previously noted, Section 106(c) required DOT to establish minimum uniform criteria for the SHA to use in certifying DBEs for eligibility to participate in the federal-aid highway program. This criteria were included in DOT's final rule of October 21, 1987. Minimum certifying criteria include, but are not limited to, on-site visits, personal interviews, analysis of stock ownership, listing of equipment, analysis of bonding capacity, listing of work completed, resumes of principal owners, financial capacity, and type of work preferred.

Uniform certification criteria has had the following positive impacts on the DBE program:

- Decreased the paperwork burden on DBEs, and
- Helped eliminate so-called "fronts" by requiring verification of information.

In 1988, AASHTO's Special Committee on the DBE program released a special practices manual entitled *Guidelines for D/WBE Program Administration* (*1*). This manual focuses on the results of a comprehensive survey, conducted by the committee, of critical methods and procedures in the administration of DBE programs. Suggestions are offered for what appear to be the most successful procedures on the basis of the committee's study of the overall program.

GOOD-FAITH EFFORT

STURAA did not include any provisions that would directly affect previous good-faith effort procedures. FHWA administrator R. A. Barnhart, in his memorandum of June 20, 1985, strongly urged states to implement a good-faith effort process and set realistic contract goals. Since this memorandum was issued, states have been encouraged to set realistically achievable contract goals and to establish a process that recognizes good-faith efforts to achieve such goals. A survey of SHAs revealed that at least 46 agencies use a contract award mechanism that recognizes good-faith efforts in meeting DBE contract goals.

A few states have expressed concerns about the subjective process of reviewing good-faith effort submissions that could lead to favoritism in awarding contracts. FHWA believes these concerns can be alleviated if realistic contract goals are established that can be met with legitimate DBEs.

DBE DIRECTORIES AND DBE SIZE RESTRICTIONS

These requirements have had minimal impact on the DBE program. Most states previously had DBE directories. FHWA has encouraged states to specify, during the annual updating of their DBE directories, the geographical areas where DBEs prefer to work.

The purpose of revising the size restriction is to ensure that the DBE program meets its objective of helping small minority and women-owned businesses become self-sufficient by graduating those firms that have average annual gross receipts of more than $14 million over a 3-year period. The $14-million ceiling includes revenues of all affiliates of the firm owned and controlled by the same individuals as well as that of the firm itself. In order to monitor this requirement properly, each state had to evaluate its DBE certification requirements. In this way, SHAs would ensure that adequate information is being collected to make this determination. However, the size restriction affected only a small number of firms.

On February 14, 1990, a notice was published by the Secretary of Transportation in the *Federal Register* adjusting the $14-million ceiling for inflation to $14.65 million.

CREDIT FOR MATERIAL SUPPLIERS

DOT's final rule permits 60 percent of the value of goods purchased from DBE regular dealers to be counted toward a contractor's or recipient's goal. The previous allowable amount was 20 percent. A regular dealer has been defined as a firm that owns, operates, or maintains a store, a warehouse, or other establishment in which the materials or supplies required for the contract are bought, kept in stock, and regularly sold to the public. Dealers in bulk items such as steel, cement, gravel, and petroleum products are not required to maintain the items in stock, but they must own or operate distribution equipment.

This increase in allowable percentage should encourage a larger number of minority and women material suppliers to seek certification. DOT is planning to reevaluate the 60 percent value after several years to determine if further adjustment is necessary. The final rule also allows credit for fees and commissions earned by suppliers and haulers, who are not regular dealers, provided the amounts paid are customary in the industry.

PROGRAM IMPACTS

In recent years, the highway construction industry including both governmental agencies (i.e., FHWA, SHAs, and AASHTO) and the contracting industry [i.e., Associated General Contractors (AGC)] have undertaken several studies to assess the impact of the FHWA's DBE program on the federal-aid highway construction program. Emphasis has been on the impact of the 1982 and 1987 acts and their 10 percent goal requirements. The studies basically have concluded that the program has had and will continue to have a significant impact on the construction industry. However, the exact level or extent of the impact remained a matter of interpretation under each of the studies.

The single most prominent positive impact has been the increase in the number and dollar amount of awards to ethnic

TABLE 1 MBE AND WBE ACHIEVEMENTS AND PERCENT OF PARTICIPATION IN THE FEDERAL-AID HIGHWAY PROGRAM

FISCAL YEAR	NO. OF CONTRACT AWARDS	AWARD AMOUNT ($1,000)	PERCENT OF TOTAL PROGRAM
1976	702	$63,800	----
1977	751	$71,530	----
1978	1,200	$126,090	----
1979	1,574	$163,700	----
1980	1,963	$281,940	3.4
1981	5,602	$520,081	6.1
1982	7,201	$653,151	8.3
1983	10,952	$974,688	8.8
1984	17,291	$1,564,471	16.8
1985	19,359	$1,585,417	14.1
1986	18,753	$1,526,362	14.0
1987	16,441	$1,525,722	14.7
1988	14,914	$1,586,991	14.9
1989	14,277	$1,571,889	14.4

minority and women-owned businesses presented in Table 1. The negative aspects are somewhat dependent on the source of data, but generally there has been some increase (amount not known) in the overall cost of contract administration and construction. Also, nonminority contractors in the specialty construction trades have been most singularly affected. The impact of FHWA's DBE program on the federal-aid highway program since 1976 is also presented in Table 1. The award amounts shown are the federal-aid funds only.

The figures represent the level of participation by minority and women-owned businesses in the years up to 1987, and since 1987 the amount of DBE participation in the federal-aid highway construction program. From these figures, the level and percentage of ethnic minority and women-owned businesses involvement significantly increased after passage of the 1982 STAA with the introduction of the 10 percent goal level.

SANCTIONS

Much of the focus has been on the certification process as the primary means of identifying DBE fronts and frauds and ensuring the integrity of the DBE program. Another significant area of concern must be the use of certifiable DBE firms by non-DBE prime contractors as pass-throughs in which the DBEs fail to perform any meaningful functions. Imposing sanctions on a DBE through certification measures is not an effective means of controlling this kind of program abuse.

The lack of prosecution by the U.S. Department of Justice (DOJ) of discovered irregularities continues to frustrate many investigative bodies, e.g., the Office of Inspector General. Lack of prosecution appears to result from the need to focus limited DOT or DOJ resources in other areas such as bid rigging.

Civil sanctions at the state level, more particularly contract administration remedies, appear to offer much greater potential for swift and equitable treatment of DBE program abuse. Contract remedies that are most commonly used are assessment of monetary penalties, withholding progress payments, suspension of work, and disqualification from bidding for a specific time period.

Improved contract administration and increased reliance on administrative sanctions at the state level rather than criminal prosecutions are the most effective ways of controlling abuse of contract requirements.

REFERENCE

1. Guidelines for D/WBE Program Administration. AASHTO, Washington, D.C., 1988.

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Women Business Enterprise/Contractor Viewpoint of the Disadvantaged Business Enterprise Program

ANNE BIGANE WILSON

The women business enterprise (WBE) viewpoint of the disadvantaged business enterprise (DBE) program is discussed. Various aspects of the set-aside program are presented as they affect a paving company in Chicago. Problems relating to being a WBE contractor are also discussed.

Bigane Paving Company is a fourth-generation family company. The company was founded in 1896 by John Bigane, great-grandfather of the current owners. Originally, the business concentrated on retail coal and oil sales, but started to diversify in the early 1950s as the third generation entered the business. Asphalt paving has been the major revenue source for the past 20 years. Diversification was especially prudent because the market for fossil fuel heat changed greatly in the early 1970s.

Currently, the focus of the company has been commercial and industrial asphalt construction. Approximately 50 percent of last year's sales were from the public sector—school districts, municipalities, and local mass transit. The remainder of sales are divided between work directly for the owner and for a general contractor. Annual sales have been in the $1 million range for the past 3 years.

The present owners are Anne Bigane Wilson and Sheila M. Bigane. Anne Wilson is a licensed engineer, who joined the firm after working for a large highway-heavy construction company for 4 years. She is now President as well as the Chief Estimator for Bigane. Sheila Bigane came to work for Bigane directly from college. She is the Corporate Treasurer as well as Office Manager.

Bigane Paving Company decided to apply for women business enterprises (WBE) certification for several reasons. Although Bigane was an established firm, the owners knew that each generation had to prove itself in the industry. In addition, the decision to apply for certification was strategic—to expand business opportunities in the types of projects and owners available. Certification has provided the company with additional access to various owners and general contractors. Besides the publicly financed projects with DBE and WBE goals, several large corporations such as AT&T and Illinois Bell Telephone have established goals for their firms.

Certification proved to be more difficult for an established company than for a new firm. Initial certification with the Illinois Department of Transportation (IDOT) took almost 12 months to complete. As an established company in the construction industry, Bigane Paving had the necessary experience for certification. Typical of many small-family firms, some of the corporate records from the early years were not available. These records included canceled stock certificates of deceased stockholders that had been repurchased by the company under terms of a buy-sell agreement. Another problem encountered was the changing of the guard from one generation to the next.

Mr. Bigane was still active with the company when his daughters joined the firm. For Bigane Paving to survive, the next generation of owners had to be fully committed to the company. Mr. Bigane felt that his commitment to his daughters and their commitment to the company could best be secured through shared ownership and control. This policy proved to be a wise business decision when he died suddenly in the summer of 1987. Even though Anne and Sheila had the education and technical experience needed to run this type of company, their father's motives for transferring ownership might have been to obtain the competitive advantage of WBE ownership. However, this policy of transferring ownership had a historical precedent within the company. Both Mr. Bigane and his father received their shares of the company shortly after they came to work for Bigane Paving full time. Another deciding factor was the fact that Anne and Sheila came from a family of all daughters. (A younger brother interested in taking over the company might have delayed the transfer process.)

In the Chicago area, certification by IDOT is not sufficient because local agencies have their own certification processes. At this time, Bigane is certified by IDOT, city of Chicago, Chicago Transit Authority, PACE (the suburban bus division of the RTA), Metra (the Metropolitan Rail division of the RTA), and state of Illinois Central Management Services. Some local agencies, such as the Metropolitan Sanitary District, certify firms on a job-by-job basis. On the federal level, certification is also handled agency by agency. None of these agencies will accept the certification of any of the other agencies.

Each certifying agency requires roughly the same information concerning ownership, management, and technical expertise. This information and documentation can amount to several hundred pages. Each agency uses a different form. All of these agencies are governed by the same standards, but each agency interprets them differently. This information must be submitted for recertification on an annual basis. Most agencies seem to be trying to simplify the recertification process. Certification has almost gotten to the point that a full-time employee is needed to keep all of the paperwork current.

Bigane Paving Co., 3583 Archer Avenue, Chicago, Ill. 60609.

As a small business, this added expense could be put to a much better use.

Universal centralized certification, such as has been proposed in HR 2351, the Women's Business Equity Act, would greatly help this situation. The bill would streamline the entire process by using one set of standards and procedures applied equally to all firms. The cost and amount of paperwork currently required discourages many DBE and WBE firms from participating in this market. This bill would allow certifying agencies to use resources, currently spent on certification, for support services for DBE and WBE contractors.

Bigane Paving Company is fortunate that as an existing concern, many crucial business relationships have been established and secured by the previous owners. However, the company is not trouble-free. Bigane has been banking with two local banks for over 30 years. One is a family-owned neighborhood bank and the other is a downtown bank. Bigane Paving suffered a series of setbacks that started in the mid-1970s with the deaths of two of the company's partners. After the recession of the early 1980s, the company began a rebuilding program that included the addition of the current owners to the management team. The neighborhood bank seemed to lose interest in the company about that time. This bank continued to make funds available and to service the account, but the loan officer never seemed to be concerned about the progress of the firm. The only bank-initiated contact was in regards to audited statements at the end of the fiscal year. The situation seemed to decline even further after Mr. Bigane's death. On the other hand, the downtown bank was always available. The commercial loan officer kept in constant contact with the company and was very helpful and supportive during the transition after Mr. Bigane's death. For this bank, a smooth transition was the norm when there was a change in loan officer for whatever reason. At the neighborhood bank, the loan officer retired during the off season and Bigane was notified only when they tried to contact him because the company was preparing for the new season. For this reason, all accounts were recently consolidated with the downtown bank. This expanded banking relationship has proved beneficial.

Commercial insurance has been a problem for the entire industry, but especially for small firms. The insurance crunch of the last several years is just beginning to ease. Bigane was again fortunate to have been an existing firm, but also to have maintained an ongoing relationship with a single insurance carrier. Cost of liability insurance in recent years has skyrocketed. In addition, the amount of insurance being requested by general contractors has also increased. The importance of maintaining a good safety record cannot be stressed enough.

The bonding market is the most difficult for all small contractors, but especially for DBE and WBE contractors. In recent years, bonding companies have set strict financial guidelines before bonding a contractor. Bigane had been with one bonding company for 20 years, without a single claim filed against one of its bonds. In 1985, Bigane was dropped for not meeting these new guidelines, which included \$100,000 in equity and working capital. These limits are hard enough for any small firm to meet, but are almost impossible for a new firm. Bigane bonds about 50 percent of the company's annual volume. These projects are usually between \$50,000 and \$200,000 each. Bigane found a new bonding company through the diligent efforts of their agent. After Mr. Bigane's death, the underwriter suspended all bonding privileges for approximately 1 month. The underwriter pushed for an interim audit of the company's books, even after company life insurance proceeds were received. These funds brought the working capital and equity into line with the bonding company minimums. However, the bonding company was not satisfied with the in-house statements provided. After much discussion and persistence on the part on the agent, the underwriter backed down. The time and cost involved were prohibitive especially in the middle of the season. Bonding seems to be the one area of the construction industry in which building a good reputation and record means absolutely nothing if financial statements do not meet strict requirements.

Bigane Paving Company takes full advantage of various support services available on the local level. IDOT offers a number of seminars each year as a means of continuing education for all DBE contractors. These services have helped Bigane reach a higher level of professionalism in addition to providing an extra resource for a growing small business. The seminars are also a means of becoming more familiar with IDOT procedures needed for a smooth-flowing project. These seminars permit a great deal of networking among the various DBE contractors that can be as beneficial as the seminar material.

The DBE and WBE programs have broadened the market for Bigane. The company has recently become involved with the Chicago Transit Authority's patching program. The DBE goals have also given Bigane a start with several general contractors on a variety of projects throughout the Chicago area. This introduction will hopefully expand to the contractor's other projects without any goals, as the program was designed to work.

Goals for minority participation on the federal level are handled agency by agency and have been used in the construction industry since the 1960s. In the last 10 years, goals for women have been added. This addition is an administrative change only because no legislation exists for enforcement. Goals were 10 percent minority business enterprise (MBE) and 3.25 percent WBE. In 1987, the Surface Transportation Act combined these goals into one 10 percent DBE goal. This change was actually a reduction in overall participation. However, the combined goal has helped women contractors in several ways. By being declared disadvantaged, various support programs previously only available to minority contractors were now available to women. These programs included insurance, bonding, and technical programs offered by the U.S. Department of Transportation. Because 70 to 80 percent of WBE contractors are Anglo women, they have seen the greatest improvement in terms of support programs available.

The effects of combined goals on actual participation seem to vary greatly with the region of the country. In Western states, the combined goals seem to have been most beneficial for MBE contractors. For example, the California Department of Transportation reports that before the combined goals WBE participation was 3½ to 4 percent. The current figure is approximately 2¾ percent, which represents a 30 percent decrease. In the Midwest, women benefited because these areas as a whole do not have as great a minority population.

A variety of opinions exist as to the effect of the combined goals, even among women's groups. Women Construction Owners and Executives of the United States (WCOE) does not support the combined goals. WCOE feels that a dual-goal system would best serve the spirit of the set-aside programs. MBEs and WBEs face totally different problems when entering the construction market. Female contractors must try to break into a market that has been traditionally male-dominated. Many women feel that most contractors would rather deal with a minority contractor who is male than a woman contractor. In addition, general contractors seem more concerned about meeting MBE goals rather than WBE goals.

WCOE is working with several members of Congress to introduce legislation that will mandate universal goals for all federal construction work. This legislation will standardize the current system, which has different goals set by each contracting agency. WCOE is also working with Congress on a universal certification process that will help eliminate the duplication of paperwork and effort that is currently necessary.

HR 2351, The Women's Business Equity Act of 1989, will also create a permanent Office of Women Business Ownership. This office will provide technical assistance needed in operating a business and will parallel the assistance available to minority businesses through the minority business development centers in the U.S. Department of Commerce. These centers are not currently available to women.

The DBE and WBE program has made advances in bringing formerly excluded groups into the construction industry. This action can continue if all those concerned work together rather than only for their own interest. Minorities and women face different problems. To argue over which is the most disadvantaged is a waste of time and money. If the goal is to give everyone involved an equal chance in a thriving industry, the money would be best spent in educating the disadvantaged in the ways of the industry.

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Is Women Business Enterprise Discrimination a Reality?

BARBARA JOANN PAYNE

Day-to-day problems faced by women who are attempting to gain or maintain their certification in the disadvantaged business program (DBE) are examined. Examples are offered of individual case histories documenting discrimination against female DBE participants and viable suggestions are made for remedying this discrimination.

Before creation of the disadvantaged business enterprise (DBE) program by Congress in 1982, two good-faith efforts existed for minorities and women in the U.S. Department of Transportation (DOT). However, with the creation of the DBE program, Congress placed the minority component into the DBE program and omitted women. This Congressional action devastated the good-faith women business enterprise (WBE) program. From January 1983 through May 1987, minority participation grew from 5 to 13 percent, whereas female participation remained stagnant. The only hope for women to prove their abilities in this industry would have to come through Congressional action. Congress had pledged assistance to disadvantaged groups and the time had come to demonstrate to Congress that women in the highway construction industry were members of one of the most disadvantaged groups in the nation.

When one looks at the reasons concerning affirmative action for minorities in our society, one cannot overlook the same parallels women have had to face in securing equal opportunity in all areas of social, economic, and political life. History afforded seeing the truth about events and struggles in the development of the nation. The truth is that women have had the same struggle for equality and justice in the nation as minorities. The truth is that white males make more money than minority males, minority males make more money than white females, and white females make more money than minority females. The truth is that minority males won the right to vote before women. The truth is that many private clubs that once excluded minority males in the 1960s are the same private clubs that even today exclude white single females. The truth is, there are more elected minorities in our nation on the local, state, and federal levels than women. This truth is reflected in the fact that there are only 25 females in the U.S. Congress. Women, like minorities, have been shackled by the chains of discrimination and second-class citizenship, fighting for the right to participate.

In 1982, when Congress created the DBE program, women were excluded. This action by Congress would devastate the good-faith administrative WBE program for the next 5 years. The only chance women had to grow and develop and have the same opportunities in the highway construction industry as minorities was to be included in the DBE program under one goal. The survival of women and their right to participate as viable, responsible contributing citizens lay in that effort. The exclusion of women in 1982 was unfair because women also deserved a helping hand in the construction industry. However, Congress has corrected the wrong and has recognized women as a part of Congress' efforts to affirm its commitment to helping disadvantaged groups.

Following extensive lobbying efforts by the National Women Business Enterprise Association (NWBEA), a nonprofit trade association for women-owned businesses in the highway construction industry, Congress recognized that women were not going to get even a sliver of the transportation pie without its intervention. In 1987, women were added to the DBE program, but were required to share the 10 percent procurement goal for minorities. Since inclusion of women in the DBE program, female participation has increased by 80 percent.

However, concentrated, discriminatory efforts continue in an attempt to remove women from the DBE program. Individuals who implement the DBE program, their leaders, and certified male DBEs have become more aggressive each day in their efforts to place pressure on women in the highway construction industry so that women will want nothing to do with the industry, much less the certification process.

The initial hope felt by females across the nation when first included in the procurement goals was quickly dissipated by the discriminatory interpretations of the federal regulations governing the DBE program. According to the *Federal Register* (*1*) the Surface Transportation Assistance Act of 1982 maintained the basic structure of the DBE program intact, with the exception of Section 106 (c)(2)(B) that provides that women, like black Americans, Hispanic Americans, and other groups currently designated in the regulations, are presumed to be socially and economically disadvantaged individuals for purposes of the DBE program.

Despite Congress' presumption (drawn following extensive, detailed testimony before Congress enumerating blatant acts of gender-based discrimination), female DBEs are continually called on to prove their social disadvantage or lose their certification. For example, experience is not a requirement to be eligible for the DBE program; however, DOT has an implied regulation that requires WBEs to have technical expertise. Historically, women have been excluded from the highway construction industry except for traditional female roles. Traditional experience as secretary, administrator, etc., does not meet DOT's definition of technical expertise. For women,

National Women Business Enterprise Association, 1313 North Hills Boulevard, Suite 302, North Little Rock, Ark. 72114.

technical expertise was impossible until recently. It is interesting that U.S. DOT believes that a secretary would not absorb the knowledge and experience necessary to operate her own company.

In many cases, women start their own company and do not apply for DBE certification for several years. Even these women, who successfully run their own companies for extended periods of time, are denied certification on the basis of lack of experience or expertise. Other areas of discrimination based on subjective interpretations of federal regulations are contributions of capital, stocks, requests for personal income tax returns, family-owned businesses, community property, cosignatures on loans, joint checking accounts, and the hiring of nonminorities and women.

Women's roles in American society have changed dramatically over the past 15 years. These extraordinary changes are evidenced by the skills women have acquired and the socioeconomic status many have attained, independently.

This advancement did not come without pain, nor without government assistance. Passage of educational legislation in 1972, including Title IX (which prohibited sex discrimination in educational institutions receiving federal funds), marked the beginning of the end of economic repression for women.

According to the National Organization for Women (*2*), between 1972 and 1976, the percentage of women entering male-dominated fields consistently increased. In 1966, 5.9 percent of first-year college women planned careers in male-dominated fields. By 1976, this figure had increased to 19.4 percent, and in 1986, to 25.2 percent. Another historically male-dominated area that has shown dramatic increases in opportunities for women is in sports. For example, the percentage of money spent on women in college sports increased from 1 percent in 1972 to 16 percent in 1982. In 1984, there were about 10,000 college athletic scholarships available for women.

The percentage of women entering professional fields has also drastically increased (*2*).

Field	*1972 (%)*	*1980 (%)*
Law	10	34
Medical	11	26
Engineering	1	25
Veterinary	12	30
MBA Degrees	4	25

Felice Schwartz (*3*) reported that over the past decade the increase in the number of women graduating from leading universities has been greater than the increase in the total number of graduates and that these women are well represented in the top 10 percent of their class. If businesses want to hire the best students then these businesses have to start hiring women. By not hiring women the businesses will have to dig deeper and settle for the least competent candidates.

Women are making progress, but against all odds. According to the most recent Internal Revenue Service figures (*4*), from 1977 to 1985 women-owned sole proprietorship nearly doubled, with an increase from 1.9 to 3.7 million. Women already own half of the retail establishments and three-fourths of the service companies in the United States and, given recent trends, by the year 2000 women may own 50 percent of all U.S. businesses. Women are starting businesses at a rate two times faster than men and are clearly the fastest growing segment of the entrepreneurial community.

Because the vast majority of these female entrepreneurs enter professional and typical service businesses, their influence will continue to grow as the country shifts further from a manufacturing-oriented economy to one based on service, high-technology, and information. In addition, women can be found succeeding in virtually every industrial category. According to a 1988 report (*4*) on the state of small businesses, expansion of women-owned businesses in nontraditional industry has increased faster than for retail trade.

Women-owned businesses are making significant contributions to the economy. Gross receipts for women-owned sole proprietorships was approximately $100 billion in 1982, according to the U.S. Bureau of Census (*4*). However, the total economic impact of these businesses far exceeds this level by taking into account the multiplier effect of these dollars as they turn over in the economy. Using a conservative estimate that each dollar will multiply 2.5 times in the local economy, women-owned businesses were already contributing $250 billion to the national economy 5 years ago. Yet, women procure less than 1.1 percent with the federal government. The time has come for government to extend DBE procurement to all federal agencies.

However, given this level of contribution to the economy, WBEs in the construction industry face a myriad of problems. For example, women have more trouble receiving bonding than any other group in the highway construction industry (*4*). In addition, lending institutions refuse to lend money to women more often than men (*4*). When lending institutions do lend money to women, a cosigner is usually required. Ironically, by using a cosigner women jeopardize their certification status in the DBE program. As a result, WBEs tend to start their businesses with substantially less capital than men and usually with no borrowed capital. According to the U.S. Department of Labor, 80 percent of all new WBEs are started from scratch and are not reformed or inherited companies.

In the highway construction industry, the participation of women has been limited in the past because of discrimination; therefore, the only experience women received was in traditional female areas. In addition, WBEs have to contend with the realization that men prefer to do business with men, regardless of color.

For the DBE program, the picture is not any brighter. For example, 60 percent of all WBEs who apply for DBE certification or recertification are refused. Certification and recertification in the DBE program is four times harder for women than for men. This discrimination is reflected in the number of certified WBEs when compared with the number of MBEs. WBEs number about 4,000 whereas MBEs number about 10,000 (*5*). Apparently, the same standards are not used in certifying WBEs that are used for MBEs.

DBE REGULATIONS AND 'JANE CROW' INTERPRETATIONS

In Texas, one WBE experienced subjective interpretation of DBE regulations regarding certification with the city of Fort Worth. Although the WBE had been certified with the city for 6 years, a narrative statement of why the woman-owner was socially and economically disadvantaged was required. Failure to submit this statement would have resulted in decertification.

How can a WBE that had been certified for many years suddenly and with virtually no change in operation not be eligible for the DBE program? An answer may lie in the personal interpretations of DBE regulations by DOT employees overseeing DBE certification, the majority of whom are male and minority males. In other words, the fate of women-owned transportation industry businesses has been placed in the hands of the one group who perceives themselves to be threatened by WBEs.

NWBEA refers to the discriminatory regulations as interpreted by DOT, as Jane Crow regulations. Following the Civil War, the Southern States passed what was known as the Jim Crow laws. Jim Crow laws were passed to exclude blacks from participating in the American democracy. Political rhetoric claimed that these laws were applied equally, when in reality they placed great hardships on only the black population. Women face the same situation with Jane Crow regulations. Interpretation of these regulations are in themselves discriminatory against women when applied equally to women and males. Like Jim Crow, the reality is the regulations really only apply to women. As mentioned previously, the women who had been certified since 1980 and participated in the highway construction industry for 10 years are now being denied recertification.

Some of the reasons being used to deny female certification include transfers of stock from a male before 1980, or that a woman-owner's father left an inheritance. Such reasons mean a WBE can no longer participate in the DBE program because they have received money from a male.

The handful of women who have managed to maintain their certification are in constant jeopardy of losing that certification. According to the FHWA civil rights director of Region 6, noncertification of women-owned businesses has increased to 60 percent over the last 3 years and newly formed women-owned companies are finding it virtually impossible to obtain certification (personal communication).

DEFINITION OF DBEs IN FEDERAL REGULATIONS

Federal regulations (49 CFR Parts 23.5 and 23.53a), define a DBE as a small business concern that is controlled by one or more minorities or women. Owned and controlled means at least 51 percent is owned by one or more minorities or women. Stock must be owned by one or more minorities or women and the company's management and daily operations must be controlled by one or more of such individuals.

The regulations go further, saying (a) the owner must be a member of a minority group, or a woman; (b) the business must be independent; (c) there may not be any formal or informal restrictions that limit the customary discretion of the minority or woman owners; (d) all securities that constitute ownership must be real and substantial and must go beyond the corporate record; (e) the minority or woman owner must enjoy the profits and the risk of ownership; (f) the minority or woman owner must possess the power to direct or cause the direction of the management of policies of the firm; (g) the minority or woman owner must possess the power to make day-to-day, as well as major, decisions on matters of management, policy, and operation; (h) a nonminority owner may not be disproportionately responsible for operation of the firm; and (i) those persons having the ultimate power to hire and fire managers are considered controllers of the business.

In context, these regulations are justifiable and reasonable. However, the interpretations that are being applied to these rules and that have led to the decertification of 60 percent of the WBEs over the past few years can hardly be seen as just.

FINANCIAL ASPECTS OF WBEs AND DBE REGULATIONS

DBE regulations specify that securities constituting ownership must be held directly by the DBE owner. However, for a woman, this rule means that she cannot inherit money from her father, brother, male cousin, or male friend and put that money into a highway construction company. If a minority male inherits money from his father, he is not penalized because the chances are that his father was minority as well (*5*).

Furthermore, if a woman worked and contributed funds to a joint checking account or savings account and tries to draw from those funds to establish a business, according to the Jane Crow interpretations the money is not hers but her husband's, and she is therefore disqualified as a DBE. However, if monies are generated from a joint savings or checking account from a minority, no penalty exists because chances are that the minority male is married to a minority female (*5*).

DOT has consistently held that ownership interests acquired through using funds owned jointly by a female owner and her husband do not meet the real and substantial requirement of the federal regulations, despite the fact that there is no mention in the federal regulations that funds from a joint checking account or savings account are not real and substantial or cannot be used by the female. Such an interpretation is inherently wrong and penalizes a woman for being married and administering her finances, as most married people do, through joint accounts (*5*).

The assumption that the male contributes more funds than the female to a joint checking or savings account is justifiable. For a woman to contribute equal funds to a joint checking account is virtually impossible when she has historically been discriminated against economically. A woman makes 65 cents to every $1.00 her male counterpart earns, so meeting this requirement may be impossible.

State transportation departments and DOT have also held that a WBE will not meet federal requirements if her capital contribution came from a bank loan on which her husband cosigned, regardless of who repays the loan, regardless of what collateral is used, and even if the banking institution required the signature.

Federal regulations must be interpreted using common sense with regard to current business practices. Today, it is still difficult, if not impossible, for a married woman to receive a loan from a bank without her husband's signature.

In U.S. society, lending institutions have not recognized that women are separate entities. And not until just recently, with the passage of a bill from Congress, was the practice made illegal for banks to require women to have cosignatures on commercial loans. However, in the case of the minority male, a joint checking account, savings account, or joint signature on a loan (unless from a nonminority spouse) would not result in the decertification of a minority company.

The only purpose that could possibly be served by this interpretation is to preclude women from participating in the program.

Perhaps the most tragic Jane Crow interpretation comes from an example of women who have been in the highway construction business for 8 to 10 years and have sole financial and managerial responsibilities of their company, including certification under these same regulations, and are now being decertified because states are demanding proof that her business was established with independent funds.

The following notification of decertification for a WBE company is further evidence that the federal standards are not being properly applied. "While the evidence of the record reveals that Mrs. X is currently exceptionally knowledged of the business, it is of the utmost importance that we look at the original shareholders and number of board stock initially issued by the board members" (*5*). This case involves a company that was formed nearly 27 years ago. This company is controlled and run totally by the female owner, who now owns 100 percent of the stock. However, there was a transfer of stock more than 10 years ago, before the DBE program existed. This WBE company had been certified since 1980 and recertified under the present regulations consecutively for 10 years. To deny recertification on the basis of an occurrence before the inclusion of women in the DBE program is a travesty of justice. This situation would seldom apply to male minorities because the transfer of stock would be from father to son, from father to daughter, from son to daughter, etc.

Another example of Jane Crow interpretations is the requirement that those applying for certification and recertification in the DBE program must present their personal income tax returns. There is absolutely no reason to request personal income tax returns, other than to suggest that the female owner would be hiding something because she is married to a nonminority male. However, to require minority males to present a personal income tax return would not apply because the chances are that his personal income tax return is a joint return with a minority female.

DOT INTERPRETATION OF CONTROL AND MANAGEMENT OF WBEs

DBE regulations require that the owner possess the power to direct or cause the direction for the day-by-day operations, as well as the major decisions on matters of management, policy, and operations. DOT's Office of Civil Rights has consistently upheld decisions to decertify WBEs on the basis of the fact that a nonminority party has experience and expertise in the construction industry.

DOT officials have twisted the regulations requiring real and substantial company control of a WBE firm to mean that the female owner should possess technical expertise. When Great Distributing Company, Inc., appealed its decertification in 1988, DOT upheld the state's decision, offering the following explanation. "The record in this case reveals that the technical expertise of Mr. Johnson is far superior to that of the female owner. This is not to say that the contributions of Mrs. Johnson are not important to the success of this business. The Regulation, however, requires more than an important contribution" (*5*).

In another instance, DOT takes their erroneous presumptions a step further: "The Department is aware that, theoretically, the female owner could replace her two sons on the Board, in the event of disagreement, through her majority position; however, this seems unlikely in view of the family relationship and her sons' superior technical expertise" (*5*). Further, in a letter to another WBE, "The Department recognizes that as a majority owner, the female owner could, at least theoretically, remove Mr. Hall from the Board. However, it appears unlikely considering the husband's extensive involvement in the firm" (*5*).

DOT's Office of Civil Rights has consistently held that small family-run businesses (a business owned by a Caucasian female who employs her husband, sons, or daughters) do not qualify for the DBE program. DOT alleges that this indicates the female owner does not control her company. However, this requirement is not contained in the federal regulation, but has been assumed by DOT. DOT's July 1989 response to the W. R. Mollohan, Inc., appeal to their decertification was, "The Regulation does not provide for the inclusion of family-run businesses where the background and technical expertise of the husband is clearly superior to that of the female owner and when the husband is also responsible for the critical activities of the business" (*5*). DOT also denied recertification for H & H Landscape Company in May of 1988, saying, "H & H is in reality a family run business with mother and son sharing management and control but with the son possessing the critical skills necessary to control day-to-day operational decisions. This type of business is not eligible to participate in the departments DBE, WBE program" (*5*).

Another discriminatory and devastating interpretation of DBE regulations is the assumption that a woman's involvement in her company is limited to administrative or clerical roles because she may not be in the field 5 days per week. No president of any corporation in the nation does all company jobs. If the woman-owner has a male working for her as a field manager, supervisor, or estimator, then she is going to be considered as not controlling her business.

It is inconceivable that DOT's Office of Civil Rights would take the position that any owner of a business must handle every aspect of that business. This view would represent an unsound business practice. Logic dictates that when a woman owner hires someone to perform a supervisory role, she will hire the most qualified person, with the most experience in each particular area. In the construction industry, this person will generally be a male who has more years of experience in construction than the female owner. This action does not mean the female owner is not in compliance with federal regulations. A more reasonable interpretation would be that the female owner must have an understanding and knowledge of her business, to the extent that she can evaluate the information supplied to her and from that information independently make decisions regarding day-by-day operations.

If DOT's interpretation is carried to an extreme, half the small businesses in the country would have to close if their owners were no longer permitted to hire individuals whose expertise outweighed their own. Apparently, WBEs are the only businesses in our society harassed for exercising good managerial skills and hiring an expert crew.

EXPERIENCE AND EXPERTISE OF WBE OWNERS

One of the main reasons WBEs are either decertified or not certified is the presumption that a particular female business owner does not possess enough expertise. This practice is merely a ploy to keep women from participating in the highway construction industry. The question must be asked, "How can a woman acquire the technical experience and expertise if she has been barred from participating in the industry in the first place?" Congress included women as socially and economically disadvantaged, for the purpose of this program, because they are convinced of the discrimination women face in this male-dominated industry. Construction is a male-dominated profession no matter what the race, creed, or color. Males have always had the opportunity to participate in this industry. Women, on the other hand, have been forced to participate in traditional roles and have encountered road blocks trying to break out of these roles.

The issue of experience and expertise is not addressed in DBE regulations. However, interpretation of the regulations by DOT's Office of Civil Rights suggests that minority or women owners must have the experience and expertise necessary to possess the power to direct or cause the direction of management and policy of the firm and to make day-by-day as well as major decisions.

DOT has constantly asserted that women do not have the experience and technical expertise necessary to run a construction-related business (*5*). Of the 153 appeals before DOT in 1989, 107 were women who were all told that they did not have the experience and expertise to conduct day-by-day business (*5*).

If this type of erroneous interpretations of the regulations is permitted to continue, women will be excluded from participating in the DBE program. One of the main reasons women were included was to allow 52 percent of this country's population access to nontraditional occupations. Common sense dictates that, with some exceptions, women do not have the 10, 15, or 40 years' experience in the construction industry. Women have been precluded from participating in the industry, in nontraditional roles, because of discrimination.

The critical point is that the regulations do not require WBE owners to have more experience than their employees. The federal regulations do not place any sanctions on hiring an employee with superior expertise. A female owner does not have to possess expertise superior to those she hires to be eligible to participate in the DBE program.

However, rarely does the experience and expertise issue apply to minority males. Generally, minority males hire minority males. Therefore, no matter who is controlling the business, a minority male is in the forefront. Because the highway construction industry has always been a white, male-dominated industry, minorities who have had an opportunity to participate in the DBE program have had the tendency to be in labor-related areas rather than management.

In this regard, the U.S. Department of Labor is investigating discrimination of women in the highway construction industry. In many cases, women are discriminated against and are not accepted to participate in union-related training for iron workers, carpenters, electrical workers, etc.

With the creation of the DBE program, many minority individuals instantly had the opportunity to run and control their own business. Women, on the other hand, who had been in the highway construction industry in whatever capacity saw an opportunity also to create their own companies. However, the only available pool of construction supervisors for WBEs is the white male. Therefore, to require that a woman have another female supervisor is ludicrous given past discrimination. However, if she hires white males to participate in her company and to help her manage and supervise, then she is considered not to be controlling her business. This is a slap in the face to women.

Women were added to this program because it was difficult for women to enter nontraditional roles, such as owning a construction company. Because women were not permitted to have nontraditional construction jobs does not mean that a woman cannot possess the requisite knowledge and power to own and control a construction company within the meaning of the federal regulations. However, a woman who is to be in the position of hiring a family member, regardless of their experience, is precluded from participating in this program by subjective interpretations of DOT's Office of Civil Rights.

In the two cases cited previously, both women were able to prove clearly that they owned their own business, capitalized with independently owned funds, with sole ownership of all stock, and had been in the industry 8 to 10 years.

Minorities are not faced with this problem, because generally, the relatives they employ are also minorities. Apparently, decertification based on employing relatives only applies to white women.

APPEALING DOT CERTIFICATION DECISIONS

DBE regulations specifically allow for a series of appeals for companies that do not agree with an initial decision of a state not to certify. However, the appeal process on the state level is conducted by the same division that refused the certification in the first place. For companies believing in our system of justice, there is a final appeal to DOT's Office of Civil Rights. But, the Office of Civil Rights is nothing more than a rubber stamp for the states. A fair appeal system does not exist on the federal level for American citizens to trust. For example, for all of 1987 and one-half of 1988, appeals requested from DOT amounted to a total of 339—170 white females, 67 minority males, 1 white male, and 1 black female. Of the 339, only 5 were reversals of state decisions. All of the white females' appeals had almost the same reasons for denial. The 67 minority denials had a mixture of reasons. Twenty-six minority males were native Indian and Hispanic who were denied certification because of a lack of recognition in their community. The 41 black males were denied certification because they were overly dependent on nonminority contracting firms. However, few minorities lost their appeals on the basis of technical expertise, or control and management of their businesses, or how they obtained their capital, or if their capital and expertise was not real and substantial, as were most denied appeals for WBEs. Apparently a double standard exists with DOT's Office of Civil Rights.

WBEs appear to be the only segment of American society not afforded the opportunity to have an unbiased, third-party review of the circumstances surrounding their expulsion and determine whether justice has been served. Granted, DBE regulations do not mandate such procedure, but they do strongly urge due process.

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Disadvantaged Business Enterprise Program Management Through Project Goal Setting

WILLIAM R. BROWN

The Surface Transportation and Assistance Act of 1982 (STAA 1982) mandated that 10 percent of all FHWA funds be expended with disadvantaged business enterprises (DBEs). STAA 1982 also required state highway authorities to establish an annual state DBE goal and to design and develop a fair, workable program to achieve this goal. In order to ensure executive review, STAA 1982 further stated that any request for a state goal less than 10 percent had to be forwarded under the governor's signature. The task of developing and implementing this new federal program was delegated to the Indiana Department of Transportation (INDOT). The first challenge for INDOT's DBE program was to figure out a method of achieving the 10 percent DBE goal in a state that is primarily rural with a minority population of less than 10 percent. The method conceived and developed was to establish a central office committee to individually review all federal-aid contracts for possible DBE participation. This Project Goal Setting Committee would set goals on individual contracts, thereby managing projected DBE participation and ensuring attainment of the state's annual 10 percent DBE goal.

When Congress passed the Surface Transportation and Assistance Act of 1982 (STAA 1982), Indiana had a 3 percent federal minority business enterprise (MBE) goal in place. An additional 5 percent state voluntary MBE goal also existed. STAA 1982 mandated that 10 percent of all FHWA funds be awarded to disadvantaged business enterprises (DBE). This legislation also established more stringent criteria for the certification of DBEs, designation of funds to be counted, and application of funds toward achieving the 10 percent goal. New procedures would be required in the areas of certification, prequalification, and compliance. Undoubtedly, the most important of these new procedures would be developing a method to ensure that 10 percent of FHWA funds were awarded to DBEs using a method that would be reasonably fair to all involved. The Indiana Department of Transportation (INDOT) was charged with establishing a program to comply with this new regulation.

In the subsequent development of its DBE program, INDOT discovered a major problem—attaining a 10 percent DBE goal in a state that is primarily rural with a minority population of less than 10 percent. In addition, the majority of the minority population in Indiana is located in three major urban centers in the northern one-half of the state (Indianapolis, Fort Wayne, and the southeast Chicago corridor—Gary, East Chicago, Whiting, and Hammond). Further investigation also determined that the Indiana state constitution prohibited setting aside contracts to create a protected environment for certain bidders. Faced with this dilemma, INDOT considered numerous scenarios that included everything from asking the governor to request a goal of less than 10 percent from FHWA to attempting to change the state constitution. A decision was made to attempt to achieve the mandated 10 percent DBE goal using a process called "DBE project goal setting."

State Office Building, Room 1313, 100 North Senate Avenue, Indianapolis, Ind. 46204.

WHAT IS PROJECT GOAL SETTING?

INDOT's version of DBE project goal setting is best described as individually reviewing every contract or project containing federal funds to determine if contracts have sufficient subcontractable items that can be used to set a DBE goal. These individual contract goals are the nucleus for the attainment of an annual state DBE goal.

HOW IS PROJECT GOAL SETTING USED TO ATTAIN ANNUAL STATE DBE GOALS?

This question can best be answered by providing the goal-setting methodology used by INDOT. In Indiana, project goal setting is accomplished by the use of a DBE goal-setting committee, which is composed of

- Chief, division of engineering services;
- Field engineer, division of operations support;
- Equal Employment Opportunity manager;
- DBE coordinator; and
- FHWA representative, who is a nonvoting member.

Additionally, the deputy commissioner of highway development periodically attends meetings to monitor the activities of the committee.

The goal-setting committee meets to set contract goals 2 months before the contract letting (selling) date. Additionally, contracts scheduled for the next letting are reviewed at this time to determine if any changes have been made in the contract that would affect the previously established goal.

A few days before the date of the goal-setting meeting, the EEO section is provided with a rough estimate (not the engineer's estimate) of each project that involves federal funds. The DBE coordinator completes an initial review and determines a recommended goal.

Before convening the goal-setting meeting, each member is provided with a map of the state indicating counties and highway districts. Each member is also provided with a DBE directory listing all certified DBE firms, the type of work these firms desire to do, and areas of the state that they desire to work in. INDOT's prequalification section furnishes the committee with an up-to-date listing of all prequalified DBEs, the amount they are prequalified for, and specialty and subspecialty work areas. This listing of prequalified DBE firms is required because Indiana requires prequalification of all subcontractors for subcontracts over $100,000.

When the goal-setting meeting is convened, each contract or project for the applicable letting is reviewed to determine if a portion of the contract can possibly be subcontracted to DBEs. Typically, numerous contact line items are taken into consideration for DBE participation, including the following.

Bridge cleaning and painting	Pipes
Catch basins	Reinforcing steel
Common excavation	Removal items
Concrete structural members	Rip-rap
Construction engineering	Seed and sod
Fence	Signs and barricades
Guardrail	Structural steel
Hauling	Temporary and permanent pavement markings
Manholes	
Material suppliers	Underdrains
Miscellaneous concrete	

This list of items considered by the committee is definitely not all-inclusive and prime contractors themselves are continuously adding new items and methods of achieving the DBE goal.

The fact that any of these contract line items that INDOT considers subcontractable appear in a contract is insufficient, by itself, to set a goal for the project. Other factors of equal importance must also be taken into consideration before setting a DBE goal for any contract. (Any item considered must be a typically subcontractable item, e.g., concrete ramps would not be considered subcontractable in a concrete paving contract nor would removal items in a demolition contract.)

1. The location of the contract. Is it urban or rural? Is it in an area that has a concentration of DBEs or relatively few DBEs?
2. The approximate size of the contract or possible subcontract. Would it be profitable enough for DBEs to travel to where the contract is located?
3. Are there any DBEs qualified for the specific job or specialty?
4. Are there any DBEs qualified for the job who have expressed a desire to work in the area of the contract?
5. If this item were subcontracted, would it constitute a commercially useful function?
6. If the job is over $100,000, are there any DBEs prequalified in that specialty or subspecialty area?

In addition, INDOT takes into consideration what is referred to as the two-by-two rule. In this respect,

1. At least two ways must exist to make the contract goal or no goal will be set,
2. At least two DBEs must be working in the area where the contract or project is located in order for any item to be considered,
3. At least two DBEs must have expressed a desire to work in the area where the contract or project is located, and
4. At least two of these DBEs must be prequalified if the subcontractable item is estimated over $100,000.

The main reasons for using this two-by-two rule is to ensure availability of competitive bidding for these contract items and to prevent any person (or the committee) from having the authority or ability to set aside any contract or subcontract for any specific firm.

While the DBE goal-setting committee reviews each contract or project, any objections or comments concerning a goal are discussed until a unanimous decision is made on establishing the contract goal. The committee normally sets goals between zero and 15 percent. Since 1984, INDOT has set DBE contract goals in excess of 15 percent on only two occasions (one 18 and one 20 percent).

IS PROJECT GOAL SETTING A FAIR METHOD FOR OBTAINING ANNUAL DBE GOALS?

Questions of fairness in the DBE program receive many comments, regardless of how the annual DBE goal is set or attained. Prime contractors invariably comment on the size of the goal and majority specialty subcontractors comment on which contract line items are considered. However, INDOT does feel that project goal setting is a reasonably fair method for setting and attaining DBE goals both for majority and DBE contractors and subcontractors because

1. All goal setting meetings are open to the public,
2. Each contract or project is reviewed separately on its own merit,
3. All subcontractable items are considered,
4. INDOT uses the two-by-two rule, and
5. Contract goals in excess of 15 percent are seldom set.

DOES PROJECT GOAL SETTING WORK?

Since beginning in 1982, INDOT's DBE program has undergone many changes. However, the methodology for setting DBE contract goals has been virtually unchanged since its inception, but INDOT has exceeded its annual goals each year (Table 1). INDOT has averaged between 110 and 135 certified DBEs since 1982. A few of these firms are small prime contractors who bid on and periodically receive contracts. Also, INDOT requires that good-faith efforts to use DBEs be taken on any item sublet on consultant contracts.

TABLE 1 INDOT FEDERAL FISCAL YEAR DBE GOAL ATTAINMENT (DOLLARS IN THOUSANDS)

YEAR	FEDERAL FUNDS	FEDERAL DOLLARS AWARDED TO DBEs	PERCENT
1983	$270,484	$17,041	8.0% (Partial Program Year)
1984	220,768	26,394	12.00%
1985	248,126	25,253	10.20%
1986	251,531	27,370	10.91%
1987	187,143	21,297	11.40%
1988	214,280	24,027	11.20%
1989	240,483	26,840	11.80%

These two factors have helped INDOT achieve the annual state goal of 10 percent DBE participation. However, the only enforceable vehicle for goal attainment during this period has been project goal setting, which has been the largest contributing factor to goal attainment.

COMMITMENT, SUPPORT, AND COOPERATION

Project goal setting, by itself, is not a method to achieve annual DBE goals. No program of this nature can possibly work without the commitment, support, and cooperation of all involved. In this respect, DBEs, majority contractors, and contractor associations have periodically monitored INDOT goal setting meetings. Further, the Indiana General Assembly passed a law that knowingly misrepresenting a firm as a DBE is a Class D felony and INDOT enacted a rule allowing the suspension of a contractor from working on highway contracts for up to 2 years for DBE program violations. Although these enforcement and regulatory actions are required, the real commitment, support, and cooperation emanates from the positive side. Indiana's governor and INDOT's commissioner publicly support and affirm their commitment to the program and to the increased participation of minorities and females in highway contracts. In addition, prime contractors support the program by achieving the goals set by the DBE goal-setting committee. Contractors also support the program by active participation in DBE training during the off-construction season. Last, DBEs themselves are the biggest supporters of project goal setting because without competent DBE firms to accomplish the contract items used in setting the goals, no goal program could work.

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