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Contents

Foreword

The papers in this volume report information on fast-track paving, statistically based specifications, bridge rehabilitation, contracts management, and automation in hot in-place asphalt pavement recycling. They should be of interest to state and local engineers of materials, construction, maintenance, and bridge design, as well as contractors and material producers.

Hossain and Wojakowski discuss the construction and performance of a fast-track concrete pavement in Kansas. They report that the performance of this pavement is excellent 3 years after construction.

Weed presents the use of composite pay equations as a means of eliminating the confusion and difficulty in administering the acceptance testing of various quality characteristics. He uses an example based on portland cement concrete pavement to illustrate the practicality of this method.

Taavoni describes the upgrading and recycling of an 1879 wrought-iron pin connected truss bridge by replacing the pins.

Abdul-Malak and Abou-Assaly investigate innovative concepts of contract management to improve the execution of contracts in Lebanon. They present an integrated contract management approach directed toward understanding the interactions, interrelationships, and interdependencies that exist among candidate concepts and strategies.

Pagdadis and Ishai discuss the use of automated equipment used for hot in-place asphalt resurfacing and the breakthroughs that have been achieved toward improving this particular site process through real-time data handling.

Construction and Performance of a Fast-Track Concrete Pavement in Kansas

MUSTAQUE HOSSAIN AND JOHN WOJAKOWSKI

The fast-track or high early strength concrete offers the opportunity of taking advantage of higher early strength gain in a smaller time for construction or rehabilitation of high volume roads and city streets serving businesses. The Kansas Department of Transportation and the city of Manhattan, Kansas, built a section of fast-track concrete pavement in an urban setting in Manhattan, Kansas, in 1989 and 1990. The mixture design was developed using a special Type-III cement and three different types of locally available aggregates. Strength gain of this mix in the field was satisfactory except on a few occasions when the daily low temperature dropped below −0°C (32°F). Two mixes with different water-cement ratios performed equally in terms of strength gain. The maturity data collected in the slabs and field beams indicate that the maturity of companion field beams lagged that of the slab bottom or top. However, the maturity number was well correlated with the 24-hr flexural strength of the beams when the beam strengths were corrected for temperature of testing. The field beams appeared to mature earlier than the laboratory beams and thus showed higher strengths at the same age. Multiple surveys of this fast-track pavement during the past few years did not reveal any major distress. Visual survey conducted in 1993 indicates that the longitudinal surface texture of the pavement is showing wear. This might happen because of the grinding action of the sand particles on the pavement surface applied during the winter months under the traffic load. Overall, the performance of this pavement is excellent.

The time window available for construction or rehabilitation of high volume roads and city streets serving businesses is an important consideration. The fast-track or high early strength concrete offers the opportunity of taking advantage of higher early strength gain in a smaller time window for pavement construction or rehabilitation. Thus, the main objective of using this concrete is to increase the rate of strength gain of concrete pavements without any reduction in the ultimate concrete strength and durability, so that the pavement can be opened to traffic in a matter of hours or days.

The Kansas Department of Transportation (KDOT) and the City of Manhattan, Kansas, built a section of fast-track concrete pavement in Manhattan, Kansas, in 1989 and 1990. The project was closely monitored and some research quality data on the concrete maturity (i.e., indication of strength gain) and strength were gathered. The project was surveyed in 1991 and 1993. The design, construction, and performance of this project to date is discussed.

PROJECT DESCRIPTION

Location and Layout

The fast-track pavement, part of the KDOT project 81 U-1122-01, is in Manhattan, Kansas, about 90 km (56 mi) northwest of Topeka. The project consisted of grading, widening, surfacing, and bridge construction from Station 54 + 00 to Station 239 + 038 on Anderson Avenue, a major arterial connecting the suburban west side with the old east side of the town. The fast-track pavement extends from Station 49 + 00 to Station 179 + 00 and includes three intersections and fifteen driveways serving various businesses between Seth Childs Road and Wreath Avenue (Figure 1). The roadway is a four-lane undivided facility with a dual left-turn lane in the middle. In this paper "the project" means only the fast-track part of the KDOT project 81 U-1122-01.

M. Hossain, Department of Civil Engineering, Kansas State University, Manhattan, Kan. 66506. J. Wojakowski, Kansas Department of Transportation, Materials and Research Center, 2300 Van Buren, Topeka, Kan. 66611.

Soils, Climate, and Drainage

The soils in the project area are mostly silty clay (Unified CL or AASHTO A-6) with low shrink-swell potential. Approximately 70 to 90 percent pass U.S. number 200 sieve. The climate in this region is moderately extreme with hot, humid summers (temperatures exceeding 35°C [100°F]) and cold (below −20°C [0°F]) wet winters. Figure 2 shows the monthly low and high temperatures in the project area since 1989. The 30-year annual precipitation is around 91 mm (32 in.). The average number of frost-free days is 176. The referenced meteorological data were recorded at the National Weather Services station in Manhattan. Drainage on the project was provided by the side slopes, curb and gutter, and intermittent inlets to the storm sewers.

Traffic History

The project was designed for 1986 ADT of 16,300. The projected ADT for 2006 is 36,430. The percentage of trucks was 5 percent in 1986 and currently consists of some heavy delivery vehicles and tankers serving the adjoining supermarket, automobile dealership, and gas stations. This mix of traffic led to the city's decision to use the fast-track concrete on this project so that vehicular access would not be denied to any business for more than 48 hr.

Pavement Section

The mainline pavement section consists of 229 mm (9 in.) of plain concrete (PCCP) placed directly over the compacted subgrade. The business entrances and turnaround areas are 178 mm (7 in.) thick. The pavement is a jointed PCCP with joints at a uniform spacing of 4.6 m (15 ft).

DESIGN DESCRIPTION

Material Characteristics

The mix design and materials were specified by the KDOT *Standard Specifications for State Road and Bridge Construction*, 1980 and 1990 editions, and *special provisions* for the project (*1*).

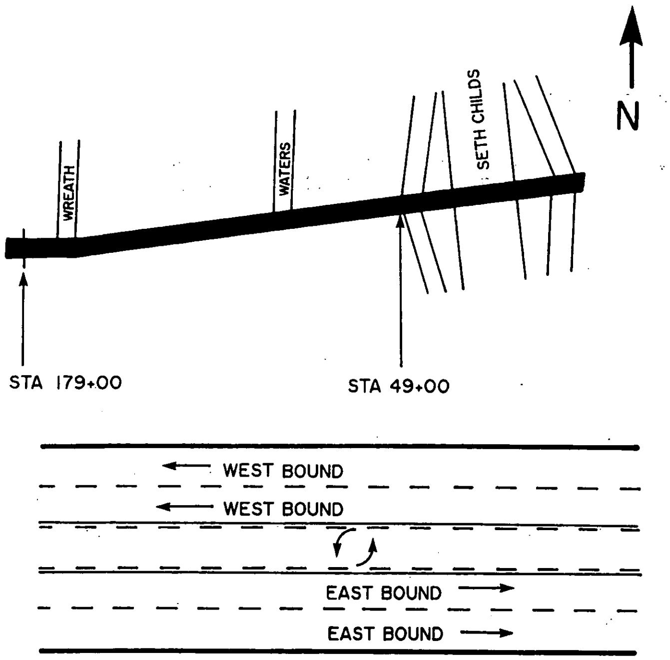

FIGURE 1 Location and layout of fast-track test section.

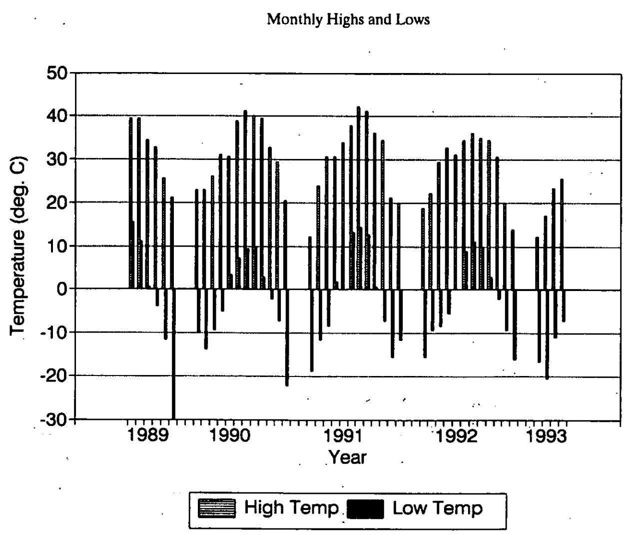

FIGURE 2 Monthly high and low temperatures for 1989, 1990, 1991, 1992, and 1993.

Cement

A special Type III portland cement was specified for this project to meet the required minimum compressive strength of 8,966 kPa (1,300 psi) at 12 hr when 51-mm^3 (2-in.^{3}) mortar specimens were tested according to the ASTM C109 (*2*). However, this cement also met the 1980 KDOT standard specifications for Type-III cement. The general properties of the cement used on this project were

- Initial setting time (Gilmore): 145 min,
- Final setting time (Gilmore): 285 min,
- Autoclave expansion: 0.100 percent,
- Blaine fineness: 560 m^2/kg.

Chemical composition of this cement is shown in the following table:

Constituents	*Percent Present*
Silicon Oxide	20.34
Aluminum Oxide	4.09
Ferric Oxide	3.23
Calcium Oxide	62.79
Magnesium Oxide	3.72
Sulfur Trioxide	3.37
Sodium Oxide	0.16
Potassium Oxide	0.48
Loss on ignition	1.71
TriCalcium Silicate	59.3
TriCalcium Aluminate	5.4

Average compressive strength of 51-mm (2-in.) standard mortar cube specimens tested according to the ASTM C106 were 3,517, 21,586, and 32,828 kPa (510, 3130, and 4760 psi) for 8 hr, 1 day, and 3 days, respectively.

Aggregates

Three different types of locally available aggregates, coarse (crushed limestone), intermediate (river gravel), and fine (river sand), were used in the design of the mix. The coarse aggregate was a KDOT Class-1 durable (freeze-thaw resistant) aggregate. The required gradations for these aggregates are shown in Table 1. The proportions specified in the concrete mixture were as follows:

- Coarse aggregate: 43 to 52 percent,
- Intermediate aggregate: 20 to 29 percent,
- Fine aggregate: 25 to 31 percent.

Trial Concrete Mix Design

Trial concrete mixture design was submitted by the cement supplier, Ash Grove Cement Company of Overland Park, Kansas. This mix design was evaluated by the Shilstone method (*3*). Table 2 shows the mix design information. Both water-reducing and air-entraining admixtures were used, and the resulting mix had about 5 percent air content.

Cylinder and beam specimens were prepared with this mix and cured under a thermal blanket. The compressive strengths were 25,172 and 27,517 kPa (3650 and 3990 psi) at 18 and 24 hr, respectively. The corresponding flexural strength values were 2,897 and 3,759 kPa (420 and 545 psi), respectively. Ultimately, all specifications for the materials and mix were developed.

The following proportioning in mix design was specified:

- kg of cement per m^3, minimum 386 (710);
- kg of water per kg of cement, maximum, 0.44;
- percent of air by volume, 6 $\pm$ 2 [as determined by Kansas test method KT-19 (Rollameter)].

The mix was required to meet the following minimum strength requirements when specimens made with the mix were cured 18 hr under a specified insulating blanket (with an R value of at least 0.5):

TABLE 1 Specified Gradations for Aggregates for Fast-Track Concrete

Aggregate	% Retained on sieve size									
	25 mm	19 mm	13 mm	6mm	#4	#8	#16	#30	#50	#100
Coarse aggregate (CA)	0	0-6	26-38	60-72	91-95	96-100				
Intermediate aggregate (SG)			0	0-5	10-20	42-70	75-98	91-100	96-100	97-100
Fine aggregate (FA)				0	0-6	9-17	28-38	55-65	86-96	94-100

Note: 1 in = 25.4 mm

TABLE 2 Trial Mix Design Information

Mix component	weight per sack (saturated, surface dry)
Cement, Type-III, kg	42.68
FA, Kaw River Sand, kg	42.25
SG, Blue River, kg	38.39
CA, Limestone, kg	86.94
Water, kg	17.07
Air Entrainment, %	5.0
Hycol, Water reducer, ml/m³	789
Daravair, air entrainer, ml/m³	371
Water-cement ratio (kg/kg)	0.40
Slump, mm.	63.5
Concrete unit weight, kg/m³	2238

Note: 1 in = 25.4 mm
1 lb = 0.454 kg
1 cu. yd. = 0.765 cu. m.
1 oz = 28.35 g

Test	*kPa (psi)*	*Test Method*
Compressive strength	20,670 (3,000)	KT-22 & AASHTO-T22
Flexural strength	2,412 (350)*	KT-231

**as determined from third point loading.*

PROJECT CONSTRUCTION

Concrete Mix

Two transit mixes with design water-cement ratios of 0.40 and 0.44 were used on this job. Detailed properties of these mixes are shown in Table 3. A water-reducing admixture meeting the ASTM standard C494 and an air-entraining admixture were used. The mix with 0.44 water-cement ratio had higher air content and slump than the mix with 0.40 water-cement ratio. This mix was used on most of the project.

Layout

Concrete was placed in the mainline pavement by using a slip-form paver. However, the turn-outs and other places were poured manually using fixed forms. In 1989, paving was done in July, October, November, and December (2 days only). In 1990, paving continued from April through October. Time of placement varied from as early as 6:00 a.m. to as late as 8:00 p.m.

TABLE 3 As-Batched Mix Design Information

Ingredient	Mix with 0.40 water-cement ratio	Mix with 0.44 water-cement ratio
	weight per sack (SSD)	weight per sack (SSD)
Cement, Type-III, kg	42.68	42.68
FA, Kaw River Sand, kg	41.39	40.32
SG, Blue River, kg	38.08	37.08
CA, Limestone, kg	86.10	83.84
Water, kg	17.07	18.73
Concrete unit wt., kg/m³	2353	2290
Ent. Air, %, (Rollameter)	3.5	5.6
Hycol, Water reducer, ml/m³	1052	1052
Daravair, air entrainer, ml/m³	148	148
Water-cement ratio (kg/kg)	0.40	0.44
Slump, mm	38.1	76.2

Note: 1 in = 25.4 mm
1 lb = 0.454 kg
1 cu. yd. = 0.765 cu. m.
1 oz = 28.35 g

Consolidation

The pavement was required to be consolidated so that the density of the concrete should not be less than 98 percent of the rodded unit weight. Density was measured by a nuclear gauge. The 98 percent density requirement was waived on miscellaneous areas such as entrance pavement and so forth.

Texturing

Texturing with the metal comb was not required on this project. Final finish was applied by a longitudinal burlap-drag.

Curing

A white curing compound and insulated blankets were used on this project. The insulating blanket cover intended for curing was specified to have a layer of closed cell polystyrene foam protected by at least one layer of plastic film with an R-value of at least 0.5. The thermal blanket was left on until the concrete had attained a minimum strength of 1,723 kPa (250 psi) modulus of rupture (third-point loading method). The curing compound was used during early morning hours and under cloud covers. However, the test results reported in this report for the maturity of the concrete in the slab were collected during curing with insulated blankets.

Sawing and Sealing

Primary saw-cuts of 3-mm (1/8 in.) width and full-joint depth were done as soon as the concrete hardened, and no ravelling or spalling due to the saw cuts occurred. These cuts were made satisfactorily throughout the project before any cracking occurred. However, on one slab, cracking occurred before the primary cut. In secondary cuts, the joints were widened to a width of 9 mm (3/8 in.) to form the sealant reservoir when concrete had attained at least 1,723 kPa (250 psi) modulus of rupture. The joints were cleaned by sandblasting, followed by compressed air blast. After installing the backer rod, a hot-poured, single component joint sealant manufactured by Koch Materials, Inc., and meeting ASTM D-1190 was used.

Opening to Traffic

The project was opened to traffic after the concrete had attained a minimum strength of 3,100 kPa (450 psi), and all joints were sealed. No business was denied vehicular access for more than 48 hr, and this fulfilled the city's objective of using fast-track concrete on the project.

STUDY OF FAST-TRACK CONCRETE

As mentioned earlier, some research quality data were gathered on this project to study the relationships among different concrete mix design parameters. The parameters studied include 24-hr flexural strength of concrete (from third-point beam test), concrete temperature (at the time of placement), concrete maturity, daily highest and lowest air temperatures, air content, and slump of the concrete mix. The following section describes the results of this analysis.

Relationship of Flexural Strength and Concrete Temperature, Daily Highest Air Temperature, Air Content, and Slump

Table 4 shows the summary statistics of 24-hr flexural strength, concrete temperature, daily highest air temperature, air content, and

TABLE 4 Summary of Statistics of Fast-Track Concrete Parameters

Water-Cement Ratio	Statistic	24-hr Flexural Strength (kPa)	Concrete Temp. (deg.C)	Daily High Temp. (deg. C)	Air Content (%)	Slump (mm)
0.40	Mean	90.19	25	25.89	4.9	40.64
	Std. Dev.	11.31	-12.78	-10.67	0.57	12.7
	C.V. (%)	13	12	16	12	762
	Range	77 - 103	17.78 - 32.78	16.11 - 36.11	4 - 5.5	25.4- 66
	Sample Size	7	7	7	7	7
0.44	Mean	91	25.33	29.78	5.5	55.88
	Std. Dev.	21.61	-14.22	-9.44	1.1	9.14
	C.V. (%)	24	8	18	21	17
	Range	42.63 - 115	19.44 - 30.56	13.89 - 41.67	4 - 9	38.1 - 66
	Sample Size	19	19	19	19	19

Note: 1 in = 25.4 mm
1 psi = 6.89 kPa
C.V. = coefficient of variation

slump of the concrete mix collected during construction of this project. The beams were cast in the field and cured under the curing blanket. The number of observations for concrete mixes with water cement ratios of 0.40 and 0.44 were 7 and 19, respectively. The variability of flexural strength for the mix with 0.44 water-cement ratio is very high. This happened because of low flexural strength attainment due to very low daily temperatures for 3 days in November and December 1989. Daily low temperatures of $-5°$, $-12°$, and $-6°C$ (23°, 11°, and 21°F) were recorded on these days, and the mix failed to reach the required flexural strength of 2,412 kPa (350 psi) in 18 hr.

Students *t*-tests were conducted between the means of 24-hr flexural strengths of field beams cast with these two mixes. The results did not show any significant difference in strength at 5 percent level of significance. There was somewhat higher variability of air content for the mix with 0.44 water-cement ratio.

To study the interrelationships among the parameters mentioned, a correlation analysis was done. Table 5 shows the correlation ratios among these parameters. The following observations can be made on the basis of the results of Table 5 and general trends for these parameters as related to the fast-track concrete:

- There are significant relationships between the 24-hr flexural strength and daily high temperature, concrete temperature, and air content. It can be generally observed that the 24-hr flexural strengths were higher when the daily high temperature exceeded 24°C (75°F) for both mixes (Figure 3). Figure 4 shows that the fast-track concrete mixture failed to achieve the desired flexural strength in 24 hr when the daily high temperature fell below 0°C (32°F). On the other hand, flexural strength was the highest when the concrete temperature was between 27° and 29°C (80 and 85°F), as shown in Figure 5.
- As can be expected, the flexural strength tended to decrease as the air content increased, as evidenced by the negative coefficient of correlation and trend in Figure 6. Air entrainment was also significantly affected by the daily high temperature.
- Air entrainment significantly increased slump of the fast-track concrete mix. However, the slump was not a direct factor in determining the strength gain of the concrete as long as it was between 25 and 76 mm (1 and 3 in.) (Figure 7).

TABLE 5 Correlation Among Fast-Track Concrete Mix Parameters

Water-Cement Ratio	Parameter	24-hr Flexural Strength	Concrete Temp.	Daily High Temp.	Air Content	Slump
0.40	24-hr Flexural Strength	1.0	0.19	0.48	-0.37	-0.33
	Concrete Temp.	0.19	1.0	0.79*	0.14	0.72*
	Daily High Temp.	0.48	0.79*	1.0	-0.20	0.55**
	Air Content	-0.37	0.14	-0.20	1.0	0.34
	Slump	-0.33	0.72*	0.55	0.34	1.0
0.44	24-hr Flexural Strength	1.0	0.46*	0.79*	-0.53*	-0.21
	Concrete Temp.	0.46*	1.0	0.69*	-0.33	-0.26
	Daily High Temp.	0.79*	0.69*	1.0	-0.41**	-0.26
	Air Content	-0.53*	-0.33	-0.41**	1.0	0.52*
	Slump	-0.21	-0.26	-0.14	0.52*	1.0

* significant at 5% level of significance.
** significant at 10% level of significance.

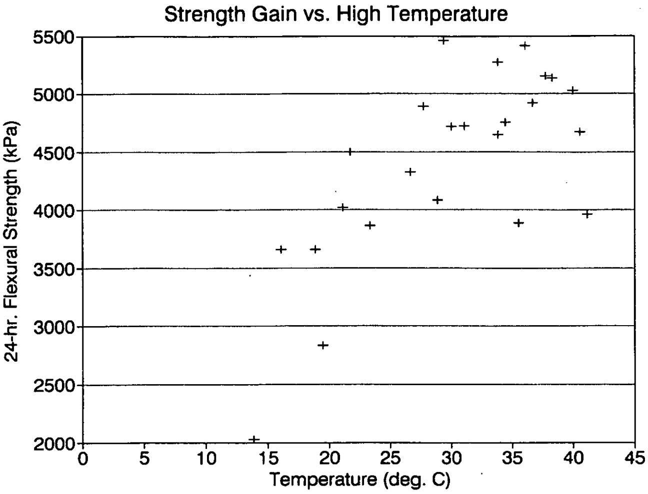

FIGURE 3 Strength gain of fast-track concrete versus daily high temperature.

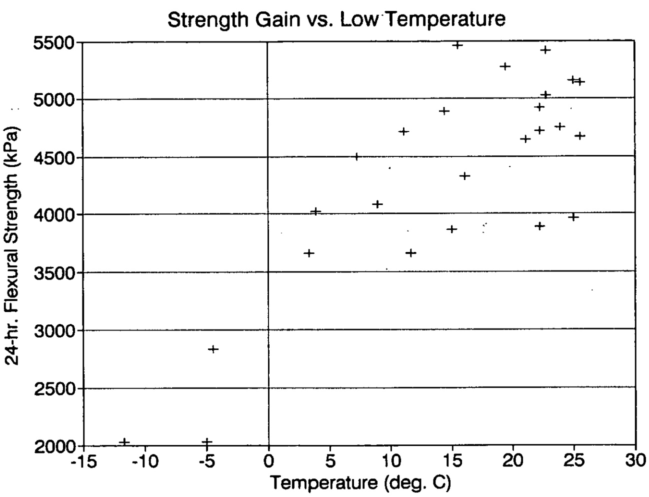

FIGURE 4 Strength gain of fast-track concrete versus daily low temperature.

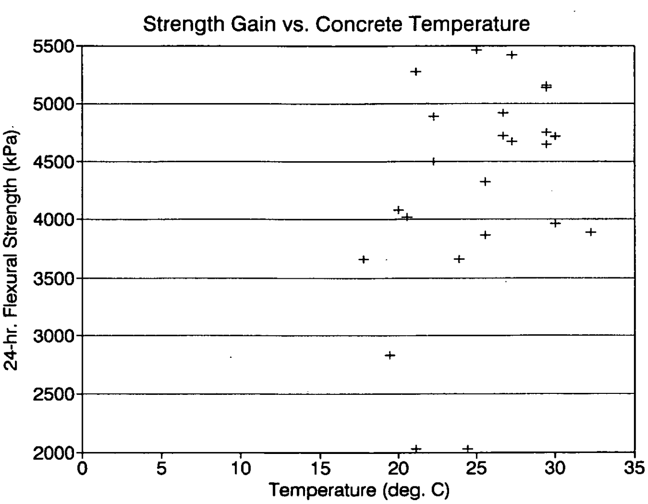

FIGURE 5 Strength gain of fast-track concrete versus concrete temperature.

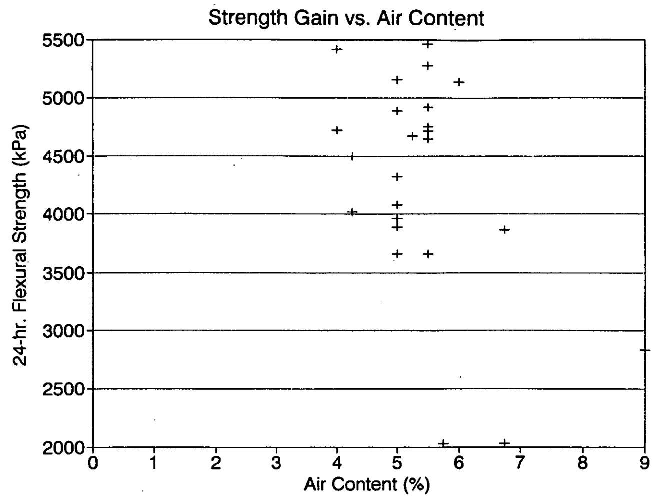

FIGURE 6 Strength gain of fast-track concrete versus air content.

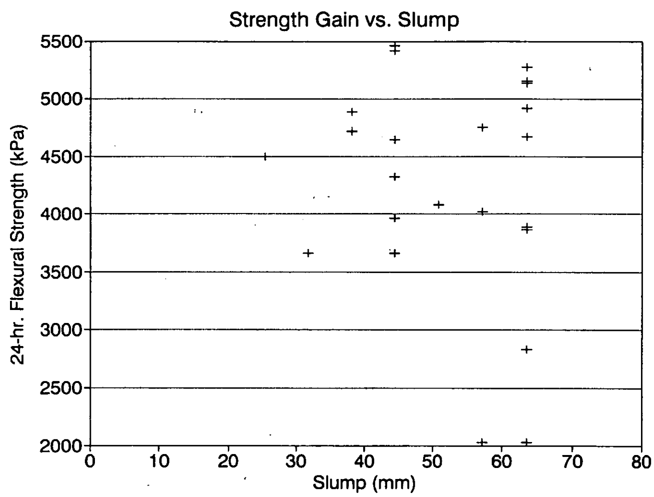

FIGURE 7 Strength gain of fast-track concrete versus slump.

Another general observation indicates that the time of day of concrete pour had little direct impact on the early strength gain other than affecting the concrete temperature slightly.

Fast-Track Concrete Maturity

The maturity of fast-track concrete was evaluated by collecting time and temperature data using a maturity meter manufactured by James Instruments. Thermocouple wires were inserted at the following locations in the slab: 13 mm (1/2 in.) from the top, center, and 13 mm (1/2 in.) from the bottom. Temperature was also monitored in the field beam where thermocouple wires were buried 76 mm (3 in.) into the beam around 152 mm (6 in.) from one end. Air temperature was also monitored. The slabs and beams were cured using blanket insulation during data collection. Maturity data were collected for 24 hr. Figure 8 shows the plot of temperature versus time for October 4 and 5, 1989. It is obvious that the maturity of a fast-track concrete slab can not be directly monitored by a companion beam.

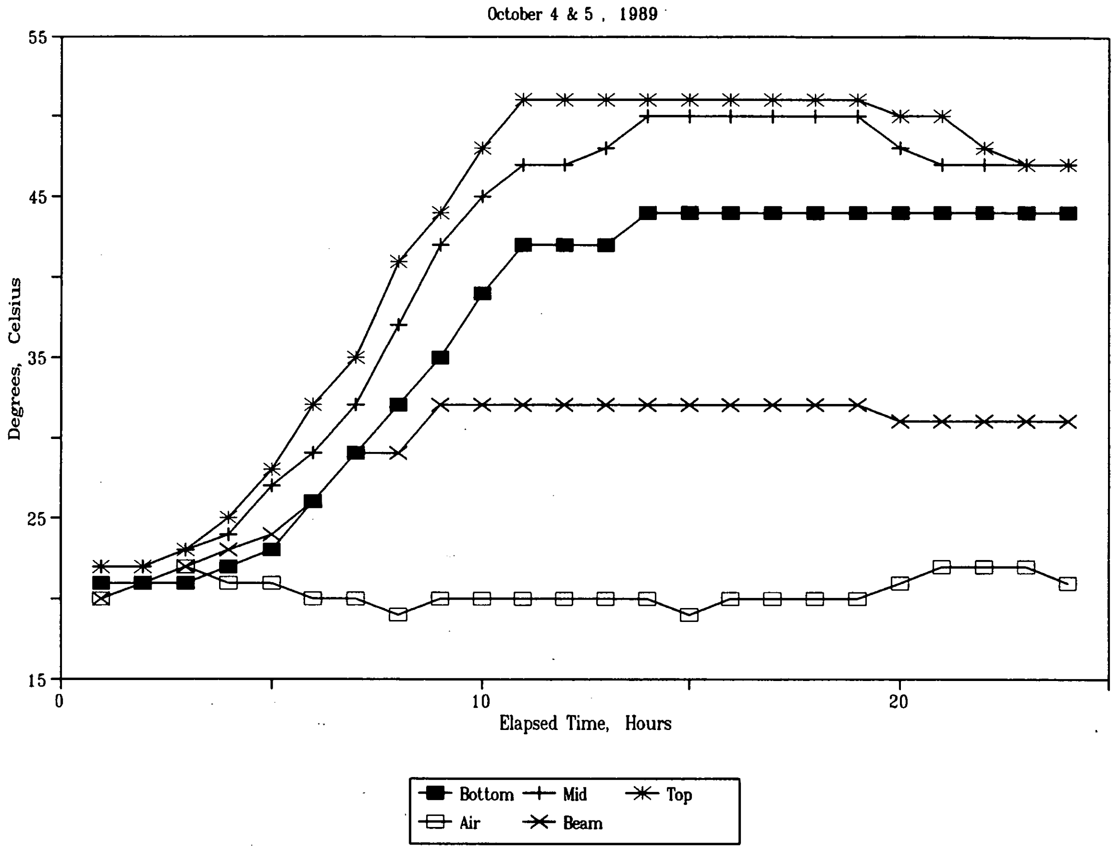

FIGURE 8 Typical time-temperature relationship for fast-track concrete.

This is expected because of the greatly different masses of concrete (and heat losses) of the slabs and the beams. Moreover, it is also the reason that the maturity concept is needed.

Correlation between Fast-Track Concrete Maturity and Strength

Figure 9 shows the maturity numbers measured in the slabs and field beams after 24 hr for the mix, with a water-cement ratio of 0.44 on three different days in 1989 and 1990. The maturity numbers of beams always lagged behind those of the slab top and bottom. The strength gain of fast-track concrete can be monitored by using the lesser of the maturity numbers at the top and bottom of the slab. To correlate the 24-hr flexural strength with the maturity number, data were collected for the mix with 0.44 water-cement ratio on 3 different days. The flexural strength data were corrected for test temperature using the correction factors suggested by Neville (*4*). Figure 10 shows the plot of these data. It is apparent that the 24-hr flexural strength has a linear relationship with the maturity number. From the extrapolation of these data, it was found that this fast-track mix had a maturity number of approximately 3,600 when it reached the 24-hr flexural strength of 3,100 kPa (450 psi).

PAVEMENT PERFORMANCE

The project was surveyed for visual distresses in April 1991 and July 1993. The 1991 survey revealed only one diagonal crack. No scaling was observed. Some areas adjacent to the joints had been ground. Some spalling was observed in a few joints. In 1993, the project was surveyed using the APWA PAVER methodology developed by the U.S. Army Corps of Engineers (*5*). A total of 120 slabs (approximately 3.7 m × 4.6 m (12 ft × 15 ft) size), which represents 30 percent of the project area, was randomly sampled and surveyed for 19 different distresses. Low severity joint seal damage was observed over the whole area. Popouts were recorded on approximately 9 percent of the slabs surveyed. Minor joint and corner spalling was also observed. No polished aggregates except in the ground areas were evident. But the longitudinal surface texture formed during construction appeared to be showing wear. This

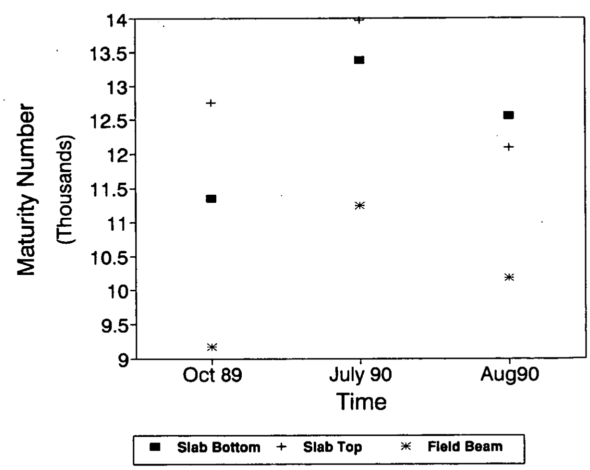

FIGURE 9 Maturity numbers for fast-track concrete beams and slabs after 24-hr curing.

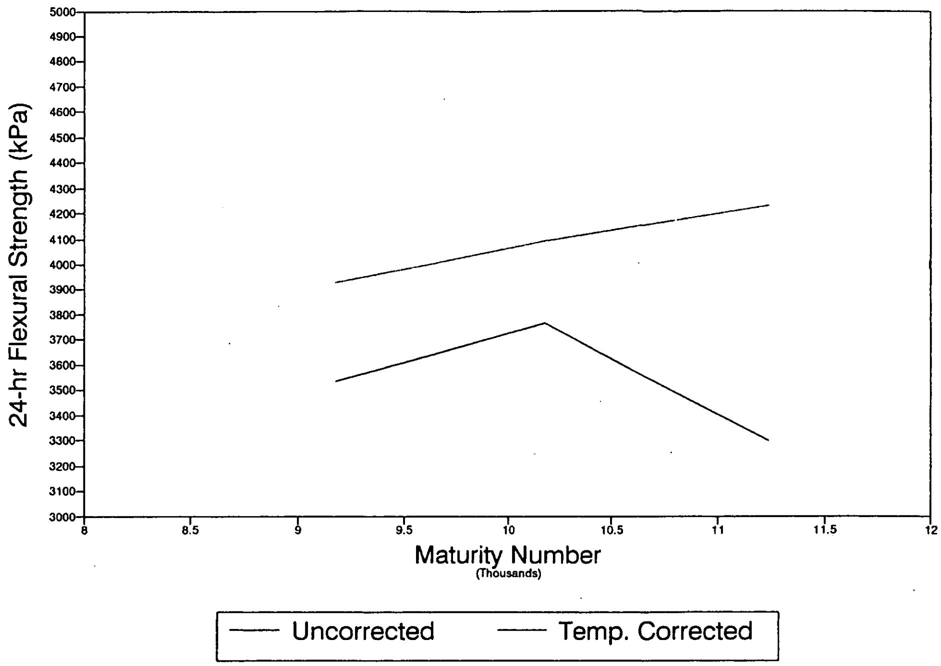

FIGURE 10 Relationship between maturity number and flexural strength of fast-track concrete beams after 24-hr curing.

might happen because of the grinding action of the sand particles on the pavement surface applied in winter under the traffic load. However, in view of the fact that the posted speed limit on this facility is 48 km/hr (30 mph), this condition should not create any friction-related problem. On the basis of the results of the PAVER survey, the pavement condition index of this pavement was calculated to be 96, which is a rating of "excellent."

CONCLUSIONS

The fast-track concrete pavement in Manhattan, Kansas, was successfully constructed using a high early strength or fast-track concrete mixture. No business in the surrounding area was denied vehicular access for more than 48 hr. The mixture was designed using a special Type-III cement and three different types of locally available aggregates. Strength gain of this mixture in the field was satisfactory except on a few days when the daily low temperature dropped below 0°C (32°F). Two mixes with different water-cement ratios performed equally in terms of strength gain. The maturity data collected in the slabs and field beams indicate that the maturity of companion field beams lagged that of the slab bottom or top. However, the maturity number has an excellent correlation with the 24-hr flexural strength of the beams when the beam strength values were corrected for temperature of testing. The field beams appeared to mature earlier than the laboratory beams and showed higher strength at the same age. Surveys of the fast-track pavements during the past few years did not reveal any major distresses. A survey conducted in 1993 indicates that the longitudinal surface texture of the pavement is showing wear. This might happen because of the grinding action of the sand particles on the pavement surface applied during winter under the traffic load. Overall, the performance of this project is excellent.

ACKNOWLEDGMENTS

The authors acknowledge the financial support provided by KDOT for this study. The authors also thank Dick McReynolds, KDOT, for his support of this study. The contribution made by Louis Funk of Kansas State University to this study is gratefully acknowledged.

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Composite Pay Equations: General Approach

RICHARD M. WEED

Highway construction specifications involving the acceptance testing of several different quality characteristics are sometimes confusing and difficult to administer. A procedure is developed by which multiple quality measures may be combined in a rational manner in a single, composite pay equation. This approach is scientifically sound and may be applied to almost any construction specification for which a relationship between quality and performance is known or can be approximated. An example based on portland cement concrete pavement is presented to illustrate the practicality of this method.

Highway agencies use many construction specifications that award adjusted payment appropriate for the level of quality received. These specifications often involve acceptance testing of multiple quality characteristics—such as, thickness, strength, and riding quality of pavement—that must be combined in some way to arrive at the overall pay factor for each construction item or lot of material. Typically, extensive specification language is required to describe exactly how the individual quality measures are to be combined. A method to simplify this process is sought.

It has long been recognized that uniformity of materials and construction is an important quality characteristic. Although uniformity is desirable, it probably is more important to ensure that very little of the construction item is of such low quality that more than routine maintenance will be required. This has led to the widespread use of the percent defective, representing the portion of the lot falling outside specification limits (or its counterpart, percent within limits) as the statistical measure of quality. This measure is well suited as an acceptance parameter because it encourages simultaneous control of both the process mean and uniformity.

The example that follows illustrates how the measures of percent defective for separate quality characteristics can be combined into a single pay equation to develop a process that is easy to understand and administer.

EXAMPLE

The New Jersey Department of Transportation (NJDOT) currently uses five measures of quality for portland cement concrete pavement: slump, air entrainment, thickness, compressive strength, and smoothness (riding quality).

Because it is possible to measure the slump and air entrainment of plastic concrete before it is placed, it has been the practice of NJDOT to accept or reject the concrete on the basis of these tests as it is delivered to the job site. Because the other three quality characteristics cannot be measured until after the concrete has been placed and cured, tests for these characteristics are typically completed a month or more after placement. In this case, the acceptance decision usually takes the form of a pay adjustment, depending upon the level of quality that has been achieved. One possible pay equation that combines these three measures for individual acceptance lots is the following:

$$PF = 105 - 0.12\ PD_{THICKNESS} - 0.10\ PD_{STRENGTH} - 0.11\ (PD_{SMOOTHNESS})^2 \qquad (1)$$

where

PF = pay factor (percent),
$PD_{THICKNESS}$ = thickness percent defective,
$PD_{STRENGTH}$ = strength percent defective,
$PD_{SMOOTHNESS}$ = smoothness percent defective length.

An advantage of using percent defective (instead of percent within limits) is the clarity it provides in the pay equation. It can easily be seen that Equation 1 pays a maximum of 105 percent when all quality measures are at zero percent defective and that this value decreases as the percent defective of any of the individual quality measures increases.

ACCEPTABLE AND REJECTABLE QUALITY LEVELS

For any statistical construction specification, an acceptable quality level (AQL) must be defined. This selection is usually based on empirical observation of quality levels that have performed well in the past, although it may be based on other engineering considerations. Values around PD = 10 below some appropriate limit have typically been used, and this is believed to be suitable for thickness and strength in this example. For pavement smoothness, various research studies of NJDOT concrete pavement with expansion joints have suggested that PD = 5 is an appropriate AQL.

At the other extreme, as a safeguard against seriously defective work, it is customary to define a rejectable quality level (RQL) at which the agency reserves the option to require removal and replacement, corrective action, or assignment of a minimum pay factor. As a general rule, RQL values must be set at sufficiently low levels of quality that such drastic action is truly warranted. Because pavement failure does not pose a major safety hazard (such as the catastrophic failure of a bridge member, for example), the RQL limits for thickness and strength can be set at relatively high levels of percent defective. For pavement smoothness, however, studies by NJDOT researchers indicate that a percent defective length of 15, computed from the cumulative length of dye marks put down in the wheel paths by a 3.05-m (10-ft) rolling straightedge, provides a ride

New Jersey Department of Transportation, Division of Research and Demonstration, 1035 Parkway Avenue, CN 612, Trenton, N.J. 08625.

that is so rough that immediate corrective action is usually required. The following table of RQL limits and pay factors is proposed:

Quality Measure	*RQL PD*	*RQL Pay Factor (Percent)*
Thickness	70	65
Strength	70	65
Smoothness	15	65

The proposed RQL of PD = 15 for riding quality is almost the same as NJDOT has used in the past. The RQL value of PD = 70 recommended for thickness and strength may appear to be more lenient than past practices but in fact is not.

The current RQL for thickness is defined as an average value that is more than 12.7 mm (½ in.) less than the design value. Based on the typical standard deviation of 6.4 mm (¼ in.) for rigid pavement thickness, this corresponds to a percent defective value of nearly 98 percent. The proposed RQL of PD = 70 is more demanding and corresponds to an average thickness deficiency of only about 3.2 mm (⅛ in.). [This is still about 12.7 mm (½ in.) less than the desired average thickness necessary to achieve the AQL of PD = 10. In other words, to meet the thickness requirement of this specification, the contractor must set a target value of about 9.5 mm (⅜ in.) greater than the design thickness.]

The current RQL for concrete pavement compressive strength is 10 percent below the structural design strength (f_c') of 20,670 kPa (3000 psi). Based on a typical standard deviation of 2067 kPa (300 psi), this corresponds to approximately PD = 85 below the class design strength of 25,493 kPa (3700 psi) for rigid pavement. In this case, the proposed RQL of PD = 70 below the class design strength is just slightly more stringent than past practices. It is believed to be sufficiently effective and administratively simpler to base the definitions of both the AQL and the RQL on the class design strength.

HOW THE ACCEPTANCE PROCEDURE WORKS

For pavement of clearly outstanding quality (zero percent defective for all three measures), Equation 1 awards a maximum pay factor of 105 percent. It is believed that the highway agency receives more than comparable value in terms of the extended service life of pavement of this quality.

It is also necessary to have some degree of bonus provision with specifications of this type for them to perform fairly. The reason for this is intimately linked with statistical sampling theory. Because of the natural variability of any sampling and testing process, some samples will underestimate the quality and others will overestimate it. Unless the adjusted pay schedule is designed to allow bonuses and reductions to balance in a natural way, the average pay factor will be biased downward at the AQL, and acceptable work may be unfairly penalized. Fortunately, even a small bonus provision usually corrects this problem.

For pavement of varying levels of percent defective in the three quality measures, Equation 1 assigns pay factors that range from the maximum of 105 percent to a minimum of about 65 percent. This minimum occurs when all three measures approach their respective RQL values.

If the RQL value is reached on any one of the quality measures, the highway agency has the option to require removal and replacement or corrective action. If for practical reasons these options are not exercised, a minimum pay factor of PF = 65 is assigned. There is no need to make the computation with the pay equation in this case.

A distinctive feature of this acceptance procedure is that, provided none of the quality measures reaches the RQL value, it permits surpluses and deficiencies in thickness and strength to offset one another. This is consistent with the AASHTO design equation, which allows the same flexibility when the initial design for the pavement is being developed. Another feature is the use of the second power (square) of smoothness percent defective in Equation 1. As a result, this term tends to exert greater influence so that relatively high levels of riding quality must be achieved for the contractor to benefit appreciably from the bonus provision.

Procedures of this type have two distinct advantages. One is technical: the equation can be designed to assign pay factors that are directly related to the value of the construction item through a rational, scientifically based process. The other advantage is administrative. The worksheet in Figure 1 shows how easy it is for anyone with a minimal amount of training to apply the procedure. In contrast, current specifications usually require careful attention to several isolated sections of text and may be more confusing and prone to error or misinterpretation.

PERFORMANCE MODELS

To the extent possible, pay schedules should be based on models that relate the various quality measures to the performance or service life of the construction items to which they are applied. When performance models are lacking and insufficient information is available from which to develop them, it will be necessary to rely on the judgment of experienced engineers.

The desire of engineers to create an incentive to control both the mean level and the uniformity of the construction process dictates the use of the percent defective as the statistical measure of quality. As already noted, some agencies use the counterpart of this quality measure, the percent within limits, which is functionally equivalent but sacrifices something in terms of clarity of the pay equation.

For pavement thickness and strength, the AASHTO design procedure (*1*) provides a convenient way to relate as-built quality to service life. Although it may eventually be supplanted by mechanistic design methods, the AASHTO procedure is widely used and provides basic guidance on the relative importance of key quality measures. For portland cement concrete pavement, it can be demonstrated that thickness and strength are of primary (and nearly equal) importance and that an increase in one can offset a decrease in the other.

For riding quality, a relationship such as the AASHTO design procedure is not available. However, an approximate performance model can be developed on the basis of expectations of experienced engineers. This will at least provide useful guidance in determining the term in the pay equation that pertains to pavement smoothness. The following assumptions are believed to be realistic:

- Design life = 20 years,
- AQL: $PD_{SMOOTHNESS} = 5$ yields 20-year life expectancy,
- RQL: $PD_{SMOOTHNESS} = 15$ yields zero life expectancy,
- $PD_{SMOOTHNESS} = 0$ adds 5 years to intended 20-year design life, and
- The performance curve should be *S* shaped and approach the *X*-axis asymptotically.

Given these conditions and assumptions, an exponential relationship such as that given by Equation 2 provides a reasonable es-

ROUTE 123 SECTION 10A LOT 21 DATE 6/1/93

START STATION 12 + 34 END STATION 22 + 34

TEST VALUES AND PD COMPUTATIONS

THICKNESS: 9.987, 10.623, 10.152, 10.229, 10.386

STRENGTH: 3825, 4230, 3870, 4520, 4015

SMOOTHNESS: 4, 3, 4, 2, 1, 3, 1, 4, 2, 4, 4, 3, 3, 4

THICKNESS	STRENGTH	SMOOTHNESS
Limit 10	Limit 3700	Total Length = 1000
N = 5	N = 5	Defective Length = 42
$\bar{X}$ = 10.275	$\bar{X}$ = 4092	PD = 4.20
S = 0.242	S = 286.7	
Q = 1.14	Q = 1.37	
PD = 12.37	PD = 6.56	NOTE: $Q = (\bar{X} - \text{Limit})/S$

RQL LIMITATIONS

THICKNESS	STRENGTH	SMOOTHNESS
PD < 70?	PD < 70?	PD < 15?
Yes ✓	Yes ✓	Yes ✓
No[a] __	No[a] __	No[a] __

(a) Removal and replacement, corrective action, or PF = 65. Skip pay factor computation.

PAY FACTOR COMPUTATION

$PF = 105 - 0.12\ PD_{THICKNESS} - 0.10\ PD_{STRENGTH} - 0.11(PD_{SMOOTHNESS})^2$

$= 105 - 0.12(12.37) - 0.10(6.56) - 0.11(4.20)(4.20)$

$= 105 - 1.48 - 0.66 - 1.94 = 100.92$

FINAL DISPOSITION OF LOT

Remove/Replace: Yes __ No ✓ Corrective Action: Yes __ No ✓

If both "No", enter pay factor: PF = 100.92

FIGURE 1 Worksheet for composite acceptance procedure.

timate of the expected years of service life ($L_{EXPECTED}$). Figure 2 shows that this equation satisfies the assumed conditions.

$$L_{EXPECTED} = 25\, e^{-0.001785(PD_{SMOOTHNESS})^3} \quad (2)$$

This performance relationship can then be combined with Equation 3 (*2*, p. 21) to develop a table of appropriate pay factors as a function of smoothness percent defective.

$$PF = 100\left[1 + (C_{OVERLAY}/C_{PAVEMENT})\,(R^{L_{DESIGN}} - R^{L_{EXPECTED}})/(1 - R^{L_{OVERLAY}})\right] \quad (3)$$

The terms and typical NJDOT values for rigid pavement with expansion joints are as follows:

PF = appropriate pay factor (percent) = dependent variable,
$L_{EXPECTED}$ = expected life of pavement (years) = independent variable,
$C_{PAVEMENT}$ = unit cost of pavement (bid item only) = \$47.85/m^2 (\$40/sy),
$C_{OVERLAY}$ = unit cost of overlay (total in-place cost) = \$11.96/m^2 (\$10/sy),
L_{DESIGN} = design life of pavement = 20 years,
$L_{OVERLAY}$ = expected life of overlay = 10 years,
$R = (1 + R_{INFLATION}/100)/(1 + R_{INTEREST}/100)$,
$R_{INFLATION}$ = long-term annual inflation rate = 4.0 percent, and
$R_{INTEREST}$ = long-term annual interest rate = 8.0 percent.

This expression can be applied to develop pay schedules for either rigid or flexible pavement. For the portland cement concrete pavement in this example typical results are as follows:

Smoothness Percent Defective	*Expected Life (Years)*	*Appropriate Pay Factor (Percent)*
0.0	25.0	106.4
2.5	24.3	105.6
5.0	20.0	100.0
7.5	11.8	86.4
10.0	4.2	69.5
12.5	0.8	60.1
15.0	0.1	58.0

Although these values must be regarded as approximate, they are the result of a rational model and procedure that relates riding qual-

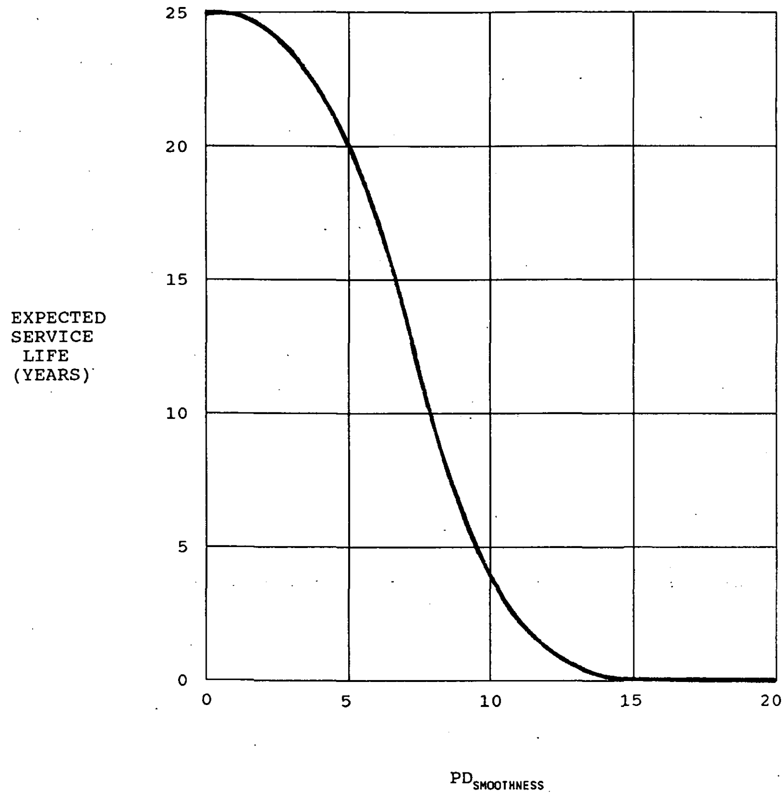

FIGURE 2 Approximate performance model for pavement smoothness.

ity to expected life and economic value of the as-constructed pavement. As such, they provide guidance useful in developing the pay schedule. Sensitivity tests have shown that these results are relatively stable over a wide range of input values. To be conservative, it was decided to set the maximum obtainable pay factor at a value somewhat less than 106 percent and the minimum pay factor for RQL work somewhat higher than 58 percent, as in Equation 1 and the in-text table of RQL limits.

DEVELOPING THE PAY EQUATION

Although there are an almost limitless number of forms the pay equation could assume, Equation 1 represents one of the simplest ways to translate the information in the performance models into a workable acceptance procedure. The coefficients of the terms in the pay equation must be chosen with care to be appropriate and defensible. A logical sequence of steps that works well with equations of this type follows:

1. Select the maximum (bonus) pay factor believed to be justified by superior quality. This is the intercept (constant term) of the pay equation.
2. Select the coefficients of the individual terms so that (a) the equation pays 100 percent when all quality measures are at their respective AQL values, (b) the magnitude of each coefficient reflects the relative importance of the corresponding quality measure, and (c) the amount of pay adjustment (bonus or reduction) is consistent with available performance models.
3. Select appropriate RQL values and the minimum pay factor to be assigned when the option to require removal and replacement is not exercised. This provision has considerable influence on how the average pay factor declines as quality decreases.
4. Check the operating characteristic (OC) curves for the complete acceptance procedure to ensure the procedure will perform as intended.

Step 1

In addition to the fairness issue discussed, both the AASHTO design equation and the approximate performance model for riding quality suggest that some degree of bonus for superior quality is warranted. On the basis of the potential savings associated with extended service life, it was decided that bonus pay factors up to a maximum of 105 percent are justified.

Step 2

The AQL values of percent defective for thickness, strength, and smoothness are 10, 10, and 5, respectively. Sensitivity tests (*3*) of the AASHTO design procedure indicate that thickness has slightly more effect than strength and accordingly warrants a slightly larger coefficient in the pay equation. The desire to make smoothness more dominant dictates a quadratic (squared) term for this part of the expression. The coefficients presented in Equation 1 were determined by trial and error to achieve the desired results.

Step 3

Another study (*4*) involving an engineering-economics analysis of portland cement concrete pavement concluded that a pay factor of approximately 60 percent is appropriate if the pavement is so poorly constructed that an immediate overlay is required. Equation 3, when combined with the approximate performance model for riding quality, produces essentially the same result. By using this information as a guide, a less stringent RQL pay factor of 65 percent has been recommended whenever the option to require removal and replacement is waived.

Step 4

As a final step, it is necessary to check the OC curves to determine how the complete acceptance procedure will perform. This is described in the following section.

SIMULATION TESTS

To determine how any statistical acceptance procedure will perform, it is necessary to examine the OC curves. A general discussion of this topic is contained in a recent TRB publication (*2*, p. 19). This step provides information in graphical or tabular form indicating the capability of the acceptance procedure to discriminate between acceptable and unacceptable work. It is through the study of such curves that the risks to both the highway agency and the contractor can be known and controlled at suitably low levels.

A computer program was developed to test thoroughly the acceptance procedure given by Equation 1 over a wide range of quality levels. Figure 3 gives an example of the output for a typical run. Figures 4 and 5 give several OC curves obtained from a series of runs with this program. It can be seen from Figure 4 that the theoretically appropriate pay relationship obtained with the tentative performance model for pavement smoothness is sloped more steeply than the current NJDOT stepped pay schedule, suggesting that somewhat larger pay reductions should be assessed for substandard quality and that some degree of bonus is warranted for superior quality. It can also be seen in this figure that, when thickness and strength are precisely at the AQL, both the pay schedule proposed as Equation 1 and the resulting OC curve lie about midway between the current stepped pay schedule and the theoretically appropriate pay levels. It is believed that such a compromise is a practical blend of effectiveness and defensibility, providing ample incentive to produce good quality pavements and at the same time adequately protecting the agency's interests. Note that, when all three characteristics are at their respective AQL levels (i.e., smoothness PD = 5 in Figure 4), the average pay factor is 100 percent, providing the contractor with full payment when acceptable work is delivered. As the smoothness percent defective approaches zero (extremely smooth riding quality), the expected pay factor exceeds 100 percent, approaching a maximum of about 103 percent.

To see what happens when thickness and strength are at other than their AQL values, a series of OC curves is plotted in Figure 5 on a more expanded scale. Curve B, for which thickness and strength are at their AQL values, is the same curve that appears in Figure 4. Note that almost the same curve is obtained when thickness PD = 0 and strength PD = 20, demonstrating that surpluses and deficiencies in these two quality measures can compensate for each other.

Curves C and D do not compensate in a similar manner to produce a single curve because the degree to which thickness and strength compensate is a function of the actual values involved. The negative effect of poor quality becomes greater as the quality decreases because there is a greater chance of triggering the RQL provision and having the minimum pay factor assigned. Curve D, because it represents a more extreme case of poor quality, indicates generally lower pay levels than Curve C. This may be a desirable effect because it tends to reward uniformity of quality.

Curve A was constructed under the assumption that both thickness and strength are at truly superior levels of zero percent defective. In this case, the curve approaches the maximum bonus pay factor of 105 percent as the pavement riding quality approaches an equally superior value. The other curves in this figure are labeled to

```
SIMULATION OF COMPOSITE ACCEPTANCE PROCEDURE FOR PCC PAVEMENT

REPLICATIONS = 1000

SEED NUMBER =   1234567

               SAMPLE     PERCENT                     RQL      RQL PAY
               SIZE       DEFECTIVE     CV            PD       FACTOR
THICKNESS      5          10            2.42          70       65
STRENGTH       5          10            7.34          70       65
SMOOTHNESS     1          VARIABLE      20.00         15       65

PF = 105 - .12 PD(THK) - .10 PD(STR) - .11 PD(SMOOTH)**2

PD(SMOOTH)     AVERAGE PF
  .0           102.7
 2.5           102.1
 5.0           100.0
 7.5            96.4
10.0            91.2
12.5            83.1
15.0            75.3
17.5            69.2
20.0            66.8
```

FIGURE 3 Typical output for computer simulation test.

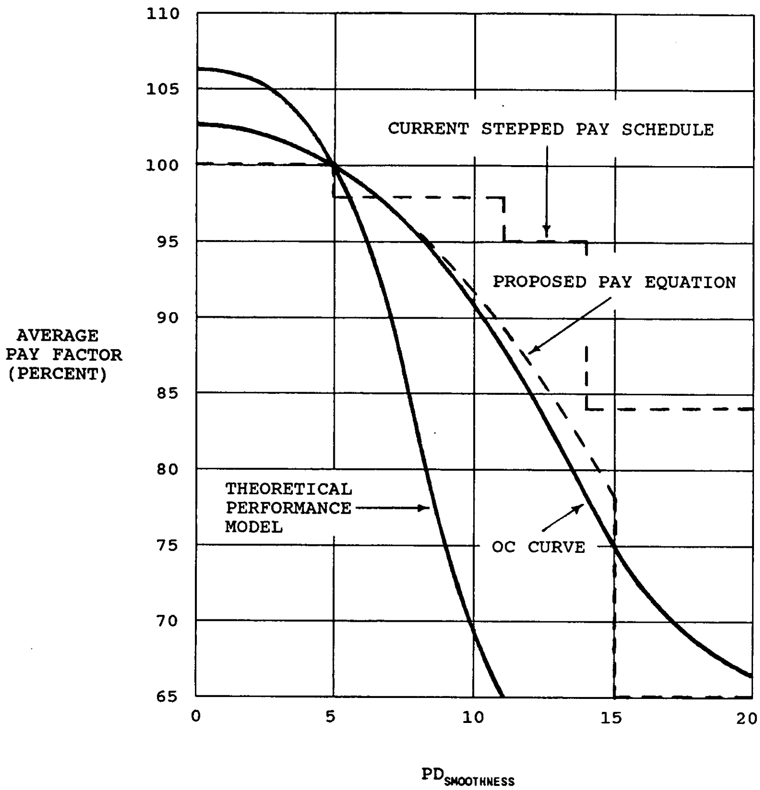

FIGURE 4 OC curve with thickness and strength held constant at AQL values.

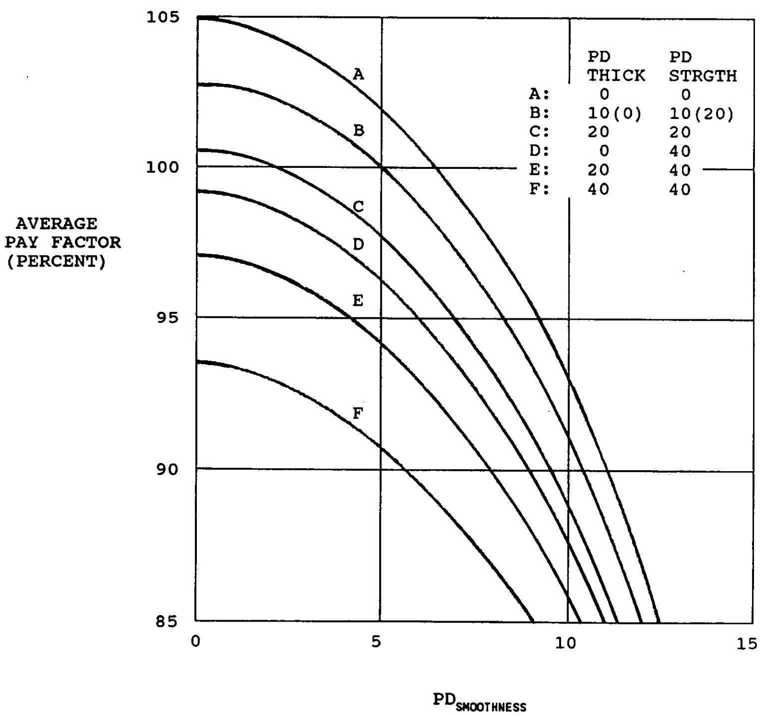

FIGURE 5 OC curves for selected combinations of quality levels of thickness and strength.

show various selected levels of quality control for thickness and strength. Curve C, for example, indicates that when thickness and strength are both marginally defective at a level of PD = 20 the pavement must be extremely smooth to obtain an average pay factor of 100 percent. Similarly, Curve A demonstrates that, if the smoothness percent defective exceeds PD = 6.5, the overall average pay factor will not exceed 100 percent no matter how thick or strong the pavement is.

OTHER PAY EQUATION FORMS

Although the linear form of pay equation may be the most practical for many applications, there may be situations in which other forms are more suitable. In Equation 1, the second power was used for the riding quality term to increase the effect of this term as the level of quality decreased. If the opposite effect had been desired, the square root (or some other fractional power) could have been used.

Another way that riding quality could have been made the dominant factor in Equation 1 would be to add the constraint that the maximum pay factor (PF_{MAX}) to be awarded is limited by a second expression, such as Equation 4. This is a more powerful restriction that was not thought to be necessary in this particular example.

$$PF_{MAX} = 111 - 2.0\,PD_{SMOOTHNESS} \tag{4}$$

There could be still other situations in which either the product or the average of the individual pay adjustments might be the most appropriate mathematical expression to use in the pay equation. In effect, Equation 1 makes use of a weighted average because the coefficients of the three terms are not identical. In certain special cases, either the largest or the smallest of a series of individual pay adjustments might be most nearly representative of the effect on the actual value of the construction item. In most cases, however, a linear expression similar to Equation 1 is likely to be satisfactory. In all cases, the construction of the OC curves will answer questions about the ultimate performance of the acceptance procedure.

SUMMARY AND CONCLUSIONS

A method was developed by which several acceptance requirements can be combined into a single, composite pay equation. The procedure is rational and scientifically based to the extent that performance models relating quality to service life exist or can be approximated. It is believed that this approach requires considerably less specification language to describe, is much simpler to understand and administer, and is less prone to error.

An example based on portland cement concrete pavement was presented. In essence, the relatively complex AASHTO design equation was combined with engineering-economics principles and restated in the form of a linear pay equation (except for one quadratic term). Of necessity, there is some loss of rigor in this simplification, but it is believed to be made up by the practicality of this approach. An extensive series of OC curves was developed to demonstrate that the procedure will perform as desired over a wide range of conditions.

This general approach can be applied to almost any construction specification involving any number of acceptance parameters. The linear form of the pay equation is probably the most practical, but other forms may offer certain advantages and should not be ruled out. The widespread interest in developing better performance relationships will continue to improve the models that support this approach. Where such models are not yet available, engineering knowledge and experience can bridge the gap to ensure that sensible results are obtained over the range of conditions likely to be encountered. A series of steps was presented that can be useful in developing the pay equation and associated RQL provisions. The final and perhaps most important step is the construction of the OC curves to verify that the acceptance procedure will function as intended.

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Upgrading and Recycling of Pin-Connected Truss Bridge by Pin Replacement

SHAHIN TAAVONI

Rehabilitation of the Carroll Road Bridge, a wrought iron Pratt through-truss bridge, built in 1879, is described. The goal was to preserve its historical character while upgrading the live-load capacity to meet all Maryland requirements for legal loads at the operating stress level. Inspection revealed that tension members had moved substantially; consequently, the load was no longer equally distributed. This effect was taken into account in pin and truss member analyses performed with the assistance of software developed by the author. Results indicated that the pins had minimal live-load capacity. To upgrade pin capacity, two methods were investigated. The first was to prevent transfer of all or part of the loads to the understrength pins. However, this could only be achieved by dramatically altering the appearance of the bridge and was, therefore, unacceptable. The second was to increase pin capacity by either prestressing or replacement. Prestressing was rejected because it could increase pin shear capacity, but not bending capacity. The only alternative was to replace all the pins with ones of higher strength. In-place replacement was cost prohibitive and time consuming, so the bridge was dismantled, and the two trusses transferred to a working space for repair. On completion of repairs and incidental substructure work, the bridge was reassembled and reopened to traffic. Three major conclusions emerged. First, in older bridge trusses, original symmetrical arrangement of the parallel components about the centerline of the pins usually no longer exists, resulting in a reduction of the load-rating capacity of truss members and pins. Second, in most cases, replacement of pins is the only acceptable solution. Third, the pin replacement method used was efficient, expeditious, economical, and suitable for similar situations.

This paper describes the rehabilitation study and subsequent rehabilitation of the 113-year-old Carroll Road Bridge. This wrought iron Pratt through-truss bridge crosses Carroll Run in a rural section of northern Baltimore County, Maryland. The bridge spans 92 ft and is supported at each end by stone masonry gravity abutments. The trusses consist of eight 11 ft 6 in. panels fabricated of pin-connected eye bars and riveted channels with lacing bars.

Most of the primary truss members have two components. Lower chord tension members consist of pairs of parallel eye bars, and compression members have two channels with lacing bars or batten plates riveted between the channels. Some of the diagonal tension members have an additional component parallel to the original pair, which was installed during previous repairs.

Spaced at 17 ft from center to center, the trusses accommodate a timber deck that provides a 14 ft 6 in. clear roadway between timber curbs. The deck is supported by stringers that transfer the loads to floorbeams. At each panel point, the floorbeams are suspended from the lower truss pins by U-bolts. Pin sizes are 2 and 2½ in. in diameter for upper chord and lower chord pins, respectively. Typical truss elevation and pin connection before rehabilitation are shown in Figures 1 and 2, respectively.

Kennedy, Porter & Associates, Consulting Engineers, 2319 Maryland Ave., Baltimore, Md. 21218.

PRELIMINARY REHABILITATION STUDY

In 1988 Kennedy, Porter & Associates (KPA) was retained by the Baltimore County Department of Public Works to make a preliminary rehabilitation study of the Carroll Road Bridge and seven other similar pin-connected wrought iron Pratt through-truss bridges. The goal was to preserve the historic character of the bridges and upgrade the live load capacity so that all Maryland legal loads could be sustained at the operating stress level.

As part of the rehabilitation study, it was necessary to check the load-bearing capacity of the pins and other connections, along with all truss and floor members, to ensure that all could sustain the proposed increase in live load.

Before the actual analyses could begin, a field visit to each bridge was necessary to measure pin size, evaluate the condition, and measure the spacing between components of the various members that bear on each pin. Small samples of material were taken from the pins and other truss members by lightly filing the surfaces. The samples were then sent to a laboratory for chemical analysis and metal identification. Because of the limited size of the recovered samples, it was not possible to perform mechanical tests. On the basis of the results of the chemical analyses and research of the technical literature, mechanical properties of the pins and other members were determined.

Research also indicated that the original design philosophy was to space adjoining truss members symmetrically about the pins (*1*). The symmetrical spacing was apparently adopted for the following reasons:

- To minimize pin stresses; and
- To induce equal strains and, hence, an equal force distribution through both components of any given truss member.

In the analyses, an equal distribution of force was assumed between parallel components of a particular truss member. Each pin was isolated as a free body with the truss member forces maintaining static equilibrium. Shears and moments were then computed on the basis of classical elastic assumptions.

The analysis was complicated because the pin loadings are a function of the placement of the truss members with respect to each other and the enormous quantity of possible live-load situations. With several pins in each truss and 11 sections of each pin to search for maximum bending and shear, the computations became extremely tedious. To expedite the process, a computer program

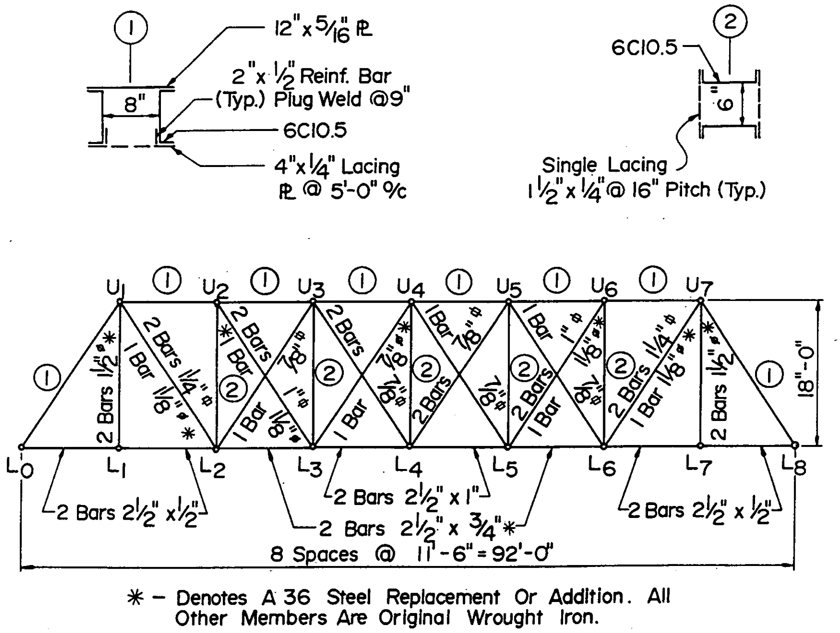

FIGURE 1 Typical truss elevation before rehabilitation.

was developed that completely evaluates truss pins under both static and moving loads (*2*).

In the pin analysis, each pin was treated as a beam with each vertical member component applying a vertical load, each horizontal member component applying a horizontal load, and each diagonal member component applying a vertical and a horizontal load. The computer program was used to calculate the vertical and horizontal components of bending and shear and the resulting bending and shear vector at each section of the pin, as a vehicle moves along the bridge. Subsequently, the maximum bending and shear at each pin were calculated and compared with the pin's capacity. These values allowed a rating value to be given to each pin. Note that as a vehicle moves along the bridge the bending and shear vectors at each section of a particular pin change direction. As a result, the live-load bending and shear vectors are not necessarily in the same direction as the corresponding dead load bending and shear vectors. Therefore, the usual method used for establishing the member rating could not be adopted. As a result, a vectorial analysis approach was adopted in the computer program for calculation of the bending and shear ratings. The computer analysis determined that in all cases the pins had a minimal live-load capacity and governed the rating of these bridges.

In spite of the analysis showing that some pins had very little live-load capacity, no pins have failed in any of these structures while operating under substantially higher live loads for the following reasons:

- The bridge was analyzed using a two-dimensional model, although in reality the two trusses and upper and lower bracings act together in a three-dimensional manner. Therefore, there are many different paths for the transfer of load from distressed pins to other less stressed parts of the structure.
- Ultimate plastic capacity of the pins is substantially higher than the elastic working capacity used in the capacity rating of the pins.

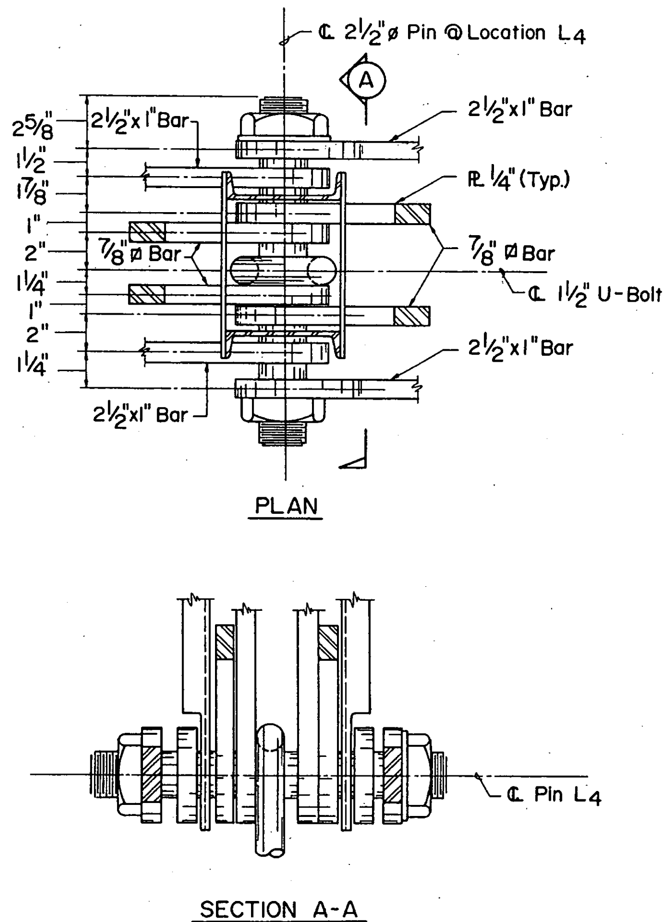

FIGURE 2 Typical pin connection at Joint L4 before rehabilitation.

• Because of the restricted width of the bridge, vehicles move slowly on the bridge. Therefore, the impact factor would be less than that stipulated by AASHTO.

FINAL REHABILITATION STUDY AND DESIGN

In 1990 the Department of Public Works decided to rehabilitate the Carroll Road Bridge to sustain all Maryland legal loads at the operating stress level. KPA was retained to provide the final rehabilitation study and contract documents for upgrading this structure. The main objectives of the design were to

• Preserve the historic character and original shape and form of the bridge because it is eligible for listing in the National Historic Register,
• Keep cost as low as possible,
• Protect the surrounding environment from the impact of the construction work in the area of the bridge, and
• Rehabilitate the bridge as quickly as possible.

For the final design, it was necessary to assess the load-carrying capacity of the pins and truss members as accurately as possible. Therefore, the truss pin analysis method used in the preliminary study was carefully reexamined. From this reexamination it became apparent that tension members, originally consisting of pairs of parallel eye bars placed symmetrically about the centerline of the pins, had in some cases moved substantially and were no longer symmetrical (see Figure 2). This condition has apparently resulted from the dynamic effects of the live load. As a result, pin deflections at points of contact with parallel truss member components were no longer equal. Consequently, the load was not equally distributed between parallel components, as was assumed in the preliminary study.

The original symmetric spacing of parallel components has also been disrupted over the years by various repairs and strengthening attempts that have added an additional component to some of the diagonal members without consideration of the effect on the pins and adjacent members (see Figure 1). This also causes an unequal distribution of force between parallel components of the diagonal members.

Note that fatigue and corrosion deformations of each parallel component (eye bar) surrounding the pins at the member ends may also cause unequal distribution of forces between or among parallel members of the truss. In other words, member ends may undergo a different sum of deformations as a result of different degrees of corrosion and fatigue deformations. However, visual inspection of the truss members and eye bars revealed that members were only moderately corroded, with no measurable loss of section. This was because most of the truss members were made of wrought iron and were well protected by paint. In addition, ultrasonic tests on all eye bars and other critical connections were conducted, and no fatigue cracks or distress were observed. Therefore, the effect of fatigue and corrosion deformations on load distribution was considered insignificant and was neglected in the analysis.

A method for distributing the force to each parallel component of a particular truss member on the basis of its location on the pin was developed. By means of this method, if the components of a truss member were not placed symmetrically about the centerline of the pin, the load would be distributed unequally between them.

The effect of unequal distribution of force between parallel components of a particular truss member was taken into account in both pin and truss member analyses. This was achieved with the help of a second computer program. This computer program is capable of providing a comprehensive analysis and rating of all of the pins and truss members (*3*).

The computer analysis of the Carroll Road Bridge, performed during the final design, determined that some of the pins and truss members had smaller rating capacities than originally found in the preliminary study (on the basis of the assumption of an equal distribution of load between different components of a truss member). The most profound result of the analysis was that most of the truss pins had minimal live-load capacity and were not capable of carrying the proposed increase in live load.

To verify the accuracy of the analysis, the Baltimore County Department of Public Works originally wanted to instrument the bridge, but because of insufficient funds the idea was abandoned.

Overall rating of the bridge was governed by the upper corner pin (pin U1 in Figure 3). The operating rating capacity of this pin for H and HS trucks was 3 and 4 tons, respectively. Other members with insufficient capacity were the deck, stringers, floor beams, and some of the truss members. This insufficiency was due to deterioration of the members or the proposed increase in the design live loads.

Replacement of the bridge with a new structure was not desirable because of the structure's historic significance. Therefore, it was necessary to identify the best rehabilitation alternative.

Retrofitting or replacement of the truss members, deck, and floor system is routine and straightforward. The most challenging aspect of the design was how to upgrade the pin capacity. Two methods were investigated:

1. Preventing transfer of all or part of the loads to understrength pins, and
2. Increasing the pin capacity.

The first method involves one of the following alternatives:

• Superposition of an additional structure, such as an arch, within the existing truss to carry a large portion of bridge live load and transfer it to the abutments;
• Placement of girders under the truss; or
• Prestressing the bottom chord of the truss to release the dead load and part of the live load on the bottom chord members and pins (*4*).

The first method would significantly change the bridge appearance and was therefore not acceptable from a historic preservation standpoint. Installation of girders under the existing truss was not possible because of intrusion on the hydraulic opening. Prestressing the truss bottom chord could reduce the load imposed on the bottom pins, thereby increasing the capacity, but could not release the load on the top chord pins to the desired amount.

The second method (i.e., increasing pin capacity) could have been achieved by prestressing or replacing the pins with new, higher strength pins. Because the prestressing method could only increase the pin shear capacity and not bending capacity, this method was discarded. Consequently, the only viable alternative was to replace all the pins with equal diameter pins of higher strength, and therefore higher capacity, material (FY = 100 ksi). Replacing the pins with larger diameter pins would also increase the pin capacity but would require increasing the eye bar hole diameter. This would require replacing the existing eye bars with larger diameter eye bars or increasing the eye bar hole diameter of the existing eye bars. These solutions were not desirable or acceptable.

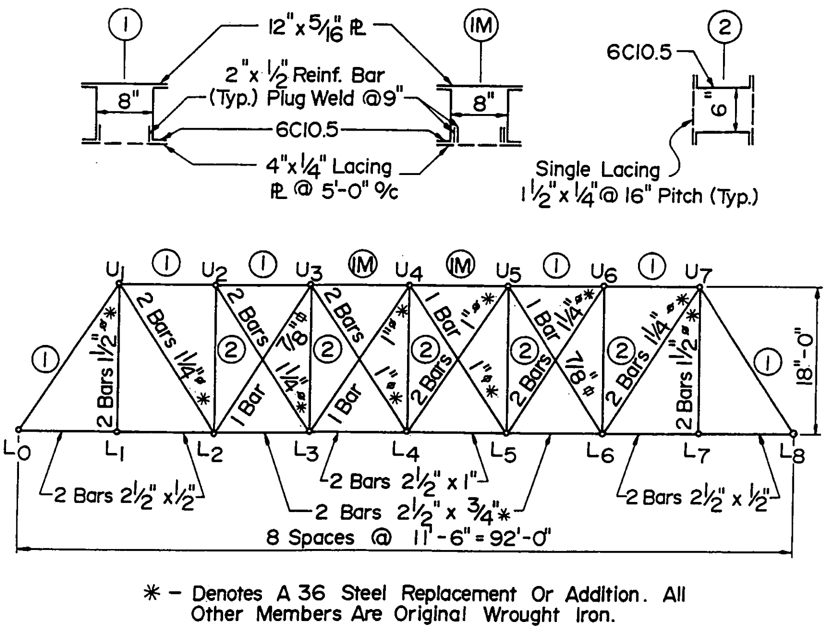

FIGURE 3 Typical truss elevation after rehabilitation.

To identify the best procedure for the replacement of the pins, several methods were considered, including:

1. Replacing pins in place;
2. Removing the complete bridge superstructure, transferring it to a work space, and replacing the pins; and
3. Dismantling the bridge, removing each truss completely, transferring them to a work space, and replacing the pins.

In-place replacement of pins requires shoring and supporting the entire bridge. This is because removal of one pin transforms the entire bridge into an unstable mechanism. This method proved to be cost prohibitive, time consuming, and obstructive to the normal flow of water. Removing the complete bridge superstructure could be done without any additional support for trusses during the lifting operation. However, the complete removal would require use of large cranes and removal of several trees near the bridge. The method finally selected was to dismantle the bridge and transfer each of the two trusses to a working space for pin replacement and repair. This method had the least adverse effects to the environment and was the most cost effective.

To reduce the stresses on the pins, optimize the existing truss member capacities, and return the trusses to their original form and shape, it was decided to remove all diagonal members that consisted of three components. These members were replaced with two components of adequate capacity, reinstating the symmetrical arrangement of truss member components about the centerline of the pins. Movement of the truss member components along the axis of the pins was evident on the existing structure and was attributed to the dynamic effect of the live load. To prevent this condition from occurring again, the rehabilitation introduced spacers where gaps between components existed. The spacers ensured that a symmetrical arrangement of member components was maintained. Figure 3 shows the members of the truss that were replaced or strengthened at the time of the rehabilitation. Because of deterioration of the deck, stringers, and severe corrosion of the floor beams, all of these members were replaced.

The existing timber deck was replaced with a new heavier timber deck of higher capacity. The decision to use timber deck to preserve the historical character of the bridge was taken. The assumed future service life of the rehabilitated bridge is 25 years.

CONSTRUCTION

Rehabilitation of the Carroll Road Bridge began in October 1990. The first step was to dismantle portions of the bridge. The existing timber deck and stringers were first removed, transferred to a working area behind the abutment, and later used as a horizontal platform for supporting the trusses during repair. Then, floorbeams and supporting U-bolts were removed and discarded. Next, the top and bottom lateral bracing was released and stored for future use. During the dismantling operation, Truss B was supported by guy wires at four points at the top chord joints and four points at the bottom chord. Truss A was supported by guy wires at two points at the top and two points at the bottom of the truss, in addition to the support provided by the lifting crane (See Figure 4). The guy wires were anchored to precast concrete deadmen, which were placed around the bridge. Figure 5 shows the connections of the guy wire to the concrete deadman and the top chord of the truss.

To strengthen the trusses during dismantling and lifting operations and to prevent buckling, two strongbacks were used for each truss (Figure 6). The first strongback was close to the lower chord and consisted of two channels placed horizontally and connected to the vertical posts by two 3/4 in. bolts. The second strongback consisted of two angles bolted to the vertical posts in the mid-height area of the truss.

To prevent any overstress in the truss members during the lifting operations, the dead load of the truss was transferred uniformly from the top joints of the truss to a lifting crane by a spreader beam.

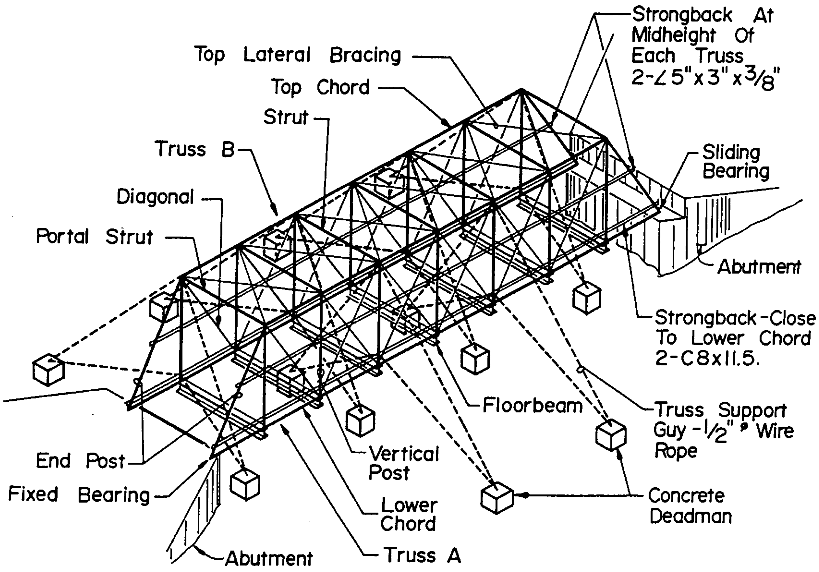

FIGURE 4 Truss stabilizing system during dismantling and re-erection of bridge.

Truss A was removed first and placed horizontally on the prepared platform, followed by Truss B. The entire dismantling of the bridge was done in one day.

Before removal of the pins, the members to be replaced were measured to determine their exact length, and the temperature was recorded. This information was later used in rebuilding these members to exact lengths, ensuring no change in the truss geometry.

Removal of the pins after more than 100 years of service proved to be a difficult task because of corrosion and permanent deflection of the pins. A special jack and sometimes heating were used for removing the pins.

After being removed, the pins were thoroughly inspected. Inspection revealed that some of the highly stressed pins had permanent deflections in the range of 1/8 to 1/4 in. This observation supported the theoretical calculations that indicated that these pins had been stressed beyond their elastic limits under live load. Several of the pins were corroded and pitted up to 1/4 in. deep; however, most were in good condition. Three pins from the upper chord and three from the lower chord were selected and sent to a laboratory for tensile and shear testing. Test results indicated that minimum yield stress was approximately 26 ksi. This value was close to what had been assumed in the rating analysis.

After removal of the pins and some other truss members, the remaining members were enclosed in a containment tent, sand blasted, and painted. Following the replacement of the removed members and the pins, the trusses were ready to be reerected.

The trusses were supported by lateral guy wires during the reerection procedures. The top and bottom lateral bracing, new steel floor beams, stringers, and timber deck were installed, and the bridge was reopened to traffic.

The overall cost of the rehabilitation of the bridge was close to $300,000.00. The operating rating for H and HS trucks was 23 and 37 tons, respectively. The member governing the rating of the rehabilitated bridge was bottom chord member L3-L4.

CONCLUSIONS

From this rehabilitation work, the following conclusions have been drawn:

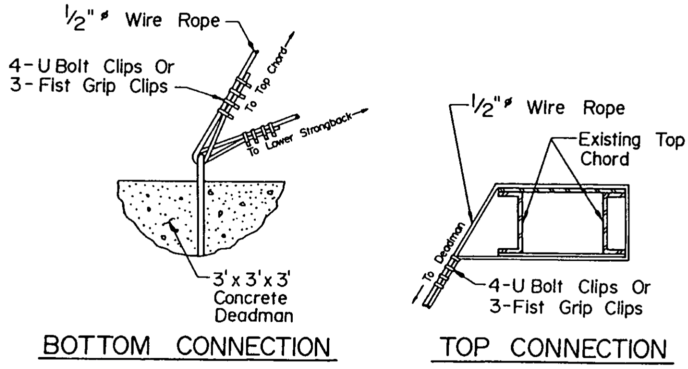

FIGURE 5 Deadman and truss connections.

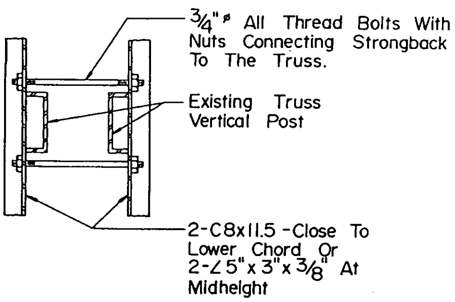

FIGURE 6 Strongback system.

• In old trusses, the original symmetrical arrangement of parallel truss member components about the centerline of the pins usually no longer exists. This results in an unequal distribution of loads in the parallel components of truss members and reduces load rating capacity of truss members as well as pins. This effect, often neglected in design and rating, should be taken into account.

• The rehabilitation of antique truss bridges requires special consideration of members that have been previously strengthened. In cases in which two component members have had a subsequent third component installed (as was the case at the subject structure), it is advantageous to replace completely the three-component members and return to a two component system. The new two-component member would be designed to sustain the proposed loads and be placed symmetrically about the centerline of the pin. This retrofit preserves the original character of the structure, is aesthetically appealing, and is preferable from a structural standpoint.

• It appears that the design and load rating of pin-connected trusses are often governed by the capacity of the pins. In cases in which the original form and shape of the truss must be preserved for historical reasons, pin replacement is the only practical solution.

• To eliminate fluctuations in the loads imposed on the pins and maintain the symmetry of the structure, it is advisable to introduce spacers where a gap exists between components bearing on the pin.

• The pin replacement method adopted for this project was efficient, expeditious, and economical, with the least adverse impact on the environment. Also it may be used in similar rehabilitation situations.

• According to the FHWA publications (*5*, p.9.8.45, *6*) definition, members with only one or two eye bars should be considered fracture critical members. In addition to the pin, each truss joint includes the ends of eye bar components, the ends of vertical components, connection plates, and so forth. These joint components are particularly vulnerable to fatigue stresses and corrosion and thereby contribute to additional nonredundancy of the trusses. It is critical for both rehabilitated and nonrehabilitated truss bridges that joints are frequently and meticulously inspected for any sign of distress due to fatigue or corrosion. Any required remedial action should be implemented immediately to ensure the safe operation of the structure.

ACKNOWLEDGMENTS

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Integrated Modern Contract Management for Highway Construction Work in Lebanon

MOHAMED-ASEM U. ABDUL-MALAK AND NAGI M. ABOU-ASSALY

After years of war, the deteriorated road network in Lebanon is in urgent need of rehabilitation and reconstruction. Previous projects have been characterized by budget overruns, poor quality, untimely completion, and disputes. This, along with the limited availability of public funds, raises the need to focus on the efficient execution of future projects to be administered by the Directorate of Roads of the Ministry of Public Works. In this research, modern and innovative concepts of contract management have been explored with the objective of improving current practices. Concepts considered include a comprehensive prequalification process, a multiparameter approach for bidding, surety bonding, night-time construction work schedules statistically based end-result specifications, adjusted pay schedules, owner-furnished equipment, prepayment and escalation clauses, and arbitration for dispute resolution. An integrated contract management approach is presented that is directed toward understanding the interactions, interrelationships, and interdependencies that exist among candidate concepts and strategies. Any attempt to deal with one contract aspect independently of the others may prove to be of little use. The integrated approach is expected to aid in (a) ensuring project success with regard to cost, schedule, quality, and safety and (b) enhancing competition and technology diffusion, while reducing disputes and contractor risks and financial obligations.

Roads in Lebanon have suffered extensive damage during 17 years of civil war, while receiving minimal maintenance and rehabilitation work. There are a total of about 3270 km (about 2000 mi) of roads in the network, 2000 km (about 1250 mi) of which have various levels of distress requiring some form of maintenance, rehabilitation, or reconstruction. The rehabilitation program for the roads network has been drawn up for execution in 5 years, with a preliminary funding requirement equivalent to more than 350 million U.S. dollars.

Despite the urgent need for implementing highway projects, the Directorate of Roads (DR) of the Lebanese Ministry of Public Works is constrained by the limited availability of public funds, from internal and external sources. This, in turn, raises the need to focus on the efficient delivery of future projects, especially because high cost, poor quality, and untimely completion have been common characteristics of previous projects.

RESEARCH OBJECTIVES

The objective is to seek and explore modern and innovative contract management practices that will ensure the most effective implementation of highway projects. Candidate areas include the prequalification process, bonds and insurance coverage, bidding policies and strategies, specifications and quality control methods, scheduling and delay problems, traffic congestion and night-time construction, safety, inflation and payment procedures, and claims and dispute resolution procedures. The ultimate objective is to provide an integrated approach, aimed at investigating and defining the interactions and interrelationships among all contract management areas studied.

The methodology used in this work involved: (a) review of current contract management practices through careful examination of the adopted construction documents (*1*), (b) interviews with a number of DR officials and prequalified highway contractors to document the major problems that they have encountered, (c) search of the technical literature for modern and innovative contract management concepts, and (d) manipulation and assimilation of the collected information into an integrated approach for implementing potential improvements.

FACTORS AFFECTING CONSTRUCTION CONTRACT

Figure 1 illustrates the factors thought to lead to the unsuccessful completion of highway projects. In view of the inherent risks, contractors incorporate varying contingency amounts in their bid prices. These contingency amounts depend on the types of risk anticipated, contractor attitude towards risks, and the level of accuracy in contractors' risk quantification (*2*). With limited financial capabilities, some contractors tend to allocate high contingency amounts, resulting in inflated bid prices. On the other hand, their need for work coupled with high competition may cause other contractors to use low contingency amounts to improve their chances of winning contracts (*3*). Such unrealistically estimated low bids may result in an increased number of disputes and, eventually, in total project costs that are higher than those of the most reasonably estimated bids.

SYSTEM WEAKNESSES AND POTENTIAL IMPROVEMENTS

This section deals with major aspects of contract management. For each aspect, the weaknesses of current practices are first identified, and potential improvements are suggested that incorporate innovative management concepts and practices.

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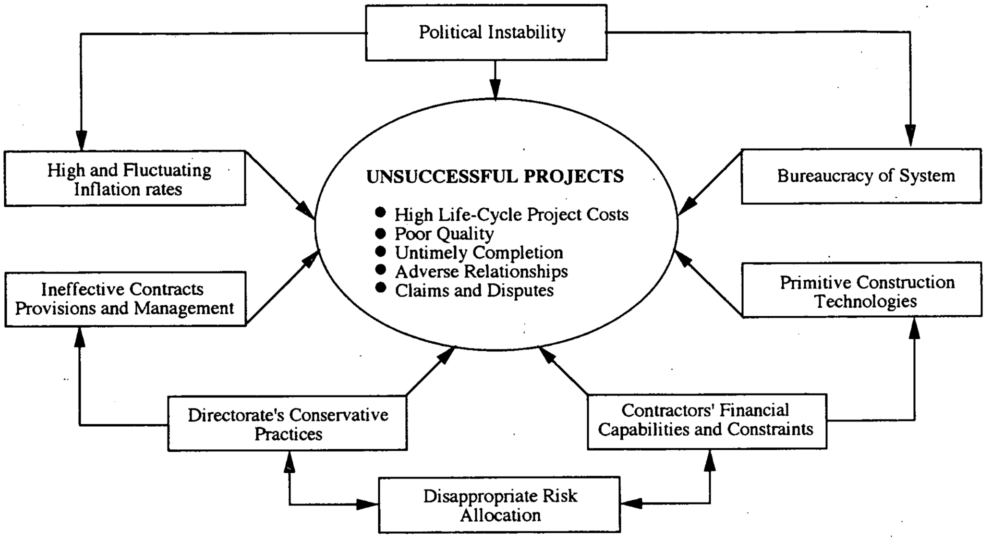

FIGURE 1 Factors leading to unsuccessful projects.

Prequalification of Contractors

Current Process

As indicated by interviewed DR officials, the current prequalification system tends to focus only on certain aspects involving the financial capabilities of contractors, the experience of the contractor's engineer, and the number and types of construction equipment owned. The system almost completely ignores other important aspects including the contractor's organizational framework, quality control systems, computer-based scheduling techniques, and safety programs. The process also limits to three the maximum number of uncompleted contracts a contractor may hold concurrently, without any consideration of individual specific qualifications and capabilities.

Systematic Approach

A more comprehensive and systematic process is needed that would evaluate all aspects of a contractor's qualifications (*4*). Under such an approach, an aggregate weighted rating is calculated for each contractor, which is then compared with an established minimum aggregate threshold. To overcome the shortcoming of a high rating in one factor compensating for a low rating in another, a minimum allowable rating for each factor may be specified. In the case of projects that require special qualifications relevant to one or more of the variables in the prequalification system, minimum allowable ratings specific to these project variables can be set. The ratings of a contractor's capabilities can also be used for determining the volume of work the contractor can undertake concurrently.

Bond Coverage

Bonding Requirements

Bid proposals must be accompanied by bid bonds in the amount of 3 percent of bid prices. A performance bond for 10 percent of the contract price must be submitted by the successful bidder within 10 days of the date of confirmation of the contract price by the Accounting Hall (*1*). This bond could be in the form of a cash deposit or a bank letter of credit. To obtain a letter of credit from a bank for 10 percent of the contract price, a contractor may have to freeze as much as one-and-a-half times that amount. Such a requirement sometimes hinders small contracting firms from tendering bids, thus reducing competition. This is because the number and amount of credit lines that a contractor could obtain from banks are greatly reduced by the amount of issued guarantees, because these are viewed by banks as contingent liabilities (*5*). Even if banks agree to finance both, it would be at higher premiums. The disadvantages resulting from the emphasis on financial bonds are illustrated in Figure 2.

Recommended Actions

The current bonding requirements often result in an adverse relationship between the DR and contractors, which may be detrimental to project success. Short- and long-term actions are recommended to provide adequate protection to the DR and, at the same time, reduce the financial burden and risk imposed on contractors. The short-term actions include (a) acceptance of collaterals as guarantees, (b) adjustment in bond amounts as work progresses, and (c) bond forfeiture on an arbitrator's statement. In the long run, surety bonds are highly recommended as opposed to financial bonds issued by banks. However, surety bonds are not available in Lebanon, and as shown in Figure 3, many obstacles exist to their availability in the near future.

Bidding System

Current Practices

Construction contracts are awarded on a competitive bidding basis in which contractors are required to complete the project within a maximum specified duration. Two systems prevail.

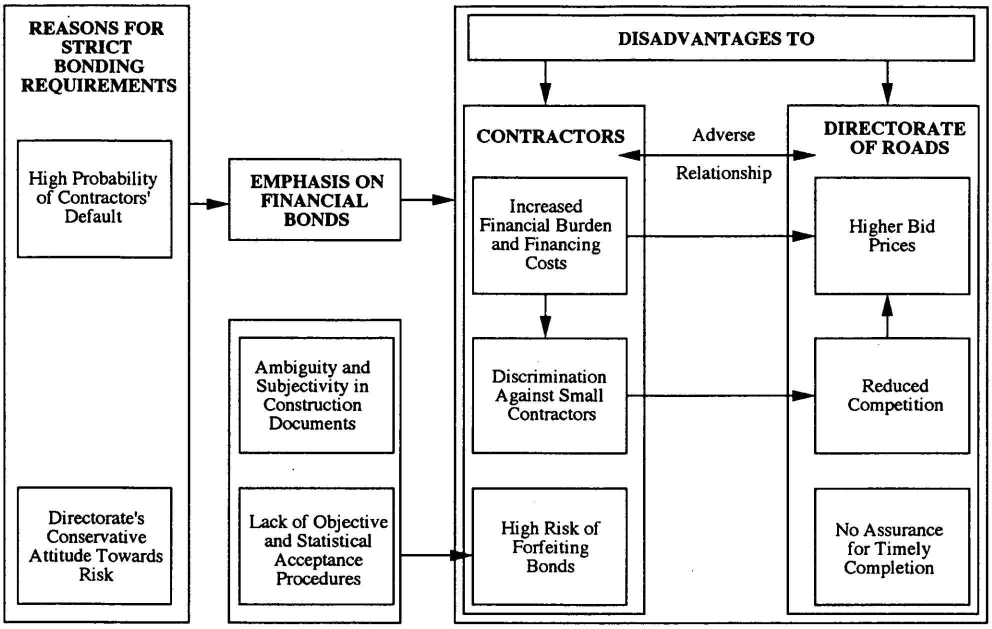

FIGURE 2 Overview of current bonding practices.

1. *Open competitive bidding:* All interested, prequalified contractors are given the opportunity to participate in the bid-tendering process.
2. *Competitive bidding by invitation:* The DR invites selected contractors with certain qualifications and experience to bid on projects. This system is more often used, particularly for urgent projects. However, the interviewed contractors reported that the selection is based on the judgment of DR officials, thus giving room for favoritism, bias, and corruption.

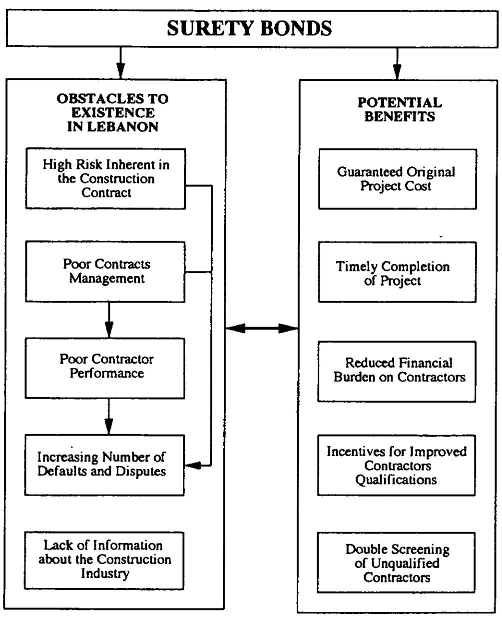

FIGURE 3 Surety bonds: obstacles to existence versus benefits.

As shown in Figure 4, the system adopted has resulted in many adverse effects on the highway construction industry in terms of higher costs, poor quality, untimely completion of projects, and reduced technology diffusion. Such a one-parameter (cost) system does not offer an incentive to contractors to shorten the projects and improve the quality of work by adopting innovative equipment and technologies and proven construction management techniques.

Multiparameter Approach

The multiparameter bidding system (*6–8*), which functions within the framework of the competitive bidding concept, may provide significant advantages over the current bidding system. The system is based on the concept that the successful bidder will be selected based on the cost, time, and quality offered by the bid.

Contract awards would then be made on the basis of the lowest total equivalent cost (TEC) calculated from a combination of bid price (BP) and schedule- and quality-equivalent cost figures. The schedule-equivalent cost (SEC) figure might be determined by multiplying a proposed duration by a predetermined time-equivalent rate, which could be set equal to the daily rate of stipulated liquidated damages. The quality-equivalent cost figure could be quantified on the basis of future performance reflecting the notion that better quality, although it entails a higher initial price, reduces project life-cycle costs (*8*). The future performance approach would allow contractors to bid on quality the same way they bid on cost and time. For instance, a contractor might propose in a bid a final mix density of 98 percent, compared with specified average density of 95 percent. This higher density would involve closely monitoring the compacting operations, which would result in increased construction costs. However, a higher achieved density would reduce future maintenance costs and increase the life span of projects. Therefore, the quality-equivalent cost (QEC) figure could be thought of as the present value of the expected

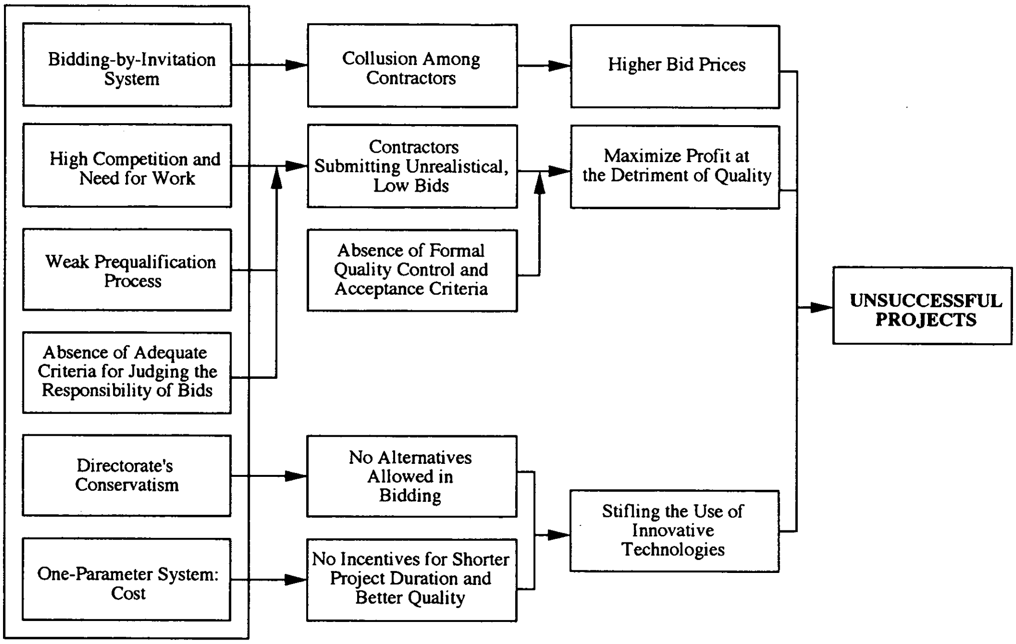

FIGURE 4 Weaknesses of current bidding practices.

annual maintenance expense associated with the specified density range or a higher proposed density spread over the corresponding project life span. Then, the total equivalent cost could theoretically be expressed as

$$\text{TEC} = w_1(\text{BP}) + w_2(\text{SEC}) + w_3(\text{QEC})$$

where w_1, w_2, and w_3 represent percent weights totaling one. This could be used to reflect the judgment of DR about the relative importance of individual cost figures.

In spite of the potential benefits of the multiparameter system, there are prerequisites to be fulfilled before such benefits may be realized. These are illustrated in Figure 5.

Specifications and QA/QC Programs

Prevailing Deficiencies

One deficiency of the contract management system is in the unstructured organization of the construction documents, particularly specifications, thus causing difficulties to contractors in locating necessary information. In addition, specifications are mostly descriptive, with almost no opportunity for contractors to consider or explore innovative materials and technologies. Moreover, the evaluation of accomplished work for acceptance and the determination of an applicable penalty for inferior work quality are based solely on the arbitrary judgment of DR engineers, and phrases such as "to the satisfaction of the engineer" or "to a reasonable quality" are not uncommon. Finally, major contractors reported that design plans, in addition to being conventional, generally fail to reflect existing site conditions; the contractors are held responsible for any design deficiency that may escape their review.

Modern Practices

It is recommended that the DR adopt a systematic approach for producing better standardized construction documents and specifications, thus helping reduce the liability and risk exposure of both parties (*9*). By providing a properly reviewed and managed design, the construction of a project would cost less and disagreements and subsequent litigation would be minimized (*10*). The DR is also urged to conduct value engineering analyses to select the most feasible designs, with consideration given to the life-cycle cost of projects instead of to the initial cost solely (*11*).

End-result specifications would give contractors the opportunity to select a method of construction that best helps them meet the performance requirements specified. In this regard, the DR is also urged to adopt formal acceptance criteria, to recognize the inherent variability in the quality of completed work, and to deal with such variability in a realistic manner. Statistically based specifications with adjusted unit prices would provide the solution (*12*). With such a system, price adjustment factors are based on the weighted means and standard deviations of quality parameters such as achieved mix density as measured by reliable tests. A price adjustment schedule would include, for example, a price reduction to be applied for deficient work and a bonus for better-than-specified work (*13*). Nevertheless, an option to require removal and replacement of unacceptable work at the expense of the contractor would still be included. A vital concern in the development of adjusted pay schedules is the determination of appropriate pay levels for various levels of quality. Such a schedule would be designed to withhold sufficient payment at the time of construction to cover the cost of future repairs made necessary by defective work.

Formal sampling procedures and test methods should be adopted or developed to cope properly with statistically based specifications. Emphasis should be placed on the use of nondestructive testing in the monitoring of construction work quality (*14,15*).

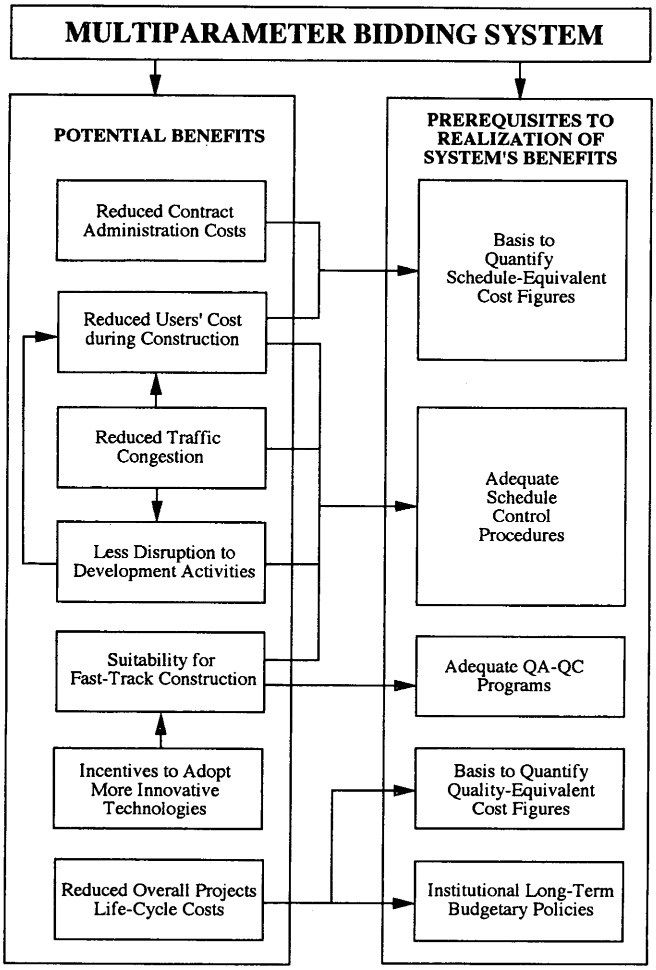

FIGURE 5 Prerequisites to realization of potential benefits of multiparameter bidding system.

Owner-Furnished Equipment

A contract management technique that should be contemplated for urgent rehabilitation projects is based on the concept that the owner may choose to furnish innovative and more productive equipment for use by contractors, particularly when such equipment is more expensive than individual contractors can afford. By adopting this concept, the DR would hedge the cost of equipment ownership against inflation and would reduce contractor financial obligations, ultimately resulting in lower bid prices. In addition, innovative equipment may result in a substantial increase in productivity that would, in turn, yield shorter schedules and cost savings (*16*).

Schedule Control Procedures

Because of institutional and political implications, little schedule control has been practiced with the exception of a rare enforcement of the liquidated damages provisions. The DR is therefore urged to use, and to require contractors to use, effective scheduling techniques to determine contract duration and to monitor and control the execution of projects. Such techniques may include the critical path method, line of balance method for linear scheduling, computer simulation of repetitive construction cycles, or a combination of these methods (*17*).

The recommended techniques should provide an objective basis for determining the necessary amount of time extensions for excusable delays instead of the subjective opinions of DR officials. Other recommended actions with the potential benefits and anticipated obstacles are shown in Figure 6.

Traffic Congestion and Night-Time Construction

Night-time construction has rarely been considered, except on a few urgent projects. However, with the severe traffic congestion experienced on most highways in Lebanon, the DR should study the feasibility of allowing night-time construction by investigating all the factors pertaining to such a decision, including highway class, anticipated level of congestion and travel delay due to the execution of work, existence of a feasible detour route, and road user cost. Where problems exist, night-time work schedules should be desirable and, therefore, be encouraged or required as part of the contract. Because of the lack of recent and complete statistics concerning the transportation sector, decisions might be based on limited traffic volume counts and queuing theory analysis, as well as on the experience and

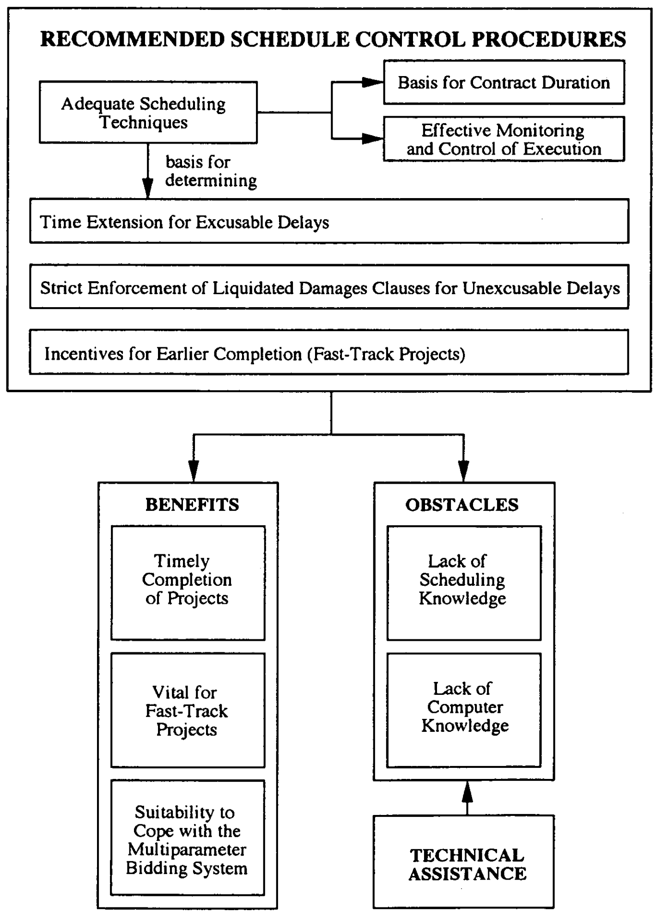

FIGURE 6 Recommended schedule control procedures: potential benefits and obstacles.

judgment of qualified DR officials. Figure 7 gives the potential benefits of night-time construction and associated limitations (*18*).

Safety Control Measures

The DR and highway contractors have failed to recognize that an effective safety program can reduce the added direct and indirect accident costs to projects. This is unfortunate because the ratio of hidden costs to direct costs can be as much as four to one or higher (*19*). In view of prevailing attitudes towards safety, the DR and contractors are urged to regard money spent on safety programs as investments and not as cost elements because a 4- to 8-dollar return can be expected for each dollar invested in safety (*20*). The following comprehensive and effective safety programs should, therefore, be developed.

- The DR should make the contractor's safety record an essential element of the prequalification process.
- The DR should develop methods of accountability and establish clear and systematic procedures for recording events and problems encountered on construction sites.
- The DR should set standards for safety measures and performance for shoes, hats, gloves, fences, adequate lighting, alarming devices, and traffic control devices.
- The DR should oblige contractors to furnish proof of insurance coverage, as an incentive to improve safety performance because insurance premiums depend primarily on the contractor's safety record (*21*).

Progress Payment and Inflation

Current Payment Procedures

Unit prices, all in Lebanese pounds, constitute the basis for payment to contractors. Progress payments are made monthly as approved by DR engineers, on the basis of their determination by field measurement of the actual quantities of work satisfactorily performed during the period covered by the payment. Ten percent of each progress payment is retained by the DR until the end of the warranty period.

Financial Risk Exposure

Interviews with prequalified contractors revealed that they experienced a number of financial problems, which can be summarized as follows: (a) high and fluctuating inflation rates, (b) provision of financial bonds and start-up expenses, (c) no reduction in the retained percentage as project progresses and no adjustment of the retained amount made to account for inflation, and (e) payment delays due to bureaucratic procedures. The results have been increased risk exposure, reduced competition, and higher bid prices.

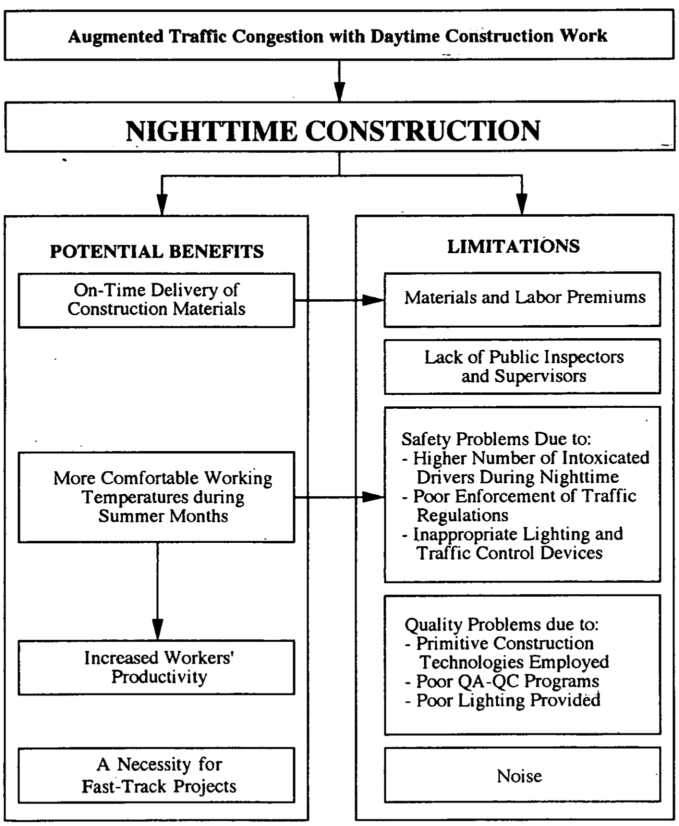

FIGURE 7 Potential benefits and associated limitations of night time construction.

Current Price Adjustment Measures

Inflation in Lebanon has gone beyond control, resulting in substantial fluctuation in the prices of materials, labor, and equipment used in construction. Consequently, inflation has become a major element of risk and uncertainty to the DR and contractors. The current contracts include compensation clauses to adjust prices for the period between the time of bid quotation and the time of work performance. Prices are revised upward or downward in accordance with the following general formula:

$$I = a_0 + a_1(D/D_0) + a_2(EFC/EFC_0) + a_3(MC/MC_0)$$

Where,

I = index reflecting price changes,
D = average exchange rate of U.S. dollar to Lebanese pound for 5 days preceding time of surveying accomplished work,
D_0 = exchange rate of U.S. dollar to Lebanese pound at time of contract award,
EFC = indexed equipment fuel cost for month covered by progress payment,
EFC_0 = indexed equipment fuel cost at time of contract award,
MC = indexed material cost for month covered by progress payment,
MC_0 = indexed material cost at time of contract award, and
a_0, a_1, a_2, and a_3 = relative individual weights for input parameters totaling 1.00.

For the case of asphaltic concrete work, only the costs of diesel and bituminous materials are individually adjusted, with relative weights of 0.25, 0.35, 0.15, and 0.25 assigned to a_0, a_1, a_2, and a_3, respectively. It is therefore assumed that the term D/D_0 would account for changes in the cost of labor, equipment ownership, and other nonbituminous materials. An upward revision is made only for the portion of a price increase that is beyond 5 percent. The final payment is adjusted upward or downward by the portion of the change in the U.S. dollar exchange rate to the Lebanese pound that is beyond 10 percent during the period between the time of certification of completion and that of approval of payment.

The deficiencies of the current compensation clause can be summarized by (a) lumping possible price changes of a number of construction resources in the term D/D_0; (b) not adjusting a sizable portion of the cost (a_0) for inflation, and (c) the inability of the indexes weights to reflect accurately the actual proportions of the different cost elements incurred by contractors. In view of these deficiencies, contractors still include, implicitly, an inflation-risk premium to preserve their profits. On the other hand, the DR has suffered a major difficulty with budget overruns because of excessive price increases.

Optimal Strategy

The optimal strategy to deal with the reported inflationary condition appears to be a combination of prepayment and price escalation clauses; the latter is to be applied only after price levels have increased beyond a predetermined level that can no longer be offset by the prepayment (22). The benefits of prepayment to the DR and highway contractors, along with its possible limitations and potential solutions, are given in Figure 8. Critical to the success of this strategy is the use of adequate indexes and relative weights in the escalation formulas (23). Work is underway to derive such weights statistically for asphaltic concrete work, because these may vary from one project to another, with variables such as the thickness of overlay and the density required. For example, a 10-cm overlay requires double the amount of material needed for a 5-cm overlay, but this does not imply that the required equipment input is doubled for the former to achieve the same final density.

Until surety bonds become available and while contractors are required to furnish financial performance bonds, it is recommended that less emphasis be placed on retainage, because this may entail an increase in bid prices to be eventually borne by the DR. It is therefore suggested that retainage be adjusted downward as the project approaches completion and that contractors be able to substitute other forms of security (such as collaterals) for a portion of the retainage, provided they show adequate performance as judged by statistically based criteria and sampling procedures. Such a practice would provide contractors with the incentives to improve performance and still ensure the DR enough liquidity to correct defective work.

Under inflationary conditions, it is only reasonable that the retained amount be adjusted to retain its value. This adjustment can be based on the change in the exchange rate of the U.S. dollar to the Lebanese pound. Finally, if payment delays are inevitable, contractors should be entitled to an equitable adjustment of the payment amount.

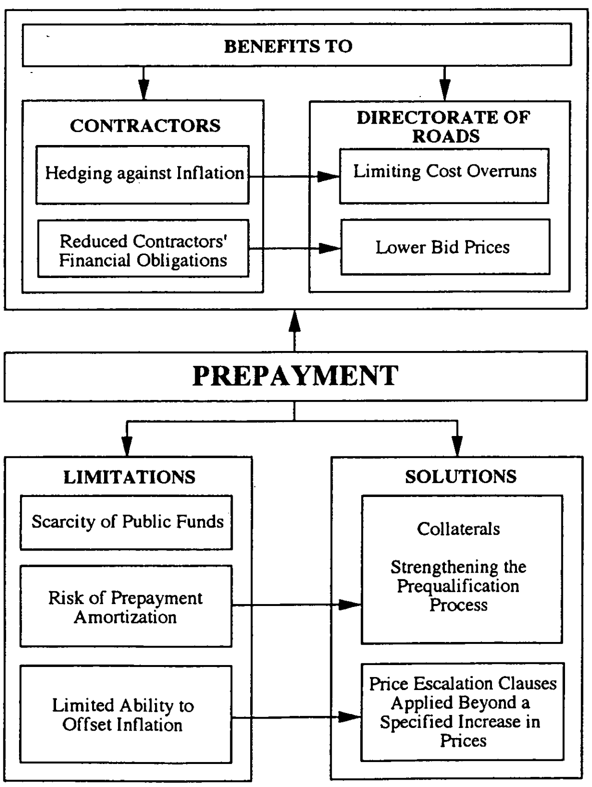

FIGURE 8 Potential benefits and limitations of prepayment.

INTEGRATED CONTRACT MANAGEMENT APPROACH

This section presents a framework for integrating the various contract management areas discussed. The integrated approach may help (a) ensure project success with regard to cost, time, quality, and safety and (b) enhance competition and technology diffusion and reducing disputes and contractor risks and financial obligations. The interactions, interrelationships, and interdependencies among the involved concepts are illustrated in Figure 9 and further discussed as follows:

- A comprehensive and systematic prequalification process would provide the DR with an effective means for screening out unqualified contractors and motivate contractors to improve schedule, quality, and safety control systems and acquire innovative technologies. In addition, strengthening the prequalification process would allow the DR to release its emphasis on financial bonds and retainage. The adjustment of the bond and retainage amounts as work progresses satisfactorily could therefore be contemplated.
- Proper measures concerning bonding and progress payment practices would help reduce contractor financial obligations, which would, in turn, be translated into lower bid prices.
- Innovativeness and technology diffusion can further be encouraged through the following. A multiparameter bidding system that emphasizes schedule and quality performance in addition to cost, thus resulting in earlier completion, better quality, and reduced life-cycle costs of projects; the use of end-result specifications that allow contractors to explore advanced technologies; and the adoption of the owner-furnished equipment concept.
- Improved schedule and quality control systems are essential to cope with the multiparameter bidding system in view of the existing trade-offs among cost, time, and quality. In addition, the use of open competitive bidding as opposed to bidding by invitation, would promote competition, minimize collusion among contractors, and reduce bid prices.
- A bidding system emphasizing quality should be accompanied by statistically based end-result specifications, along with formal and reliable sampling and testing procedures to judge the level of quality achieved. Also, adjusted pay schedules should be designed to account for individual levels of quality to which contractors are committed as part of their bids.
- Obliging contractors to provide insurance coverage will reduce their financial risks in case accidents occur and provide them with incentives to improve their safety programs to keep down premiums.

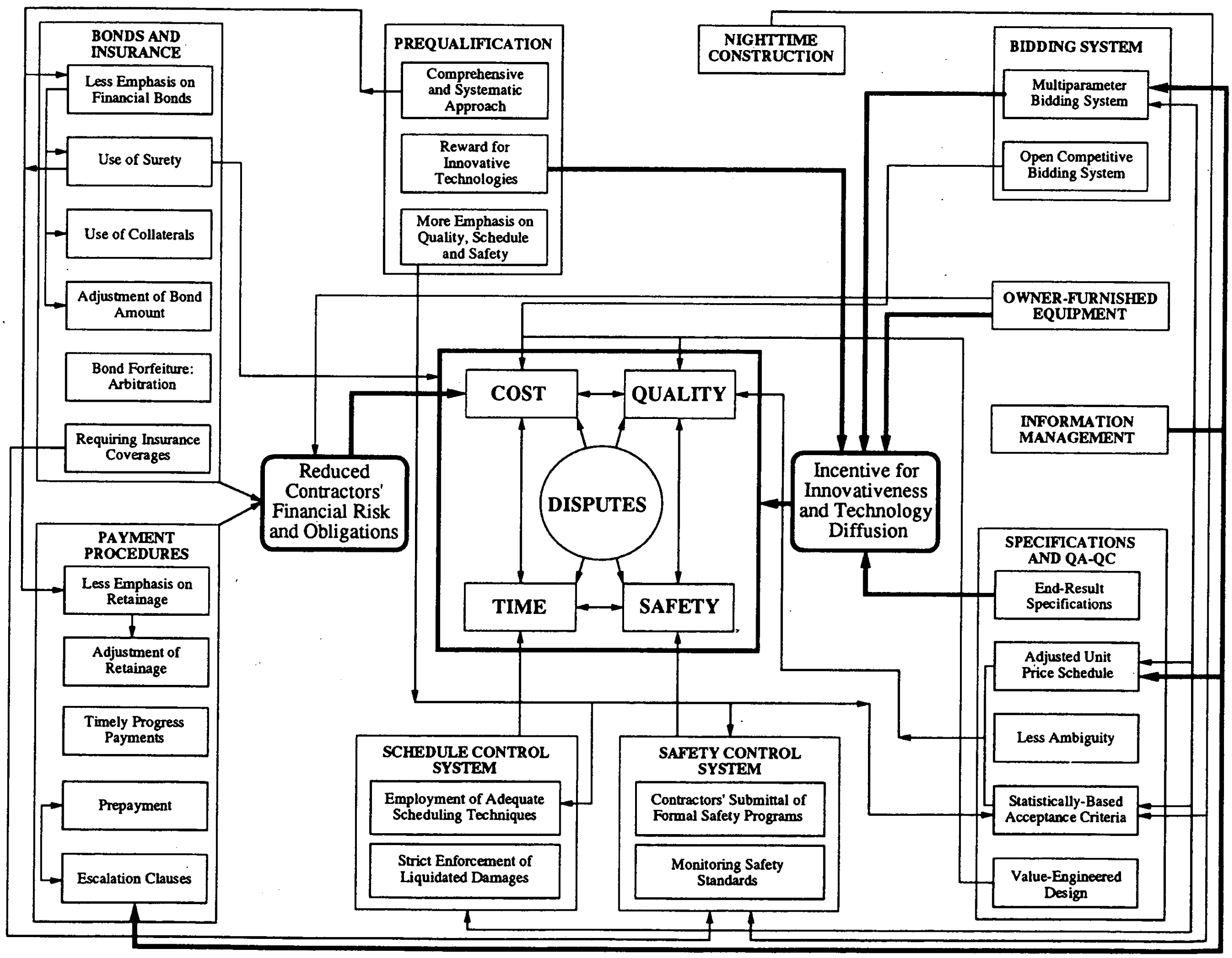

FIGURE 9 Integrated modern contract management system.

• Night-time construction, as a feasible work schedule for fast-track projects, should be accompanied by effective safety and quality control systems to avoid possible accidents and defective work.

• The key to avoiding disputes is the implementation of an adequate contract management system and assignment of competent and qualified personnel to handle claims promptly with a minimum of bureaucratic processing. For inevitable disputes, arbitration should be encouraged to avoid the expenses of litigation and ensure a quick settlement of disputes (*24*). Arbitration is also essential to cope with the problem of bond forfeiture, currently arbitrarily decided by DR engineers in the absence of statistically based acceptance criteria.

• Quantification of schedule- and quality-equivalent cost figures in the multiparameter bidding system, formulation of the escalation clauses, and development of adjusted pay schedules require the handling and manipulation of a large volume of data pertinent to cost, quality, and schedule. Therefore, collection of relevant data and development of adequate data bases and integrated information management systems become of critical importance for the successful implementation of such concepts (*25*).

CONCLUDING REMARKS

Cost, schedule, quality, and safety effectiveness could be improved by structuring the construction contract carefully to reflect clearly project objectives and by allocating risks fairly to both parties. The risk allocation mechanism should ensure optimum assignment of project risks among the parties involved. As such contractors would be held responsible for the controllable risks of meeting contract schedules and quality requirements, whereas the DR would bear the uncontrollable risks such as inflation and adequacy of design. However, this does not imply a complete separation in the risks allocated to each party. Instead, the risk mechanism should emphasize cooperation, reasonableness, and commitment by both parties.

To minimize the impacts of controllable and uncontrollable risks, improvements to the current contracting practices incorporating modern, proven concepts and strategies are warranted. However, special attention must be directed to understanding the interrelationships that may exist among candidate concepts, for any attempt to deal with one aspect independently of the others may prove to be ineffective.

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Design of Real-Time Site Operations Control for In-Place Asphalt Pavement Recycling

SOTIRIS A. PAGDADIS AND ILAN ISHAI

Recent improvements in computer technology have advanced industrial processes toward more autonomous control, with opportunities that offer superior handling, reliability, and enhanced overall quality. Although the highway construction process has advanced over the years as a result of rapid improvements in material technology, the on-site construction process still remains, for the most part manual. This is attributed primarily to the variability that exists on the road sites, creating situations that, under normal circumstances, are difficult to project and replicate from site to site. The use of automated equipment for hot in-place asphalt resurfacing is discussed as a modern technique that has advanced and provides the potential for further automation and improvements to current in-place technology. Breakthroughs that have been achieved by the authors toward improving this particular site process through real-time data handling are also discussed. The specific approaches are addressed that have been taken to automate and improve the process by incorporating the development of a nondestructive field-sensing process for identifying pavement and material characteristics, a blackboard procedure for field decision making based on real-time decision response to field conditions and information, and computerized process control that allows the integration of these new technologies with present equipment technology.

For some time the recycling process, with the use of glass and rubber materials, has been an ecologically attractive alternative in highway construction. These products have been used so successfully that now they are often mandated for use. With such technologies, recycling of the asphalt pavement itself has rapidly developed in the last decade in response to continued scarcity of materials and as a result of environmental, economical, and societal issues. Recycling of pavement materials offers several advantages over the use of conventional new materials. Observable benefits include the conservation of aggregates, binders, and energy; reduced hauling of materials; and the preservation of the environment and existing highway geometries (*1*).

Despite its attractive advantages, pavement recycling is associated with material-handling problems. Two fundamentally different approaches are applicable to the recycling process: one that considers the use of a central plant for preparation of the asphalt and one in which the asphalt is prepared in-place, i.e., at the site.

When central plant recycling is used, many steps are involved. They are milling of the old pavement surface; collecting and loading of the milled materials; hauling the reclaimed asphalt pavement material (RAP) to a central asphalt mixing plant; recycling RAP by hot mixing with virgin aggregates, fresh asphalt cement, and rejuvenating agents; hauling material back to the site; laying the recycled mix (after cleaning and tack-coating the new pavement surface); and rolling and compaction. Side issues are also involved, including the physical evaluation of the RAP properties, fresh asphalt, and virgin aggregates; design of the recycled mix; and quality control for evaluating the properties of the final mix. These steps usually enforce a substantial time gap between hauling the RAP to the mixing plant and the production of the recycled mix.

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IN-PLACE HOT PAVEMENT RECYCLING PROCESS

Generally, modern hot in-place surface recycling methods use a piece of equipment known as a remixer, which adopts a train-like moving operation (see Figure 1). A large portion of the in-place hot pavement recycling technique is conducted on site, with the need to haul only the additive components to the field location. The existing damaged wearing course is softened by preheaters and the remixer. Rotating scarifiers on the remixer loosen the bituminous material mixture, which is augured to the center of the machine where it enters a pugmill mixer and is mixed thoroughly with virgin materials. The new mix is then placed evenly to grade and slope by a compacting screed (*2,3*). One of the most useful remixer technologies is the WIRTGEN system, available in the marketplace for more than a decade.

Heating Pavement

Heating the pavement surface to between 140°C and 170°C softens the bituminous layer. Softening is done by clusters of infrared heaters fed with propane gas, which radiate thermal energy onto the wearing course. The heater units are quickly adjusted to the actual working width of the remixer. Because the gas pressure of the heater unit is individually adjustable, heat output is easily adapted to the ambient temperature, working depth, and material stiffness to avoid overheating of the bitumen.

Scarifying Wearing Course

Rotating scarifier shafts, fitted with spiral shaped carbide-tipped teeth, scarify the surface to the required depth. The scarifier can be adjusted to suit varying road lane widths. A leveling blade to the rear of the scarifier precisely skims the loosened material so that it can be augured to the mixer.

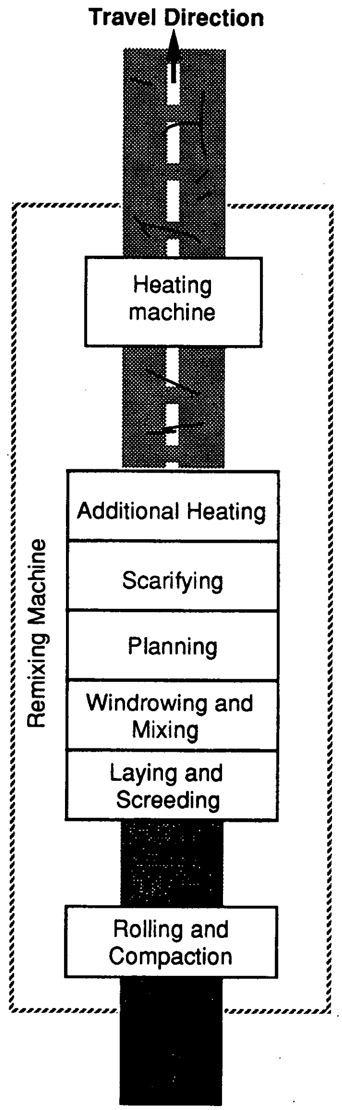

FIGURE 1 Train-like process for hot in-place asphalt pavement recycling.

Mixing Reclaimed Material

With the current technology, the design formula for the admixture is based on a prior core analysis of the existing pavement. The new material is either a hot bituminous mixture, a graded aggregate, a binder, or a rejuvenator. The virgin bituminous mixture for the new wearing course is mixed together with the reclaimed material in the compulsory pugmill mixer on the road. Trucks transfer the virgin material to the remixer and tip it into the receiving hopper. From there, it is conveyed by a drag-slat conveyer to the mixer. Exact mixing ratios are achieved by calibrating the speed of the electronically adjustable drag-slot conveyer to the foreword advance speed of the remixer. If the bitumen or rejuvenator is to be mixed with the reclaimed material, this can be supplied from a heated tank installed on the remixer. The virgin binder or rejuvenator is sprayed onto the reclaimed material by an adjustable pump. After thorough mixing, the material is discharged from the mixer through a window in front of the auger and screed.

Placing Mix

After the recycled material is discharged from the mixer onto the heated base, the mix is accurately placed to profile by an infinitely variable screed. The base material is heated separately to ensure a firm bond with the recycled mix by "hot in hot" placing. Final compaction is done by rollers.

SITE OPERATIONS CONTROL TO IMPROVE IN-PLACE ASPHALT RECYCLING

The central site recycling process has remained the favorite of the two processes because it allows greater control of the final product through centralized processing. It nonetheless dictates a more manual approach to be used at the site. The in-place recycling technique is much more efficient and innovative with respect to site material handling, yet several important problems that need to be addressed predominate. Because of the continuous nature of the process, there is no time to evaluate the old pavement material along the road for a proper design of the recycled mix. This requires tedious coring and sampling before construction operations. Furthermore, the inhomogeneous nature of old pavement surface and the inability to sense this variability along the road for a proper design and to provide continuous correction of the recycling formula usually cause follow-up variability of the recycled layer (an intrinsic difficulty in accounting for 100 percent recycling of old asphaltic materials characteristic of in-place operations). These difficulties and the sensitivity of in-place recycling to the variability in the old pavement cause a reluctance among pavement engineers to use the in-place technology.

Several novel techniques have been designed to improve the in-place recycling process. They are (a) discrete in-place field sensing, (b) intelligent monitoring and interpreting sensed properties in real time, and (c) continuous corrective control of the material component, handling, and quality control of the in-place recycling process.

By incorporating these technologies, a continuous travel condition is achieved with minimal delays through providing in-place discrete sensing, decision analysis, and feedback control. The sensing devices and units are based on new spectrometer technology applied to asphalt paving materials. The conceptually integrated procedure includes the following:

- Sensor unit for monitoring the old asphaltic layer and component properties;
- Heating unit for heating the top layer to be recycled and integrated with a sensing device that will monitor pavement temperature;
- Remixing machine that contains additional heating units, a scarifying unit, a planning unit, a windrow and mixing unit with sensing and control devices monitoring quality and quantity, integrated with storage and feeders for soft asphalt, fresh bituminous mix, and rejuvenating agent; and
- Laying and screeding units.

Furthermore, the procedure is designed to use a sensor unit for monitoring the new recycled asphaltic layer and its component properties, other conventional sensors integrated in the traveling recycling unit as installed by the manufacturer, an intelligent blackboard-based computer environment for decision analysis and feedback (which provides highway data on solutions to deterministic and nondeterministic asphalt conditions and knowledge sources on highway specifications and performance characteristics), and appropriate control characteristics, which implement corrective control to the overall process. The characteristic of monitoring and feedback of the system in real time provides a dynamic means for implementing quality control of the final product. This characteristic improves construction operations by optimizing material handling and also improves product quality and saves unnecessary costs.

Practical and economical reasons are obvious for maintaining a system with minimal manual intervention. This issue is further encouraged by the need for maximum efficiency. The use of advanced technologies for implementation in ill-structured construction environments encompasses the selective use of information systems, sensors, locomotion, and manipulation techniques, as well as control systems based on task needs (*4*). As these technologies are improved, their implementation through technology transfer ensures the growth of the benefiting fields. To achieve an integrated cycle as described, the site process must be controlled in real time. This requires design standards, field data collection, field management, site constraints, and project designs to be handled together. Intelligent data handling at the site level is difficult. An artificial intelligence procedure driven by a Site Operations Control System (SOCS) has been developed to overcome the data handling complications, which preclude the process from being achieved in real time (*5*). The SOCS is designed to interface with the remixing machine to provide intelligence, control, and advice. To achieve these characteristics, the SOCS includes

- A dynamic data base,
- Modular knowledge sources,
- A control module,
- An interface with the remixer's on-board controller,
- An interface with the site supervision group, and
- An interface with external files that characterize design information.

Blackboard System for Intelligent Data Handling in Real Time

The central component of the system is a blackboard control environment. The blackboard is a problem solving model that provides a conceptual framework for organizing knowledge and a strategy for applying that knowledge (*6*). It consists of three basic components: (a) blackboard data structure, (b) knowledge sources, and (c) a control structure. The blackboard data structure is a global data base that holds the current solution space, which is gradually built through the opportunistic contribution of the knowledge programmed into the machine's memory (knowledge sources). The control mechanism monitors the changes on the blackboard and allows the knowledge sources to respond to those changes. The system is integrated into the working structure of the remixer's onboard computer. The blackboard is used to monitor and control the flow of materials and provide feedback to the site supervisor. It is essentially an intelligent analyst and site inspector.

In general terms, the blackboard has been chosen because of its specific functionality. It has unique characteristics that are useful for site management and the remixer. They include its

- Support for the independence of expertise,
- Opportunistic problem-solving methodology,
- Ability to consider external information that arrives sporadically, and
- Event-based activation mechanism.

The blackboard views each knowledge source as a black box. In other words, it supports modularization of expertise. This is useful to the system primarily because of the different types of knowledge—operational, procedural, and organizational—that exist. Modularity of knowledge permits modification, addition, or deletion of knowledge source without affecting the other knowledge sources, thus for example, making changes to asphalt mix without interfering with road geometry and design specifications (see Figure 2).

The opportunistic reasoning mechanism provides the ability to consider external information that arrives sporadically. Field problems are not always a result of a single condition that arises. In the field, conditions change sporadically. It is important for the system to keep track of the information as it arrives and assign importance to the use of this information. This ability of the blackboard to consider information that arrives sporadically and to update impacts on the basis of this information is the only way that has been identified to allow the remixer to continue in its travel direction without interruption.

The blackboard provides an event-based activation mechanism. Instead of letting each knowledge source scan the blackboard system searching for an event in which it is interested, the blackboard keeps records of the information as the remixer moves down the travel direction and tracks what knowledge sources are to be considered for activation whenever the scheduled event occurs. Each source of knowledge (knowledge source) contains information about the situation it is interested in. The blackboard is assigned a strategy (function) to make local decisions about when each knowledge source should contribute to the problem-solving space.

The characteristics of the blackboard and its contributions to realtime process control are realized more fully by investigating specific contributions to the improved recycling system functions, namely, the real-time field sensing, process control, and real-time data monitoring that takes place.

Spectrometry

A method for characterization and prediction of different asphalt parameters using Near Infra-Red (NIR) spectrometry has been developed by the Transportation Research Institute of the Technion in Haifa, Israel (Ishai, unpublished report). The method is generally based on scanning the reflection of NIR radiation from the surface of the material. It is known that the NIR spectrum is sensitive to the movement of large molecules, thus providing the possibility of predicting different chemical and physical properties of the material.

The Transportation Research Institute of the Technion in Haifa, Israel, in cooperation with Pazkar Asphalt Company, has conducted a number of experiments that indicate the technology's success in predicting pure asphalt parameters, such as penetration (accuracy of ± 1.5 dmm), softening point (± 1.1°C), and polymer (SBS) content in modified asphalts (± 0.02 percent). After calibration and verification, a test measurement may take only a few minutes, including sample preparation, as compared with conventional methods, which take hours or days. Although development is in its infancy, it is believed that this technology will represent a breakthrough in

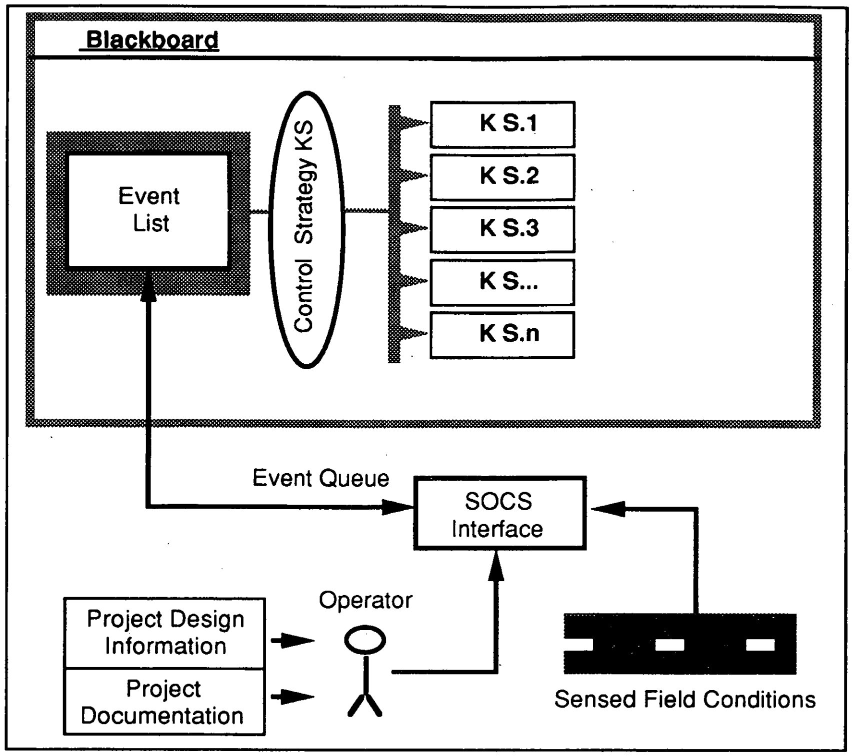

FIGURE 2 Blackboard structure for real-time control.

asphalt testing and prediction. Developmental issues investigated for using this technology on the remixer are multifaceted. They are

- Characterization of pure empirical and aged asphalt cements. Detailed experiments have evaluated such parameter characteristics as ductility, chemical composition, absolute and kinetic viscosity, flash point, and so forth. All these parameters are measured before and after thin film oven testing.
- Characterization of the loose bituminous mixture with respect to asphalt content and asphalt characteristics.
- Characterization of compacted or cored bituminous concrete samples in the laboratory with respect to density, asphalt content, and asphalt characteristics.
- Sensing and characterization of the asphaltic surfaces with respect to the same properties as noted previously.

The sensing system is composed of a fiber-optic sensor with a computer interface for scanning the road surface, a spectrometer responding to the reflected infrared light from the road surface, and a computer unit with the required interfaces for data acquisition. This technology is essential to the improved design of the remixer. Information is fed into the on-board computer of the remixer (which is controlled by the blackboard) through the sensors at various instances (see Figure 3). The blackboard priority ranks the incoming information and activates the appropriate knowledge source needed to feed information further down the train-like process. The knowledge sources represent information that has been provided through historical characterization in the laboratory to be available to the system in real time.

Controls for Material Handling

Automatic control and handling of materials are key features in the integration of computer-based information systems with automated equipment. The control issues for the materials handling of the asphaltic materials in the system reflect a proper response to the following impending requirements:

- Integration of handling and temporary storage activities in a feasible manner that includes reception, inspection, and temporary storage of asphalt materials;
- Effective use of available in-vehicle storage cubic space for handling the immediate and end-point materials;
- Mechanization of the handling process to increase efficiency and economy; and
- Application of quality control principles to the end product.

The blackboard controller is designed to coordinate the entire process. Proper sensing through weight, proximity, and motion sensors (tachometers, position encoders) provides the necessary feedback information for maintaining a stable system. There are three essential areas that require integrated controls functioning between the blackboard computer environment and the automated equipment.

1. The blackboard will evoke an event to read the pavement temperature, which is transmitted through the spectrometer and a discrete event sensor. With the spectrometer, a processing element is used to adjust the signal of the infrared heaters according to the desired temperatures and the actual temperatures as measured by the

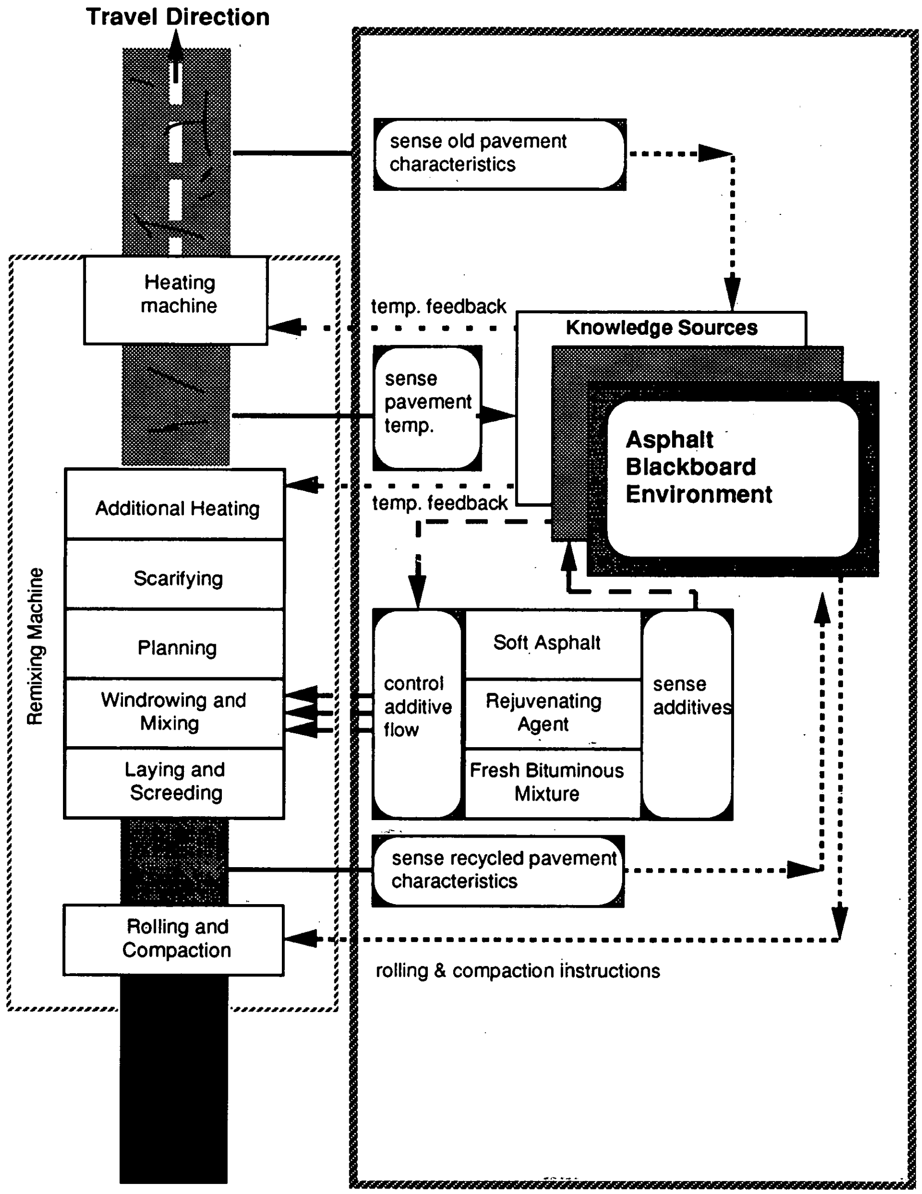

FIGURE 3 Overview of feedback system design for asphalt remixer.

thermocouple. A second signal establishes a decentralized control loop, for feedback of additional heating to the road surface (see Figure 4). An analog-to-digital converter translates the signal, which is read by the computer data bus, and the information is fed to the blackboard. This event on the blackboard immediately recalls the reference knowledge source that provides feedback about temperature sensitivity to the design parametry.

2. Specific NIR sensors are used to measure asphalt viscosity and asphalt content in the mixture relating to the external needs of the closed-loop recycling system. These sensors have their signal converted through A/D to the computer data bus where the information is retrieved by the blackboard and processed for instructions. The knowledge sources each provide reference data about asphalt content and viscosity that will be required externally by the system to meet design standards. The blackboard interprets the recommendations of the knowledge sources, which are then transmitted as instruction signals through the controller reference input (see Figure 5) as a means of controlling the additive flow (see Figure 6). The information is relayed as requests for additional material, which is received by the storage bins on the remixer. The controller reference input constantly monitors flow to ensure design parameter requirements.

3. The last monitoring function of the system is to provide reference quality of placed material during rolling and compaction and computer feedback, which is stored as memory further down the travel direction. This feature is important to the system because it provides a means for updating the blackboard data base and storing physical actions taken in correcting the mixture (see Figure 7). This closed-loop feedback suggests that the system has a capacity of learning from its own prior experiences. Essentially the rolling and compaction requirements decided on the field are based on field evaluation and inspection. The decision to proceed with rolling and

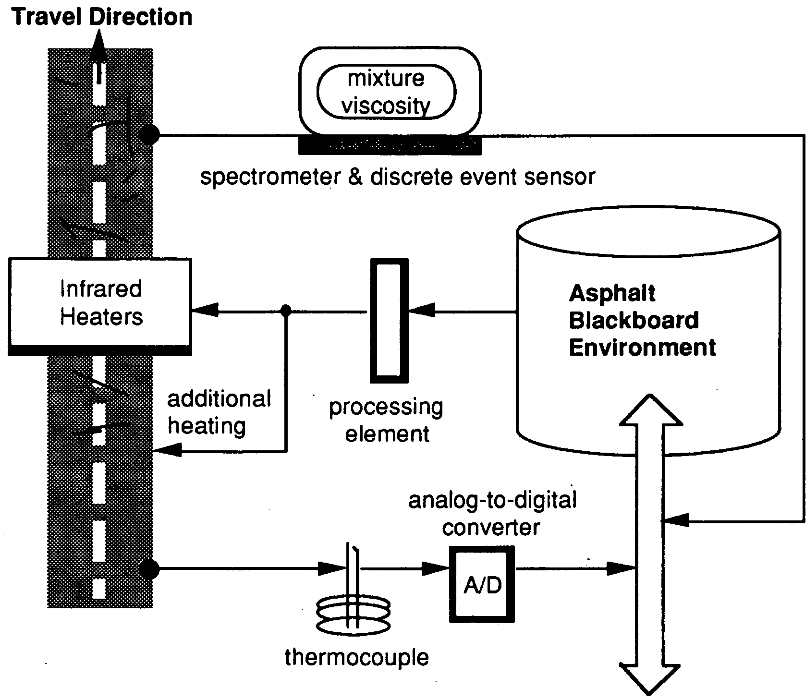

FIGURE 4 Temperature feedback and control for remixer.

compaction is provided to the field supervision group by the blackboard control system, which has interpreted the scenarios that required such actions previously and has chosen the most appropriate recommendations. The system then has the capacity to review the compacted asphalt and decide whether the present conditions of the pavement meet the intended specifications.

Asphalt Knowledge Sources

The knowledge sources are structured to learn from design characteristics integrated into the pavement data base, and the design problems are integrated to complement field uncertainties, operational procedures, and other bottlenecks. For example, design characteris-

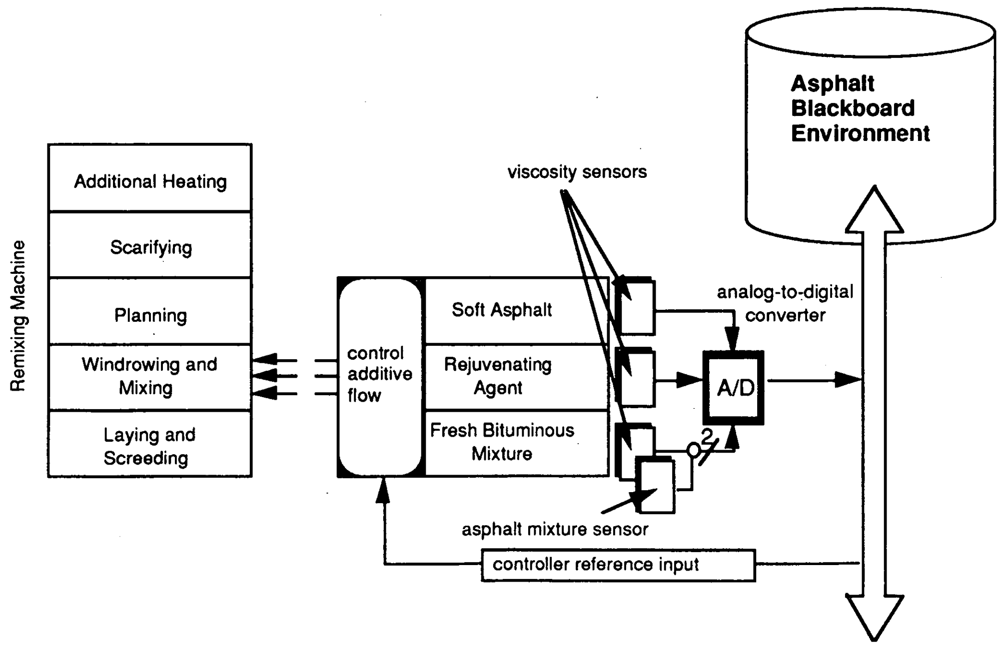

FIGURE 5 Control of reference inputs for asphaltic parameters.

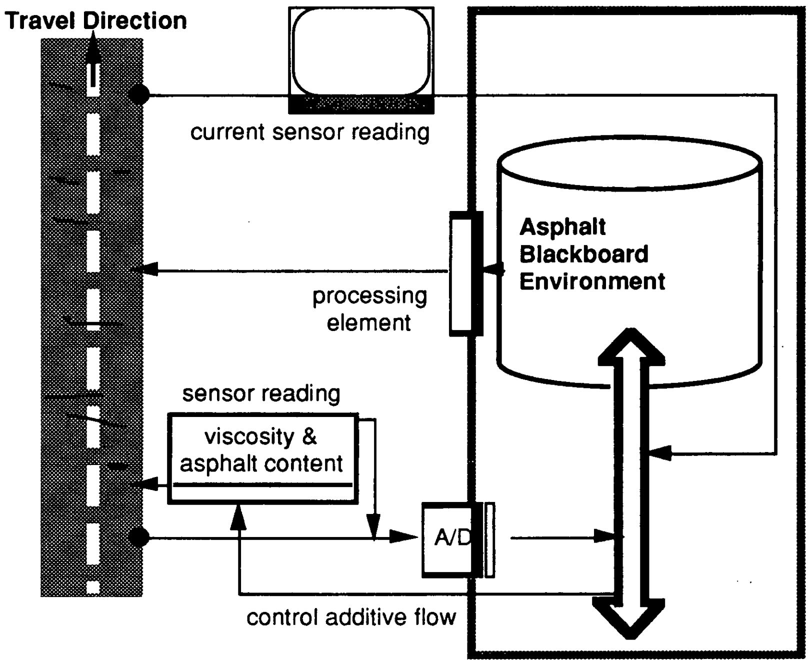

FIGURE 6 Monitoring and control of additive flow.

tics relating to the properties of the material at varying temperatures are integrated to provide essential feedback to system equipment responding to mix, placement, and rolling. The following specific operations are addressed in the asphalt knowledge source:

- A reference, about asphalt hardness: the knowledge source diagnoses heating temperature feedback on the basis of asphalt characteristics such as penetration, viscosity, softening point, and ductility.
- A reference regarding material content: while the mix is prepared, the system maintains an ability to monitor the asphalt being removed from the paved surface, diagnoses its worthiness, projects the requirements of additional materials to achieve optimal design conditions, and controls the mixing process.
- A reference about the finished product being placed on the road surface: the system monitors the placement of the final product with respect to density, composition (asphalt content), and asphalt characteristics.

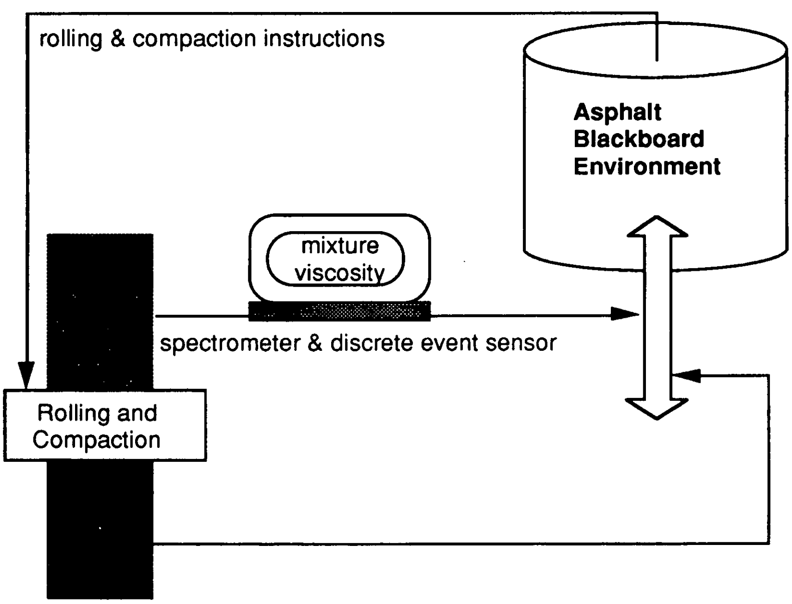

FIGURE 7 Feedback and control of rolling and compaction instructions.

Many organizations have focused on the need to evaluate condition data relevant to the pavement management system (*7*). Several ongoing efforts of the Strategic Highway Research Project have responded to the data acquisition needs of the system, such as the need to focus on material properties, environmental conditions, maintenance and rehabilitation, surface condition, and pavement response (*8*). This information provides a richness of data available to the blackboard. The knowledge made available to the blackboard will only create a system with experience if that information has been based upon field experience.

CONCLUSIONS

This report has presented an alternative approach to conducting asphalt resurfacing. Despite modern hardware and software technologies and significant economic, engineering, and material handling advantages, in-place pavement recycling is not extensively used. Primary concern among pavement engineers is the sensitivity of in-place recycling to the variability in the old pavement. Even with sophisticated equipment technologies, such as the WIRTGEN equipment, the process still relies heavily on laboratory testing of the old pavement and ad hoc evaluation of material mixture during placement, with little feedback on quality control. The novel approach of implementing smart tools through field sensing, field analysis, and feedback control provides a breakthrough in allowing the technology to be used in real time. The potential for its use is immense, especially for applications in which shut downs of the road are practically difficult, such as highway ramps and bridge crossings. The technologies brought together in this paper are still young and expected to evolve. The concept design considerations nonetheless are complete. The breakthroughs in the technology provide a unique opportunity for the highway community to review the process of construction. Through such a review it is expected that this technology will not only improve construction operations by optimizing material handling but also improve product quality and overall economy.

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