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Concrete Pavement Construction and Joints and Loader-Truck Production Studies

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FOREWORD

The 8 papers in this **RECORD** contain timely information on concrete pavement construction, joint sealing for pavements and bridges, cover over reinforcing steel in bridge decks, and determining optimum construction methods. All of these papers will be of particular interest to construction engineers, researchers, and materials engineers. In addition, some of the papers will be of benefit to maintenance, bridge, and pavement design engineers.

Bryden, in his paper on rigid pavement roughness, describes the development of a specification for surface tolerances for new pavements. He found the 25-ft California profilograph correlated highly with the roadmeter and can therefore be used to control pavement roughness during construction to ensure user satisfaction. Specifications were developed relating payment received by the contractor to the riding quality produced. The author reports that these new specifications can be met using current procedures and equipment.

Bryden and Phillips discuss the construction and performance of 5 different concrete pavements using plastic-coated dowel bars in the transverse joints. The plastic-coated dowels were welded or clipped to a basket and staked to the subbase. The researchers found some minor problems with misalignment and damaged coating, but performance in the joints has been good, with no pavement distress. They feel these dowels are promising as load-transfer devices in heavy-duty portland cement concrete pavements.

Brunner, Kilareski, and Mellott report the use and performance of 5 different concrete pavement joint-sealing materials they tried in an effort to reduce pavement failures resulting from incompressibles and water entering through poorly performing joints. They found that an improved rubberized asphalt was better than the conventional rubberized asphalt, that cold-poured component polymers required care in mixing and handling, and that preformed neoprene joint sealers performed well. They also discuss the effects of slab length and joint reservoir shape.

The paper by McBride and Decker describes their work in evaluating joints in concrete pavements that had been in place from 6 months to 10 years. They found that the existing joint seal design practices were inadequate, resulting in overstressing the sealers and premature failure. They recommended wider transverse contraction joints using either preformed neoprene or PVC sealers. Knoblock, in his discussion of the paper, suggests further refinement of the cost figures that would result in an advantage for PVC sealers. McBride and Decker counter that they do not expect significant cost variations from those they reported.

Watson discusses the use of mechanically locked seal elements for bridges to eliminate testing and performance inadequacies of compression seals. He also describes rehabilitation techniques for older structures, armoring joints for protection against snowplows, techniques for direction changes for joint systems, the use of modular systems for large movements, and methods for raising the joint surface elevation for asphalt overlays.

A demonstration project on the construction of a prestressed concrete pavement is described by Brunner in his paper. The project involved construction of 23 slabs 600 ft long keyed together at their ends. Construction details and cost data are given. It is reported that the riding quality of the slabs and joints is excellent. This type of paving requires less concrete through a reduced slab thickness and less steel than conventional reinforced concrete pavements.

Amsler and Chamberlin present the results of the study on the depth of concrete over bridge deck reinforcement. They used a pachometer on 50 bridge deck spans and found that 27.7 percent of the time the minimum cover requirement of 2 in. (51 mm) was not met. They found a relationship between compliances with specification requirements and uniform reinforcement depths, thus indicating the importance of construction practices. Their recommendations included a construction tolerance of $\pm\frac{1}{2}$ in. (13 mm) and tie-down for the reinforcing mat.

The use of time-lapse photography and computer simulation as analysis techniques for determining optimum construction production methods are reported by Willenbrock and Lee. They discuss a data collection method using time-lapse photography and the development of computer simulation using the SIMSCRIPT programming system. They found these to be effective techniques for production studies. They also discuss the method for validation of the model.

—Dale E. Peterson

DEVELOPMENT OF A SPECIFICATION TO CONTROL RIGID PAVEMENT ROUGHNESS

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During a recent study of factors influencing the riding quality of rigid pavement, compliance with the existing roughness specification was found not to ensure a smooth pavement. Because the 10-ft (3.05-m) straightedge used to check the surface can detect only large bumps, the remaining undetected roughness may result in unsatisfactory riding quality. This paper describes the development of a specification to ensure good riding quality in new pavements. The California profilograph was selected as the measurement device because it provides detailed information. Based on results of a subjective panel rating of pavement riding quality in New York State, a project average profile index of 12 in./mile (190 mm/km) and a daily average of 15 in./mile (237 mm/km) are allowed. A limit is also placed on the size of individual bumps. These limits ensure user satisfaction but can be met by paving contractors using current procedures and equipment. Responsibility for controlling roughness during paving is left to the contractor, and the state measures the quality of the completed pavement. To ensure compliance with the specification, the payment received depends on the riding quality achieved. Development of the reduced payment schedule—based on the cost of overlaying the pavement before the end of its design life—is outlined. The years of service expected are related to the initial roughness by means of equations developed in the AASHO Road Test.

•IN 1973, the New York State Department of Transportation completed a study of the causes of built-in roughness of rigid pavements in New York State (1, 2). A number of factors affecting initial riding quality of portland cement concrete pavements were identified, and several changes in design, construction methods, and specifications were made to implement the research findings. That study further found that some pavement being constructed was very rough, partly because of the factors identified and partly because the 10-ft (3.05-m) straightedge used to control roughness during construction was not capable of ensuring smooth pavement.

In 1971, the Department launched a pavement management program (which included an inventory of ridability) to establish maintenance and reconstruction needs and priorities for in-service pavements (3). Because of the emphasis this program placed on pavement roughness and the recognition that the riding quality of a pavement in service depends considerably on initial riding quality, implementation of this research included development of a new specification to ensure acceptable built-in riding quality on rigid pavements. This paper describes the development of that specification.

SELECTING A MEASURING DEVICE AND ESTABLISHING A QUALITY LEVEL

Before deciding on the form of the specification, a device for measuring pavement roughness had to be selected and a satisfactory quality level established. The final

Publication of this paper sponsored by Committee on Rigid Pavement Construction.

research report on roughness (2) included a discussion of 3 measuring devices—the fixed 10-ft (3.05-m) straightedge, the rolling 16-ft (4.88-m) straightedge, and the California profilograph—with the following conclusions:

1. The fixed straightedge is the most economical to buy, the least complex to use, and the only one that can be used on plastic concrete. Its value however, is limited to detecting large bumps during paving, and it cannot adequately control roughness of the finished pavement.
2. Although a rolling straightedge can detect more roughness than a fixed one, it too is still mainly a bump detector and provides only limited information; in addition, it cannot be operated until the concrete hardens.
3. The profilograph is more expensive to purchase and more complex to operate than the other two, but it does provide much more complete information about pavement riding qualities as well as a permanent record of surface profile.

The extra information provided by the profilograph far outweighs its disadvantages, and it was selected as the measuring device for roughness control purposes. The measure obtained with the profilograph is the Profile Index (PI), expressed in inches per mile. The data reduction technique is explained elsewhere (1). The statewide roughness measurements on the existing highway system are made with a Portland Cement Association "roadmeter" and are expressed in terms of Present Ridability Index (PRI), which is a mechanical approximation of a subjective panel rating of pavement riding quality based on an ascending scale from 0 (worst) to 5 (best). Because the initial quality level affects roughness for the entire life of the pavement, these two mechanical measurements had to be related so that initial roughness could be discussed in the same terms as that measured later with the PCA roadmeter.

In the summer of 1973, 30 rigid pavements were measured with both devices. Most test sections were approximately 0.5 mile (0.8 km) long, although a few were limited to 0.4 mile (0.32 km) by intersections or restricted sight distances that made it hazardous to operate the profilograph in traffic. Both measurements were made on a given section within a few days of each other to minimize any effects of weather or subgrade moisture condition. The results are given in Table 1 and shown in Figure 1 along with the regression line relating the two measurements. The data reveal two distinct zones in the relationship. For very low values of PI, the PRI shows little change with an increase in PI. Because of this, pavement sections with a PI less than 7 in./mile (111 mm/km) were not included in the regression analysis.

Although this zone in the relationship may at first seem puzzling, it has a logical explanation. The PRI is a mechanical estimation of the rating a pavement would receive from a panel of highway users. Below a certain level, the panel would no longer be able to discern any appreciable changes in roughness and would rate all such pavements close to 5. The profilograph, on the other hand, is a more precise instrument capable of detecting small differences even at very low levels of roughness. Therefore, while the profilograph reported measurable differences in roughness between four test sections (sites 25, 26, 27, 28), the roadmeter rated all of them very close to 5. For the other 26 test sections, however, PRI decreases as PI increases. Although the relationship shows some scatter, a correlation coefficient of 0.942 was obtained, indicating a close relationship. The 90 percent confidence limits for predicting PRI from PI by use of the regression equation are also shown in Figure 1.

Once the relationship between the profilograph and roadmeter has been determined, a desirable initial riding quality can be selected that will be consistent with both. The California Division of Highways uses a profilograph to judge the acceptability of new pavements and specifies a maximum initial PI of 7 in./mile (111 mm/km). As can be seen in Figure 1, a pavement this smooth would probably receive a rating very close to a perfect 5 by a New York State panel. Although such perfection may be ideally desirable, it could be very expensive and difficult to obtain. The roughness specification used in New York has generally limited surface deviations to ⅛ in. (3 mm) in a 10-ft (3.05-m) straightedge. Based on the preliminary results of this research, a special specification has been used on a small number of contracts. It requires mea-

Figure 1. Correlation of roadmeter and profilograph.

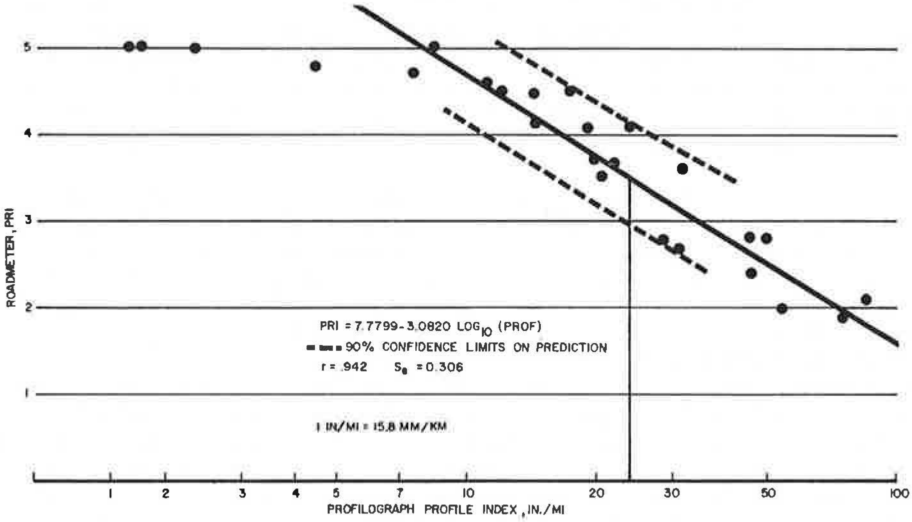


Table 1. Profilograph-roadmeter correlation sites.

		Roadmeter		Profilograph	
Site	Year Built	PRI	Length, miles	Roughness, in./mile	Length, miles
1	1941	2.10	0.52	83.9	0.50
2	1951	3.42	0.51	24.9	0.50
3	1958	3.55	0.53	20.5	0.52
4	1971	4.57	0.52	11.2	0.51
5	1941	3.58	0.50	31.9	0.49
6	1971	4.01	0.51	23.7	0.50
7	1970	4.79	0.52	4.4	0.51
8	1950	3.47	0.51	16.6	0.50
9	1926	2.09	0.40	54.5	0.40
10	1958	2.69	0.50	31.2	0.50
11	1947	2.78	0.51	28.6	0.50
12	1960	3.35	0.40	26.5	0.40
13	1940	2.81	0.51	49.1	0.41
14	1957	4.15	0.51	14.8	0.49
15	1957	4.50	0.54	12.0	0.53
16	1949	4.46	0.50	17.2	0.49
17	1970	4.71	0.53	7.5	0.52
18	1971	3.68	0.43	21.7	0.42
19	1971	4.07	0.51	18.9	0.50
20	1966	4.47	0.49	14.3	0.47
21	1966	4.45	0.50	17.3	0.49
22	1941	2.41	0.51	45.7	0.51
23	1943	1.92	0.53	74.9	0.52
24	1962	3.75	0.50	19.7	0.49
25	1967	5.00	0.51	2.3	0.50
26	1967	5.00	0.52	1.3	0.51
27	1967	5.00	0.52	1.5	0.50
28	1960	5.00	0.51	8.5	0.50
29	1958	4.15	0.51	14.4	0.50
30	1948	2.80	0.52	45.2	0.51

Note: 1 mile = 1.6 km, 1 in./mile = 15.8 mm/km.

surement of pavement roughness with the profilograph, limits the size of bumps on the profilograph trace to ½ in. in 25 ft (13 mm in 7.62 m) and limits the PI to 30 in./mile (474 mm/km). Bump occurrence, however, increases dramatically when the PI exceeds 10 to 15 in./mile (158 to 237 mm/km). The specification thus advises the contractor to strive for a PI below 12 in./mile (190 mm/km) to guard against a large number of out-of-specification bumps. In terms of PI and PRI, 7 of 9 slipformed contracts monitored under that specification were below 12 in./mile (190 mm/km) and above a PRI of 4.5 (Figure 2).

Figure 3 shows the roughness of all pavement samples measured during the research project. These are not completely representative of all paving in the state during that period, since some changes in paving procedures were made deliberately to effect the results. However, overall state results would be similar to these. Although it is evident that achieving a smooth pavement was difficult with form paving equipment, the results with slipform equipment were very good. Most slipform samples were below 12 in./mile (190 mm/km) and had PRIs above 4.

With these historical data and the known relationship between the profilograph and roadmeter, a roughness level to be sought on new construction could be selected. Three major criteria must be satisfied by this value. First, it must be smooth enough so that most highway users would express satisfaction with the riding quality of new pavements. It need not be too smooth, however, because the user cannot discern differences between very smooth pavements. Any extra effort to obtain such very smooth pavement would be wasted. Finally, the level selected must be reasonably obtainable by experienced contractors using present methods and equipment; if not, bid prices would be expected to rise sharply.

The roughness level selected to meet these criteria was 12 in./mile (190 mm/km). There is approximately a 95 percent certainty that the PRI is above 4 for a PI of 12 in./mile (190 mm/km), so most road users would judge that the pavement rides very well, and there would be no dissatisfaction with it. However, 12 in./mile (190 mm/km) is still within the zone where the rating panel can discern differences in riding quality. If the pavement is much smoother than 12 in./mile (190 mm/km), the probability of increased user satisfaction decreases rapidly. At 8 in./mile (126 mm/km), for example, there is only about a 50 percent likelihood that the PRI would be higher than at 12 in./mile (190 mm/km). Going the other way, user satisfaction decreases markedly above 12 in./mile (190 mm/km). For example, at 17 in./mile (269 mm/km), there is less than 50 percent probability that the PRI will be above 4.

Although roughness data presented indicate some difficulties in achieving 12 in./mile (190 mm/km), much of the rough pavement can be attributed to the causes reported in the research study, many of which can be corrected by the changes already implemented. Experienced contractors using slipform equipment thus would have little difficulty in meeting this specification.

In addition to the roughness level of 12 in./mile (190 mm/km) for the entire project average, 15 in./mile (237 mm/km) was selected as a maximum for any particular day's paving. Although experienced contractors can maintain a project average below 12 in./mile (190 mm/km), occasional sections may be rougher due to bad weather, equipment breakdowns, or other unavoidable circumstances. At expressway speeds, a motorist passes over an entire day's paving in less than a minute. A slightly higher roughness level for this short section thus would not have a very unfavorable effect on one's overall impression of the project. At the same time, the contractor is not unnecessarily penalized for what often are unavoidable circumstances.

A maximum limit on individual bumps is important, because large ones are noticed by all highway users and have an adverse effect on their opinion of riding quality, particularly on pavement that is otherwise very smooth. Therefore, a maximum size for individual bumps was set at ½ in. in 25 ft (13 mm in 7.62 m) on the profilograph trace. This limit has been specified on a number of paving projects under the special specification mentioned earlier and, in the opinion of Department engineers, is in the range where noticeable discomfort becomes apparent. Since the profilograph and the 10-ft (3.05-m) straightedge respond differently to bumps of different wavelengths, roughness cannot be compared directly from one to the other. However, a bump of ½ in. in 25 ft (13 mm in

Figure 2. Roughness measured under special specification.

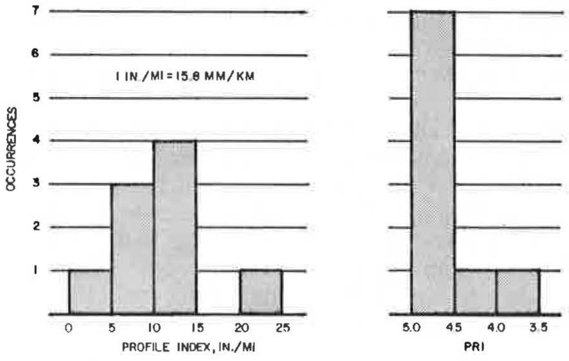
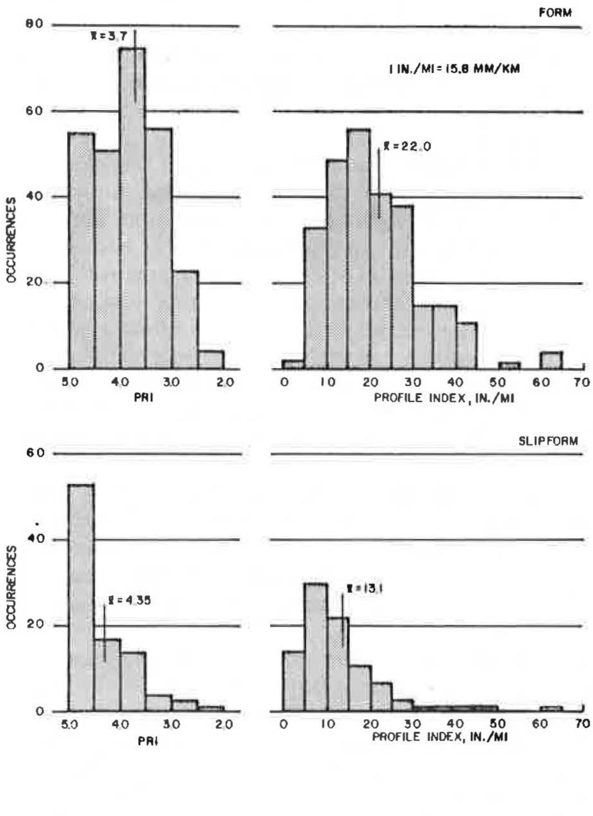


Figure 3. Roughness measured on research project.



7.62 m) on the profilograph trace is roughly equivalent to a deviation of ⅛ in. (3 mm) from a 10-ft (3.05-m) straightedge—the traditional Department specification for rigid pavement roughness. Experience of the Materials Bureau on three projects under the special specification has shown this to be a reasonable value from the standpoint of contractor compliance. On those contracts, less than 2 percent of the pavement had to be corrected to meet this figure.

To summarize, the profilograph was selected as the roughness-measuring device to be used because of the completeness of the data it provides. Average roughness for an entire paving project was set at 12 in./mile (190 mm/km), while the limit for an individual day's paving was raised to 15 in./mile (237 mm/km) to allow for unavoidable circumstances that sometimes result in rougher pavement. The limit for individual bumps was set at ½ in. in 25 ft (13 mm in 7.62 m) on the profilograph trace. These values all ensure a smooth-riding pavement but do not require a pavement smoother than can be appreciated by the user. In addition, these limits can be met by experienced contractors using modern equipment.

SPECIFICATION FORMAT

Once the riding quality level was selected, the next step was to decide on the type of specification to achieve it. Traditionally, the Department has employed a method-type specification—i.e., the contractor is told step by step how to place and finish the pavement. In addition, the finished pavement profile was limited to a maximum deviation of ⅛ in. in 10 ft (3 mm in 3.05 m). Any larger deviations had to be corrected or the contractor was forced to remove and replace the pavement.

That specification, however, has not always yielded smooth pavement in the past. In the first place, the quality level specified, ⅛ in. in 10 ft (3 mm in 3.05 m), did not ensure a smooth ride. Even when maximum bump size was not exceeded, considerable roughness could be present in the form of small bumps. In addition, the present specification has another shortcoming: The primary responsibility for quality control is retained by the state rather than being placed with the contractor. When following a step-by-step specification, the contractor cannot be expected to have complete control over the finished product. At the same time, the state can only try to control those items that are directly covered, and even the most comprehensive specification cannot cover every detail. As a result, control over product quality is not complete, and on occasion rough pavement is built in spite of the best efforts of both the contractor and state forces.

The alternative approach is to place responsibility for finished product quality primarily with the contractor, since he is doing the work and can best control the paving process. The state would protect its interests by placing only general limitations on the methods used by the contractor and specifying an acceptable quality level to be achieved in the finished pavement. A suitable acceptance sampling procedure would ensure that the desired quality level is achieved.

Since the second approach places responsibility for quality control with the contractor, the state must retain some method of ensuring compliance with the specification. The first method would be to remove most process controls but to require correction or removal of all defective material. Correction of pavement roughness by grinding the surface with a diamond cutting tool can achieve fairly good results, but this process is very expensive if more than small areas of pavement are involved (2). Therefore, grinding has not generally proved effective, in New York's experience, for general reduction of average roughness, and its use is generally reserved for correction of individual bad bumps. Complete removal and replacement of the pavement are very expensive and can be justified only in cases of extremely rough pavement.

Pavement built somewhat rougher than the desirable quality level can still provide a number of years of service, although the comfort level is lower and the total years of service would be fewer. Therefore, the second method is to base the contractor's payment on the quality of the finished pavement. This provides a strong monetary incentive to meet the specified quality level but at the same time leaves the contractor relatively free to choose the methods and equipment that he feels will best achieve the desired results. This also answers the question of what the Department should do about pavement of lower quality, since it can now be bought at a bargain price.

Theoretically the reduced payment could be applied to pavement of any riding quality, but it is not desirable to accept very rough pavement at any price. The complaints generated would be very serious if the pavement were too rough, and the available life before resurfacing would be very short; any economic advantage of the lower price would therefore be lost. In this case, the roughness level chosen as an absolute maximum is 36 in./mile (569 mm/km). At that point, the PRI would most likely be between 2.4 and 3.7, with a mean value of 3.0. Certainly, pavement at that level would not feel very smooth to most highway users and would even border on being unsatisfactory in some cases. Therefore, any pavement rougher than 36 in./mile (569 mm/km) will not be accepted and must be removed and replaced by the contractor at no cost to the state.

In conclusion, the most effective means of controlling quality is to make it the responsibility of the contractor. To ensure that he provides the desired quality level, his payment will be based on the riding quality of the finished pavement.

REDUCED PAYMENT CALCULATION

Several approaches were considered in establishing the payment schedule. The easiest would be a completely arbitrary schedule, the only consideration being that the penalty is sufficiently harsh so the contractor will try very hard to comply with the specification. No weight would be given to the amount of reduced comfort experienced by the pavement user or the reduction in pavement life. This, however, has two serious drawbacks. First, because it lacks a rational basis, it is difficult to justify the figures chosen and may not be accepted by the paving industry. Second, the penalty chosen may be either too severe or not severe enough, resulting in either increased bid prices or ineffective roughness control.

The second approach is the opposite: The payment schedule would be based entirely on the degree of comfort afforded the motorist and the pavement life provided. This is completely rational and seemingly completely fair but is very difficult to implement. It is possible to measure the initial ridability and predict the years of service to be provided, but overall quality of service for the life of the pavement is very difficult to predict. Therefore, the reduced payment schedule would still have to be based on some arbitrary assumptions, which would be difficult to derive.

A third approach, used here, bases the reduced payment on the extra cost of rehabilitating the pavement earlier than was assumed in its initial design. This has the advantage of being rationally based on performance of the pavement, with only a minimum of assumptions required to derive the payment schedule. Although it does not consider that motorists using the pavement will be subjected to a rougher ride until the pavement is overlaid, it does consider what may perhaps be the most important problem in pavement management—the expenditure of extra capital construction funds at an earlier date than originally planned. If funds are not available to resurface a pavement when it reaches the terminal PRI, the motorist will be subjected to even greater discomfort.

To compute the payment schedule, the initial riding quality was related to the number of years of service provided before reaching the terminal PRI. The equations developed at the AASHO Road Test (4) were used as follows:

$$P = C_0 - (C_0 - C_1)\left(\frac{w}{W}\right)^{\beta}$$

where

P = PRI at the time in question,

C_0 = initial PRI,
C_1 = terminal PRI,
W = load applications to C_1,
w = load applications to P, and
β = a constant depending on certain pavement characteristics.

For these calculations, the following assumptions were made:

$C_0 = 4.0$
$C_1 = 2.0$
$\beta = 2.0$

To make the equation general to fit any pavement, W was taken as 100 percent and w as a lesser percentage. This is based on the assumption that each pavement is designed to carry 100 percent of its design traffic load before reaching the terminal PRI at the end of the design life. Although actual design traffic load will vary from pavement to pavement, the design thickness is selected to last for the design life of the pavement, regardless of traffic.

Figure 4 shows this equation. The upper solid curve is a pavement starting at a PRI of 4.0 and carrying 100 percent of its design traffic before falling to a PRI of 2.0. The lower broken curve is identical except it started at a lower initial PRI, retained the same vertical offset from the upper curve for the life of the pavement, and reached a PRI of 2.0 before the end of its design life.

Some other assumptions were necessary in these calculations. The design life of a pavement with initial PRI of 4.0 was assumed to be 15 years to the construction of the first overlay at a PRI of 2.0; this is the design life currently used by the Department. The life of the overlay was taken as 8 years, and it was assumed to deteriorate on a straight-line basis. The cost of the original pavement was assumed to be $10/yd^2 ($12/m^2). Finally, the time cost of money was set at 6 percent annually (1.5 percent quarterly).

The calculations involved in deriving the payment schedule for each level of roughness are given in Table 2. For the sake of clarity, these calculations will be explained here for one level of roughness—24 in./mile (380 mm/km). Calculations for this value are underlined in the table. Column 1 lists the measured profilograph roughness, 24 in./mile (380 mm/km) for our example, and column 2 gives the predicted PRI from the regression equation. Referring to Figure 1, we see that the corresponding PRI is 3.53. Column 3 gives the PRI that is 95 percent certain to be exceeded for the particular roughness level, which is equivalent to the lower 90 percent confidence limit in Figure 1. In this case, the value is 3.00. By using this value as the starting point in the analysis rather than the value predicted by the regression equation, we have much greater confidence that the pavement life calculated will be reached or exceeded. Column 4 is the numerical difference between the predicted PRI and 4.0, which for our example is 0.47. Assuming that the y residuals about the regression line are normally distributed, which seems reasonable for these data, we divide the column 4 value by the standard error (0.306) to obtain the value in column 5—1.54 in the example. From a normal distribution table, one can determine the proportion of the total area under the curve below this value. This proportion, appearing in column 6, is 0.9382 in our example. In other words, based on the scatter of data obtained in this correlation, for an initial roughness of 24 in./mile (380 mm/km), the probability is 0.9382 that the PRI as measured by the PCA roadmeter will be below 4.0.

By constructing curves parallel to those shown in Figure 4, one can estimate the percentage of design life (15 years) that would be achieved for any initial PRI. For our example, the dashed curve starts at an initial PRI of 3.0 and results in an expected life of 71.5 percent of the design life (column 7). This percentage is converted to 10.72 years in column 8 and 43 quarters in column 9. Column 10 is the present-worth factor used to express the value of money at the end of the pavement's predicted life as a

Table 2. Reduced payment calculation.

Measured Profile Index, in./mile (1)	PRI Predicted (2)	95 Percent Confidence (3)	4.0 Minus Predicted (4)	Z Statistic (5)	Probability PRI <4.0 (6)	Percent Design Life Expected (7)	Expected Life Years (8)	Expected Life Quarters (9)	Present-Worth Factor (10)
10	4.70	4.15	-0.70	-2.29	0.0110	100.0	15.00	60	–
11	4.57	4.02	-0.57	-1.86	0.0392	100.0	15.00	60	–
12	4.45	3.90	-0.45	1.47	0.0708	99.0	14.85	59	0.4154
13	4.35	3.81	-0.35	-1.14	0.1271	96.5	14.47	58	0.4217
14	4.25	3.71	-0.25	-0.82	0.2061	93.5	14.02	56	0.4344
15	4.16	3.62	-0.16	-0.52	0.3015	91.0	13.65	55	0.4409
16	4.07	3.53	-0.07	-0.23	0.4090	88.5	13.27	53	0.4543
17	3.99	3.45	+0.01	+0.03	0.5120	86.0	12.90	52	0.4611
18	3.91	3.38	+0.09	+0.29	0.6141	83.0	12.45	50	0.4750
19	3.84	3.31	+0.16	+0.52	0.6985	81.5	12.22	49	0.4821
20	3.77	3.24	+0.23	+0.75	0.7734	79.0	11.85	47	0.4967
21	3.71	3.18	+0.29	+0.95	0.8289	77.0	11.55	46	0.5042
22	3.64	3.11	+0.36	+1.17	0.8790	75.5	11.32	45	0.5117
23	3.58	3.05	+0.42	+1.37	0.9147	73.5	11.02	44	0.5194
24	3.53	3.00	+0.47	+1.54	0.9382	71.5	10.72	43	0.5262
25	3.47	2.94	+0.53	+1.73	0.9582	69.0	10.35	41	0.5431
26	3.42	2.89	+0.58	+1.90	0.9713	67.5	10.12	40	0.5513
27	3.37	2.84	+0.63	+2.06	0.9803	65.5	9.82	39	0.5595
28	3.32	2.79	+0.68	+2.22	0.9868	63.5	9.52	38	0.5679
29	3.27	2.74	+0.73	+2.39	0.9916	61.0	9.15	37	0.5764
30	3.23	2.70	+0.77	+2.52	0.9941	59.5	8.92	36	0.5851
31	3.18	2.64	+0.82	+2.68	0.9963	58.0	8.70	35	0.5939
32	3.15	2.61	+0.85	+2.78	0.9973	56.0	8.40	34	0.6028
33	3.10	2.56	+0.90	+2.94	0.9984	54.5	8.17	33	0.6118
34	3.06	2.52	+0.94	+3.07	0.9989	52.5	7.88	32	0.6210
35	3.02	2.48	+0.98	+3.20	0.9993	50.5	7.57	30	0.6398
36	2.98	2.44	+1.02	+3.33	0.9996	48.5	7.27	29	0.6494

Measured Profile Index, in./mile (1)	Present Value of Overlay, dollars (11)	Remaining Overlay Life, quarters (12)	Overlay Salvage Value, dollars (13)	Present Value of Overlay Salvage, dollars (14)	Net Overlay Cost, dollars (15)	Net Cost Times Probability PRI <4.0, dollars (16)	Payment Reduction, percent (17)	Grouped Payment Reduction, percent (18)	Payment Schedule, percent: Entire Project (19)	Payment Schedule, percent: Single Day (20)
10	–	–	–	–	–	–	–			
11	–	–	–	–	–	–	–	0.0	100.0	100.0
12	1.66	31	3.87	1.62	0.04	0.0028	0.02			
13	1.69	30	3.75	1.56	0.13	0.0165	0.16			
14	1.74	28	3.50	1.46	0.28	0.0577	0.58	2.0	98.0	100.0
15	1.76	27	3.37	1.41	0.35	0.1055	1.06			
16	1.82	25	3.12	1.30	0.52	0.2127	2.13			
17	1.84	24	3.00	1.25	0.59	0.3020	3.02	5.5	94.5	98.0
18	1.90	22	2.75	1.15	0.75	0.4606	4.61			
19	1.93	21	2.62	1.10	0.83	0.5798	5.80			
20	1.99	19	2.37	0.99	1.00	0.7734	7.73	9.0	91.0	94.5
21	2.02	18	2.25	0.94	1.08	0.8952	8.95			
22	2.05	17	2.12	0.89	1.16	1.0196	10.20			
23	2.08	16	2.00	0.83	1.25	1.1434	11.43	12.5	87.5	91.0
24	2.11	15	1.87	0.78	1.33	1.2478	12.48			
25	2.17	13	1.62	0.68	1.49	1.4277	14.28			
26	2.20	12	1.50	0.63	1.57	1.5249	15.25	16.0	84.0	87.5
27	2.24	11	1.37	0.57	1.67	1.6371	16.37			
28	2.27	10	1.25	0.52	1.75	1.7269	17.27			
29	2.30	9	1.12	0.47	1.83	1.8146	18.15	19.5	80.5	84.0
30	2.34	8	1.00	0.42	1.92	1.9087	19.09			
31	2.38	7	0.87	0.37	2.01	2.0026	20.03			
32	2.41	6	0.75	0.31	2.10	2.0943	20.94	23.0	77.0	80.5
33	2.45	5	0.62	0.26	2.19	2.1865	21.86			
34	2.48	4	0.50	0.21	2.27	2.2675	22.68			
35	2.56	2	0.25	0.10	2.46	2.4583	24.58	26.0	26.0	77.0
36	2.60	1	0.12	0.05	2.55	2.5490	25.49			

proportion of its present value, based on the 1.5 percent quarterly time cost of money. The overlay cost of $\$4/yd^2$ ($\$4.80/m^2$) at the end of the expected life is multiplied by the present-worth factor to obtain the present value for the overlay given in column 11. In this case, the present worth factor of 0.5262 results in a present value for the overlay of $2.11.

Because an overlay's life is assumed to be 8 years, it may have some useful life remaining at the end of the 15-year analysis period. Based on a straight-line deterioration of the overlay, its remaining life in quarters and salvage value in dollars at that time are given in columns 12 and 13, which in this example are 15 quarters, with a value of $1.87. Column 14 is the present value of column 13, $0.78. Column 15 is the net cost of the overlay in terms of present value, which is simply the initial cost (column 11) less the salvage value (column 14). For this example, this amount is $1.33. Because the predicted life of the pavement was based on the lower confidence limit, there is 95 percent certainty that this cost will not be exceeded if the pavement deteriorates according to the curve in Figure 4.

Using this value as the basis of the reduced payment would provide high assurance of regaining any losses caused by reduced pavement life, but such an approach may be unduly harsh. Column 6 lists the probability that reduced pavement life would occur because of initial PRI less than 4.0. The cost of reduced pavement life can be combined with the chance of its actually occurring to obtain the probable cost to the state. This value, the product of columns 6 and 15, appears in column 16—$1.25 for the example. Column 17 is that cost expressed as a percentage of the original pavement cost, $\$10/yd^2$($\$12/m^2$)—12.5 percent in this case.

To lessen difficulties in administering the specification that might arise from minor measurement and data reduction difficulties, the reduced payment schedule is set up for roughness intervals of 3 in./mile (47 mm/km). To obtain the reduced payment for each interval, the percentages in column 17 were plotted in Figure 5. Since several roundings were applied in the calculations, there are small deviations from the straight line. The actual reduction to be used for each roughness interval was fitted to the line as seen in the figure. The reductions in payment appear in column 18 and the percentage to be paid in column 19. Since daily roughness averages may reach 15 in./mile (237 mm/km) instead of 12 in./mile (190 mm/km), the contract payment schedule was offset by one roughness group to obtain the daily payment schedule in column 20. For the sample calculations, the reduction in payment is 12.5 percent for contract average roughness up to 24 in./mile (380 mm/km), which is equivalent to a payment of 87.5 percent of the bid price. For a single day's paving, roughness up to 24 in./mile (380 mm/km) would receive a 91 percent payment.

SPECIFICATION HIGHLIGHTS AND USE

The main points of the proposed specification are noted here, and three examples are given to show how it will be applied. The specification is an addendum to the New York State Department of Transportation Standard Specifications of January 2, 1973, and contains appropriate references to those specifications[1]. Its main features are as follows:

1. It applies only to main-line paving. Ramps, acceleration and deceleration lanes, and bridge approaches and decks are excluded, since meeting the proposed limits would be very difficult in those areas. Accepting a lower riding quality in those isolated areas is preferable to paying the high cost of meeting the limits.
2. The profilograph for roughness measurements will be provided by the contractor and operated by state personnel. Each day's production will be profilographed in each wheel path after paving, and the contractor will be informed of results, allowing him to take corrective action if necessary.

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[1] The proposed specification is available in Xerox form at cost of reproduction and handling from the Transportation Research Board. When ordering, refer to XS-56, Transportation Research Record 535.

Figure 4. Deterioration in pavement serviceability with traffic.

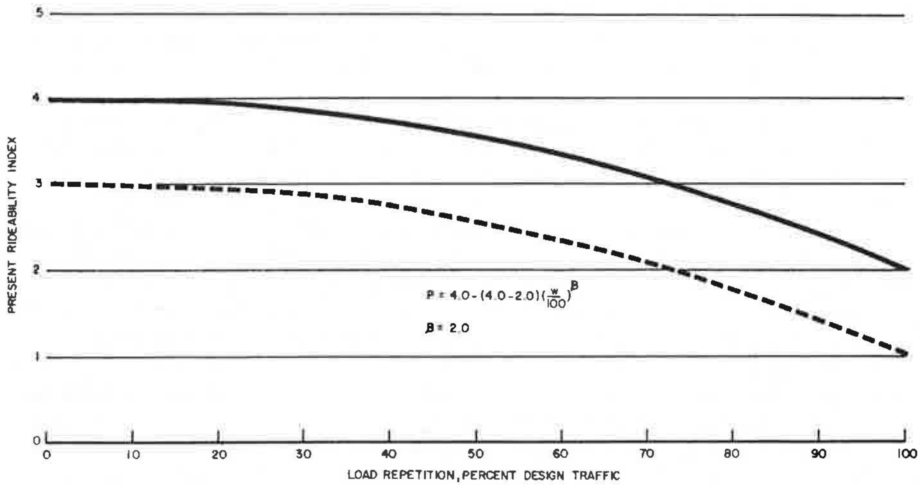

Figure 5. Reduced payment schedule.

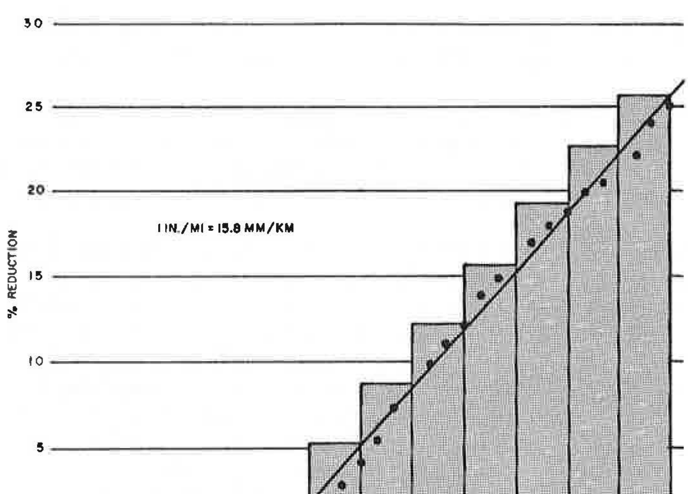

Table 3. Example 1, application of proposed specification.

Day	Final Profile Index, in./mile	Length, miles
1	13.1	0.85
2	12.7	0.76
3	10.5	1.02
4	15.2	0.97
5	18.3	0.35
Total		3.95

Contract average final profile index = 13.33 in./mile (210 mm/km).

Table 4. Example 2, application of proposed specification.

Day	Final Profile Index, in./mile		Length, miles	
1	9.7		0.65	
2	8.6		0.97	
3	17.2	10.3	0.10	0.95
4	9.5		0.85	
5	16.5		0.57	
6	11.2		0.92	
Total			4.06	

Contract average final profile index = 10.87 in./mile (172 mm/km).

Table 5. Example 3, application of proposed specification.

Day	Final Profile Index, in./mile	Length, miles
1	7.2	0.95
2	5.1	0.62
3	10.3	0.73
4	11.4	1.05
5	14.2	1.02
Total		4.37

Contract average final profile index = 10.66 in./mile (168 mm/km).

3. All control of the longitudinal profile during paving will be the responsibility of the contractor. State personnel, however, will continue to check the transverse profile with a straightedge during paving, which must still meet a tolerance of ⅛ in. in 10 ft (3 mm in 3.05 m).

4. Bumps on the profile trace will be checked with a template to determine compliance with the limit of ½ in. in 25 ft (13 mm in 7.62 m). Any bumps exceeding that limit must be corrected by grinding or by removal and replacement of the pavement. After correction, the affected areas will be remeasured with the profilograph.

5. A final profile index is computed for the entire main-line pavement on the project, and for each separate day's paving, after all bumps are corrected. A day's paving of less than 1,000 ft (305 m) will be grouped with the following day for purposes of this specification, to avoid penalizing the contractor for small areas of rough pavement that result from uncontrollable circumstances such as rain or equipment breakdown.

6. The project average profile index must be below 12 in./mile (190 mm/km), and each day's paving must be below 15 in./mile (237 mm/km). If the project average is above 12 in./mile (190 mm/km), all pavement will receive the same reduced payment shown in the specification. If the project average is below 12 in./mile (190 mm/km), each day's profile index must still be below 15 in./mile (237 mm/km). Any day exceeding 15 in./mile (237 mm/km) will receive a reduced payment in accordance with the specification. If any day exceeds 36 in./mile (569 mm/km), the pavement must be removed and replaced at the contractor's expense.

The following examples show how this specification would be applied. The profile indexes used in these examples were measured after all bumps were corrected and are final profile indexes.

In the first example (Table 3), because the final profile index exceeds 12 in./mile (190 mm/km), the contractor would receive a reduced payment for the entire project. The contract average—13.3 in./mile (210 mm/km)—falls between 12.1 and 15.0 in./mile (191 and 237 mm/km), so the contractor would receive a 98.0 percent payment for the entire main-line pavement (Table 2, column 19).

In the second example (Table 4), production for the third day was less than 1,000 ft (305 m) in length, so it was lumped with the fourth before applying the specification. For this example, the contract average is less than 12.0 in./mile (190 mm/km), so the entire contract is not subject to reduced payment. Each individual day must still meet the 15 in./mile (237 mm/km) limit. Day 3, which was less than 1,000 ft (305 m) in length, was combined with day 4. Because the resulting profile index for the 2 days is below 15.0 in./mile (237 mm/km), no penalty results. Day 5, which exceeded 1,000 ft (305 m) in length, has a profile index of 16.5 in./mile (261 mm/km). Therefore, a reduced payment must be paid for that day. According to the specification, the payment for a profile index between 15.1 and 18.0 in./mile (239 and 284 mm/km) for a single day is 98 percent (Table 2, column 20).

In the third example (Table 5), the contract average is below 12.0 in./mile (190 mm/km), and no individual day's average exceeds 15.0 in./mile (237 mm/km). Therefore, the contractor would receive full payment for the entire pavement.

SUMMARY

The research on rigid pavement roughness conducted by the New York State Department of Transportation confirmed that the present specification does not ensure the construction of smooth pavement. A new specification has thus been developed, shifting the emphasis for quality control to the contractor and providing for acceptance sampling of the completed pavement by the state. The California profilograph was selected as the roughness-measuring device, since it provides more detailed information than the 10-ft (3.05-m) straightedge presently specified.

The initial riding quality levels selected were based on considerations of what can realistically be achieved and what is necessary to ensure user satisfaction and reasonable pavement life. Finally, a reduced payment schedule based on the cost of overlaying

rougher payment at an earlier age was selected as the most effective means of enforcing the quality levels specified.

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This paper's contents reflect the author's opinions, findings, and conclusions and not necessarily those of the New York State Department of Transportation or the Federal Highway Administration.

REFERENCES

1. J. E. Haviland and R. W. Rider. Construction Control of Rigid Pavement Roughness. Highway Research Record 316, 1970, pp. 15-32.
2. J. E. Bryden and R. W. Rider. Construction Control of Rigid Pavement Roughness: Final Report. Research Report 16, Engineering Research and Development Bureau, New York State Department of Transportation, Nov. 1973.
3. Explanation and Guide to 1972 Survey Operations and Data of the Pavement Serviceability Program. Special Soils Report SSR-1/72, Soil Mechanics Bureau, New York State Department of Transportation, Oct. 1972.
4. The AASHO Road Test: Report 5—Pavement Research. Special Report 61E, Highway Research Board, 1962.

NEW YORK'S EXPERIENCE WITH PLASTIC-COATED DOWELS

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Because of past difficulties with joint supports in concrete pavements, New York began an investigation of plastic-coated dowels in 1972. This paper describes the construction and early performance of 5 pavements built to satisfy 3 major objectives: first, to identify construction problems related to the dowels; second, to determine if uniform joint movements are maintained; and, third, to determine the long-range corrosion resistance of the dowels. The plastic-coated dowels evaluated have a 2-layer coating of 4 mils (0.1 mm) of asphalt covered by 17 mils (0.4 mm) of polyethylene; they were welded or clipped into basket assemblies and staked to the subbase before paving with a slipform paver. Construction evaluation consisted of observing installation, checking alignment and coating damage, and noting joint cracks after paving. Six dowel samples were removed from the completed pavement for laboratory testing. Joint movement and pavement cracking have been monitored for up to 2 years. Observations and measurements during construction indicate that assemblies of plastic-coated dowels were easy to install and provided satisfactory control of joint crack formation. Some problems with dowel misalignment, damaged coatings, and slippage of coatings off the dowel ends were observed, but these are not considered serious inasmuch as they can be corrected. Performance observations indicate that joints are moving uniformly in all 5 pavements, and no distress has appeared that can be related to the dowels. Based on these observations, plastic-coated dowels show promise as transverse joint load-transfer devices for heavy-duty portland cement concrete pavements.

•STEEL dowel bars have long been used as load-transfer devices in the transverse joints in portland cement concrete pavements. Considerable difficulties have been experienced, however, due to dowel misalignment during construction and dowel corrosion, either of which may lead to premature pavement distress, including midslab cracking, blowups, joint spalling, and faulting. To alleviate these problems, in 1964 the Ohio Department of Highways (1) installed a number of dowel bars coated with a 2-layer system consisting of yellow polyethylene plastic over an inner layer of asphalt mastic, developed by Republic Steel Corporation for gas lines. This coating was intended to provide corrosion protection at a considerably lower cost than the stainless-steel sleeves in use by some states (2) and at the same time to eliminate the need for a grease or oil bond-release agent. The load-transfer capability of these dowels was reported to be nearly as good as plain steel dowels for a total coating thickness of 21 mils (0.5 mm), although thicker coatings resulted in a loss of load transfer (3, 4). Since then, other plastic coatings have been introduced by other manufacturers. New York State has used both dowels and various proprietary load-transfer devices (mainly malleable-iron castings) over the years, experiencing difficulties with both (5, 6).

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Since the plastic-coated dowel seemed to offer an economically attractive means of overcoming the difficulties previously experienced, an investigation of this device was begun in 1972. Plastic-coated dowels were installed in part of one paving contract in 1972 and throughout 4 others in 1973.

This paper describes the construction and early performance of the 5 pavements. The research was intended to satisfy 3 major objectives: (a) to identify construction problems related to the dowels and basket assemblies; (b) to determine if plastic-coated dowels are capable of maintaining uniform joint movements; and (c) to determine the long-range corrosion resistance of plastic-coated dowels in service.

INVESTIGATION

Load-Transfer Devices

The 3 types of load-transfer device under study are shown in Figure 1. The malleable-iron sleeve had been the standard device used by New York from the late 1950s until 1972 and was included as a control in the first test pavement. Both plastic-coated dowels are of the same type, a steel dowel 1⅛ in. (29 mm) in diameter coated with a 4-mil (0.1-mm) asphalt coating and a 17-mil (0.4-mm) polyethylene outer layer. However, the dowels were fabricated into one type of joint assembly by welding and into the other by means of metal clips. All 3 devices were assembled into 12-ft (3.66-m) units and staked to the subbase prior to paving.

Test Pavements

The 5 test pavements (Table 1) all have dual pavements 24 ft (7.32 m) wide. The first—a parkway—is 8 in. (203 mm) thick, whereas the others are 9 in. (229 mm) thick. All were paved with a CMI slipform paver, supplied with central-mixed concrete, over a 12-in. (305-mm) gravel subbase. Sections containing the sleeve devices have joints spaced at 61 ft, 6 in. (18.74 m); the others are spaced at 63 ft (19.20 m). Reinforcing mesh with No. 0 longitudinal wires at 6-in. (152-mm) spacings and No. 3 transverse wires at 12-in. (305-mm) spacings was used in all the pavements, with a clearance of 3 in. (76 mm) provided between the ends of the mesh and the load-transfer devices.

Test Procedures

The evaluation consisted of carefully inspecting the load-transfer devices before paving, observing their installation and the paving, installing pins at the joints to measure joint movements, inspecting the finished joints after paving, and measuring joint widths. Several test sections of 30 joints each have been selected for intensive study, but paving operations and subsequent pavement performance are being monitored for entire contracts. Weather conditions and concrete properties were documented during paving, since they ultimately may affect performance of the pavement. After paving was completed on the first contract, a total of 6 dowels were cut out of the pavement and subjected to laboratory pullout tests.

Semiannual inspections started after paving was completed and will continue for a number of years. These include joint-width measurements, crack surveys, and riding-quality measurements. Joint faulting will be measured if it develops.

Figure 1. Load-transfer devices (not to scale).

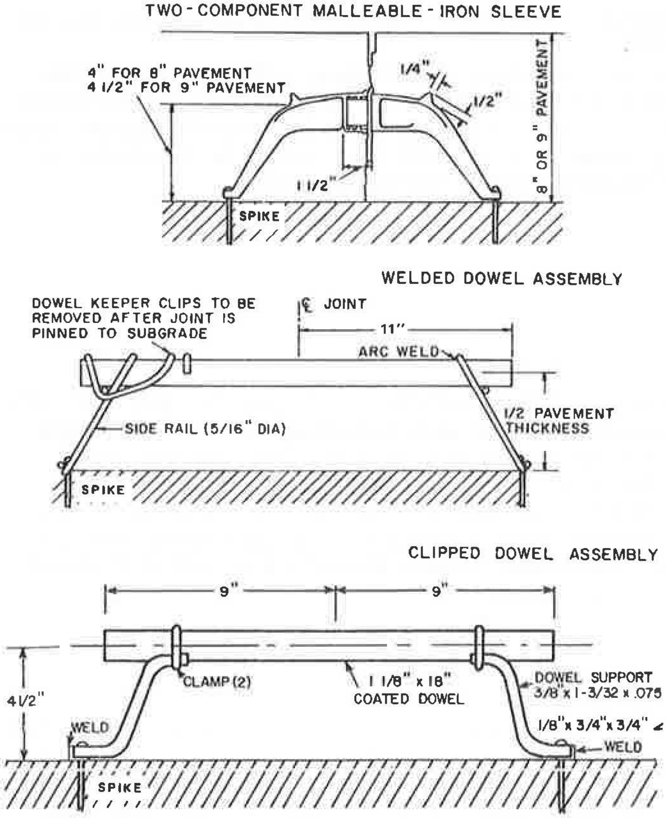


Table 1. Pavement details.

Location	Route	Year Paved	Section	Load Transfer (Figure 1)
Point Breeze	Lake Ontario State Parkway	1972	1	Sleeves
			2	Welded dowels
			3	Welded dowels
			4	Welded dowels
Avoca	Interchange, Southern Tier and Genesee Expressways	1973	–	Clipped dowels
Hornell	Southern Tier Expressway	1973	1	Welded dowels
			2	Welded dowels
Olean	Southern Tier Expressway	1973	1	Welded dowels
			2	Welded dowels
Oneonta	Susquehanna Expressway (I-88)	1973	1	Clipped dowels
			2	Clipped dowels

RESULTS

Installation of Joint Assemblies

Installation of joint supports was observed to determine the time and effort required to align them properly and stake them in place. At Point Breeze, a point was set on the pavement centerline to locate each joint longitudinally. A crew consisting of a foreman and 4 laborers unloaded the devices and set them in place. The 12-ft (3.66-m) sections of either device were handled easily by 2 men. Once set in place, they were aligned by eye to a right angle with the centerline and staked to the subbase. At first J-shaped hooks made from No. 4 (13-mm) reinforcing bars were used as pins, but they were difficult to drive into the compacted subbase, so 12-in. (305-mm) 60-d spikes were substituted. Although much easier to drive, the spikes were still not ideal because their small heads provided relatively little horizontal surface area to grip the transverse wire of the basket. In all, 7 or 8 spikes were used for each 12-ft (3.66-m) section of both joint support types. Considerable caution was necessary in staking the dowel assemblies to avoid hitting the wire basket and damaging it. While the 2-component devices were easier to stake (the base angles are provided with holes for this purpose), this advantage was far outweighed by the inherent instability of the assemblies. Great care had to be taken while handling them to avoid damage, and they had to be carefully aligned after being placed on the grade to ensure that each casting would open and close without binding. After the dowels were staked in place, nails were placed just outside the pavement edges at the center of the dowels to align the sawcut. For the sleeve device, a cotter key was attached to each end of the centerplate, and a wire attached to it ran to the outside of the pavement. After the paver passed, this wire was pulled out to locate the cotter key and thus the ends of the centerplate, marking the position for the sawcut. Although production varied from time to time, the stake-out crew could generally prepare about 25 dowel joints per hour but only 8 or 9 sleeve joints in the same period.

On the other 4 pavements, 3 points were set on the subbase to align each joint—one on the centerline and one outside each pavement edge, making alignment much easier. These devices were pinned to the subbase as on the first pavement. The clipped devices had holes through the base angles for this purpose, which made them the easiest of the three types to install.

Joint Alignment

The joint assemblies were checked carefully before paving for misalignment or other problems; the types of alignment checked are shown in Figure 2. Because the dowels were shop-fabricated into baskets, both transverse spacing and horizontal alignment were extremely consistent. No assemblies were found with any appreciable error in either respect. Vertical alignment, however, did present minor problems. A number of joints were checked on each job, with the results given in Table 2. Generally, no more than 1 or 2 dowels per joint were misaligned, although in a few cases there were several, generally near the ends of the assembly and probably due to rough handling during transportation and installation. The vertical alignment errors detected could have been prevented by more careful handling or corrected before paving, which was effectively accomplished on the 3 contracts having few vertical alignment errors.

At Point Breeze, longitudinal alignment of the dowels was poor. Variations of up to 2 in. (51 mm) were noted in a few instances, and errors of 1 in. (25 mm) occurred in nearly every assembly; this can be seen in Figure 3, which shows a typical joint assembly (one dowel near the center of this assembly is also vertically misaligned). Bowing of the basket assembly (Figure 4) was common at Oneonta and to a lesser degree at the other 3 locations. Both problems resulted in the same defect—decreased embedment length of some dowels.

The joint devices were watched closely during concrete placement for signs of

Figure 2. Dowel alignment errors.

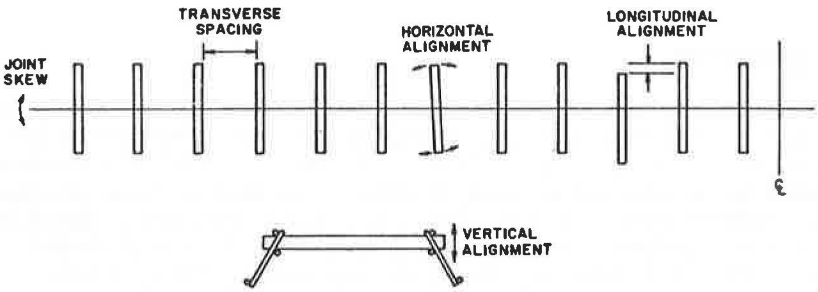


Table 2. Summary of vertical dowel alignment.

Location	Dowels Inspected	Dowel Misalignment, in. ½	¾	1	1¼
Point Breeze	15,720	60	12	1	0
Avoca	720	0	0	0	0
Hornell	1,440	8	1	1	1
Olean	1,440	2	0	0	0
Oneonta	1,334	3	0	0	0

Note: 1 in. = 2.54 cm.

Figure 3. Dowel assembly installed on grade at Point Breeze.

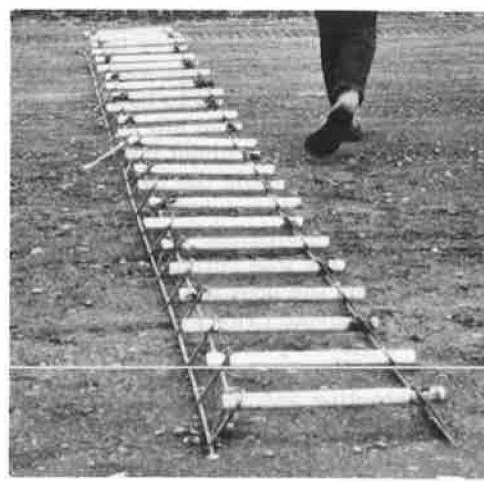
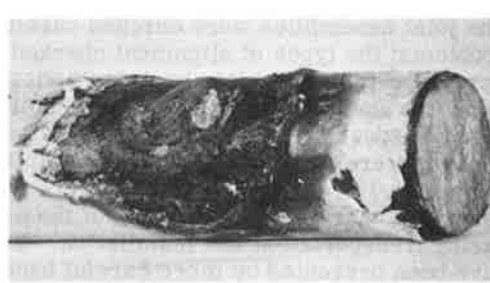


Figure 4. Bowed dowel assembly at Oneonta.

Figure 5. Coating damaged by welding.



pushing or shoving as a result of concrete pressure against the dowels or sleeve devices. At no time was any appreciable movement detected.

Finally, the joints were checked after paving to detect any deviation from a right angle with the pavement centerline. On the 4 pavements where 3 points were used to set the joint alignment, no errors were found. However, at Point Breeze, where only one point was set, alignment errors as great as 13 in. (330 mm) were found. Of 120 joints checked, 33 were skewed less than 1 in. (25 mm), 36 between 1 and 4 in. (25 and 102 mm), 34 between 4 and 7 in. (102 and 178 mm), 12 between 7 and 10 in. (178 and 254 mm), and 6 between 10 and 13 in. (254 and 330 mm). Joints containing sleeves and those containing dowels both were affected.

Since the ends of the centerplate and centers of the dowels were positively located to mark the sawcuts, their alignment corresponds closely to that of the joint support. Therefore, the 13-in. (330-mm) misalignment in a pavement 24 ft (7.32 m) wide is equivalent to a horizontal misalignment of ¾ in. (19 mm) in the 18-in. (457-mm) length of the dowels. However, all dowels in a joint would be parallel.

Coating Damage

A second type of deficiency was damage to the plastic coating, which was most severe on the welded baskets. The weld was designed to be placed approximately 1 in. (25 mm) from one end of each dowel so, when placed properly, only 1½ to 2 in. (38 to 51 mm) of plastic on the end of the dowel was damaged by the weld. However, if the dowel was misaligned in the basket, as was the case at Point Breeze, the damaged coating extended as much as 3 or 4 in. (76 or 102 mm) into the length of the dowel. While most welds were properly placed, a few dowels in each basket at Point Breeze generally approached this extreme. Figure 5 shows a badly burned dowel. While the plastic coating on the clipped dowels was generally in very good condition, a few dowels in most assemblies had approximately 1 in. (25 mm) of coating missing from one end. The dowel bar stock was coated in 21-ft (6.40-m) lengths and the coating shrank about 1 in. (25 mm) on each end. When cut to 18-in. (457-mm) lengths, two dowels were left with short coatings and were painted with red lead primer to inhibit corrosion (Figure 6).

Joint Crack Formation

Since the dowel assemblies did not contain centerplates to control the location and shape of the joint crack, there was some concern that the cracks would not form properly, although each joint was sawed to approximately one-third the pavement depth the night after paving with a diamond-blade concrete saw. Therefore, the time of crack formation and condition of the joint cracks were noted. Because most of these projects were located several hundred miles from the Department's main office, research personnel were not at the job sites on weekends and holidays to check the previous day's placement of concrete for crack development, so only part of the joints were checked. In addition, widths of the shrinkage cracks were determined on three projects by measuring the distances between the joint pins before and after cracking. The results are given in Table 3.

The number of cracks occurring the first night varied from section to section because the weather during paving varied considerably. Sections paved during hot weather, however, cracked completely by the second or third night after paving; in those sections paved in cooler weather a substantial number of cracks appeared by the second day after paving. The time cracks occurred had no apparent effect on initial crack width. Six of the 7 test sections measured had nearly identical widths, between 0.052 and 0.071 in. (1.32 and 1.80 mm), and all 7 were uniform (low standard deviation). Even in those sections where the cracks developed a few at a time over several days, all the joints developed about the same initial width.

None of the pavements experienced problems with cracks occurring before sawing nor with spalling due to early sawing. The lack of a centerplate seemed to be no

Figure 6. Bare dowel ends painted with red lead.

Table 3. Widths of initial joint cracks.

Location	Section	Width, in. $\overline{X}$	Width, in. σ	Percent Cracked First Night	Percent Cracked Second Night	Percent Cracked Third Night
Point Breeze	1	NA	NA	53	NA	100
	2, 3, 4	NA	NA	51	NA	100
Hornell	1	0.062	0.024	80	100	–
	2	0.052	0.016	100	–	–
Olean	1	0.059	0.006	0	100	–
	2	0.037	0.010	70	80	NA
Oneonta	1	0.055	0.017	50	63	NA
	2	0.071	0.015	0	NA	NA

Note: NA = data not available (no Avoca data available).

Figure 7. Pavement sample pullout test.

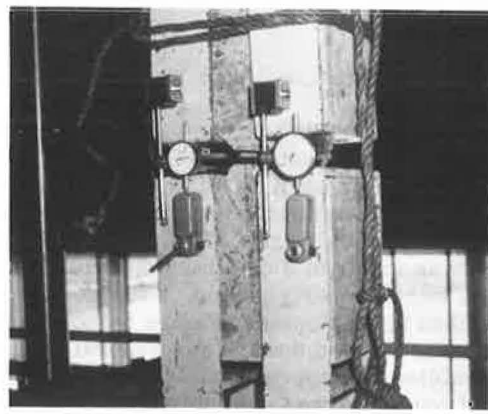


Table 4. Summary of joint movement.

Location[a]	Section	n	Summer to Oct. 1972 $\overline{X}$, in.	σ, in.	ΔT, deg F	Oct. 1972 to Jan. 1973 $\overline{X}$, in.	σ, in.	ΔT, deg F	Jan. to June 1973 $\overline{X}$, in.	σ, in.	ΔT, deg F	June 1973 to Jan. 1974 $\overline{X}$, in.	σ, in.	ΔT, deg F
Point Breeze	1	30	0.118	0.0059	40	0.007	0.0058	2	0.120	0.0106	31	0.165	0.0123	53
	2	30	0.127	0.0059	50	0.016	0.0076	2	0.146	0.0120	31	0.175	0.0124	53
	3	30	0.104	0.0059	45	0.018	0.0062	2	0.112	0.0103	27	0.171	0.0192	49
	4	30	0.137	0.0104	45	0.016	0.0098	2	0.142	0.0131	24	0.177	0.0105	46
Hornell[b]	1	30	–	–	–	–	–	–	–	–	–	0.136	0.0412	37
	2	30	–	–	–	–	–	–	–	–	–	0.117	0.0208	37
Olean[c]	1	30	–	–	–	–	–	–	–	–	–	0.159	0.0162	27
	2	21	–	–	–	–	–	–	–	–	–	0.191	0.0148	27
Oneonta[c]	1	30	–	–	–	–	–	–	–	–	–	0.110	0.0314	44
	2	26	–	–	–	–	–	–	–	–	–	0.085	0.0329	44

Note: 1 in. = 2.54 cm, 1 deg F = 0.55 deg C.

[a]No Avoca data available. [b]Initial reading July 1973. [c]Initial reading August 1973.

Table 5. Pullout test results for Point Breeze dowels.

Sample	Maximum Load, lb	Joint Opening at Maximum Load, in.	Load at ½-in. Joint Opening, lb
2A	740	0.157	560
2B	260	–	220
3A	600	0.065	460
3B	1,000	0.141	640
4A	600	0.147	180
4B	440	0.100	440

Note: 1 in. = 2.54 cm; 1 lbf = 4.448 N.

Table 6. Point Breeze dowel positions.

Sample	Distance From Joint to Fixed End, in.	Depth to Welded End, in.	Depth to Free End, in.
2A	8	4¼	4
2B	7½	4	4½
3A	10	4 3/16	–
3B	8	4½	4½
4A	9½	6	5½
4B	8	3½[a]	3¾[a]

[a]Distance from slab bottom.

disadvantage in crack control, since no cases were found where the crack deviated appreciably from the vertical or from a straight line.

Joint Movement

Widths of transverse joints in the test sections have been measured semiannually since the completion of paving; these results are summarized in Table 4 for all but one pavement, where no summer reading was obtained in 1973. The amount of movement varies somewhat among test sections over similar temperature ranges, but since joint width depends on temperature history before each measurement as well as on exact temperature at the time of measurement, these differences are not surprising. Width changes within each test section are very uniform, as evidenced by the low standard deviations. In comparing the uniformity of width change between test sections, some differences are seen, but they are small and show no advantage for any test section or joint type.

Joint Pullout Tests

At the conclusion of paving at Point Breeze, joint samples were removed from the end of each of the 3 sections containing dowels for testing in the laboratory. The pavement was sawed full-depth with a diamond-blade concrete saw to form a block 2 ft (0.6 m) square along the edge of the pavement. These blocks, containing 2 dowels each, were bolted rigidly across the joint to prevent any movement during removal and shipping. In the laboratory, each sample was cut in half (each half containing 1 dowel) and prepared so the sample could be fitted into jigs for the pullout test (Figure 7).

As the pullout load was applied, joint opening was recorded from dial indicators. The maximum load and joint opening at which it occurred were recorded, as well as the load at the ½-in. (13-mm) joint opening; these are given in Table 5. The loads required to open the joints ½ in. (13 mm) are very low compared to earlier results (5) where loads ranged from 4,100 to 14,000 lb (18 237 to 62 272 N) to open joints containing plain steel and stainless-steel-clad dowels that had been in service several years. After the pullout tests were completed, the concrete was carefully broken apart to examine dowel condition and position. In spite of low pullout loads, the plastic coating was not an entirely effective bond-release agent. The dowels apparently slid inside the plastic sleeve, and the plastic had slipped approximately ¼ in. (6 mm)—half the joint movement—off the free end of the dowel. In addition, concrete around the dowels was stained yellow—further evidence that the plastic had bonded to the concrete. Ohio had reported a similar problem (1) for plastic-coated dowels installed there. Otherwise, the plastic coating was in good condition and showed no signs of abrasion or other deterioration. The distance from the joint face to the fixed end of each dowel and the depth from the pavement surface to the center of the dowel on each end are both given in Table 6.

The design thickness of the Point Breeze pavement is 8 in. (203 mm), and thus the depth to the centerline of the dowels should be 4 in. (102 mm). The depth from the base of the slab is also set at 4 in. (102 mm) by the basket assembly. However, if the pavement thickness varies, more than 4 in. (102 mm) of concrete will be above the dowels, as was the case for sample 4A. Concrete depth above the dowel for sample 4B (from the same joint as 4A) was also considerably greater than 4 in. (102 mm), but the concrete broke in such a manner that it was impossible to obtain an accurate measurement; instead, depth to the bottom of the pavement is reported. Table 6 shows that some of these dowels were poorly aligned—2 by ½ in. (13 mm) and 2 others by ¼ in. (6 mm). This is considerable and surprising, since the dowels displayed good alignment before paving. Large variations from the joint to the fixed end, however, are not surprising, considering the fabrication deficiencies mentioned earlier. For the 18-in. (457-mm) dowels, the joint should have been 9 in. (229 mm) from each end, but instead it was as low as 7½ in. (190 mm).

Pavement Cracking

Since the completion of paving, the test roads have been observed semiannually for signs of pavement cracking or deterioration. Through January 1974, no deterioration other than a few small isolated spalls was noted at any of the transverse joints, and, except at Point Breeze, no midslab cracks have appeared. The first transverse crack at the latter site (paved in July 1972) was noted during the October 1972 survey, appearing as a hairline across both lanes at approximately the one-third point of a slab with doweled joints. By the June 1973 survey, 36 had occurred, all very tight hairline cracks perpendicular to the centerline. Of 101 slabs with sleeve joint supports, 10 had a total of 11 transverse cracks across the driving lane; 2 extended on across the passing lane. Of 1,120 slabs with doweled joints, 25 cracks had formed in 25 slabs, with 15 extending across the passing lane. None of the cracks observed showed signs of movement or broken mesh, and none were faulted.

By January 1974, 760 midslab cracks had developed; all were very tight hairline cracks, usually near either the third point or the center of the slab. In some cases, 2 or 3 were noted in a single slab, although there was generally only 1. None were major structural cracks, and they are attributed to high temperatures during paving. Similar cracking has been noted on a few other pavements in New York, and thus these were not attributed to the use of dowels. Further, pavement sections with sleeve-type joint devices have developed as much cracking as those with dowels.

DISCUSSION AND SUMMARY OF FINDINGS

Based on the information gained from the field test installations, plastic-coated dowel assemblies may prove to be satisfactory load-transfer devices for transverse joints in heavy-duty concrete pavements. Construction problems are minimal and installation proceeds rapidly. Even without a centerplate, sawing the pavement to about one-third its depth has provided excellent control of joint crack formation. In most cases, joint cracks have appeared within 3 days after paving and have been straight and vertical with no spalling or secondary cracking. In addition, initial crack widths have been very uniform, even where all did not occur the same day.

While a few alignment deficiencies have been noted in the dowel assemblies, they generally affected only a small percentage of the dowels, and these could be prevented or corrected. Longitudinal misalignment, however, is an exception, since bowed assemblies and staggered dowels were quite common. This condition, which results in a reduced embedment length, was most severe when combined with weld damage to the plastic coating and could lead to corrosion of the unprotected dowel ends. Careful alignment of the assemblies and careful inspection before paving, combined with improved fabrication techniques, are essential to eliminate these deficiencies. Another potential problem is slippage of the plastic coating on the dowels, as discovered on specimens removed from one test pavement. This slippage could lead to failure of the coating and provide a potential starting place for corrosion.

In conclusion, plastic-coated dowels show promise as a transverse-joint load-transfer device. However, adequate care must be taken to ensure that the dowels are properly aligned before paving. Furthermore, improved fabrication techniques are needed to lessen damage to the plastic coating due to welding, and steps are necessary to lessen slippage of the plastic coating on the dowel. Several years of performance under traffic will be necessary to determine the overall suitability of these devices, but, except for a few correctable deficiencies, it now can be concluded that these devices are satisfactory from a construction standpoint.

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REFERENCES

1. J. C. Dixon, W. J. Anderson, and J. T. Paxton. Investigation of Plastic-Coated Dowels in Ohio. Ohio Department of Highways, June 9, 1970.
2. W. Van Breeman. Experimental Dowel Installations in New Jersey. HRB Proc., Vol. 34, 1955, pp. 8-33.
3. E. A. Broestl. The Deflection of Plastic-Coated Highway Dowel Bars in Concrete. Surface Chemistry Division, Republic Steel Research Center, Jan. 3, 1966.
4. R. C. Best. Plastic-Coated Dowel Bars. Paper presented at Symposium on Portland Cement Concrete Paving Joints, sponsored by U.S. Department of Transportation and Portland Cement Association, Washington, D.C., Sept. 22-23, 1970.
5. J. E. Haviland. Performance of Transverse Joint Supports in Concrete Pavements. Research Report 66-2, Bureau of Physical Research, New York State Department of Public Works, Dec. 1966.
6. J. E. Bryden and R. G. Phillips. Performance of Transverse Joint Supports in Rigid Pavements. Research Report 12, Engineering Research and Development Bureau, New York State Department of Transportation, March 1973.

CONCRETE PAVEMENT JOINTING AND SEALING METHODS

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A comprehensive program to study the performance of concrete pavement joint sealing materials and related practices is being conducted in Pennsylvania. The experimental pavement consisted of combinations of various sealant materials, joint shapes, and slab lengths; a total of 1,020 transverse joints were involved. The materials include conventional "improved" rubberized asphalt, cold-poured, 2-component polymers, and preformed neoprene seals. Various sizes of step-cut joints were tried. These joints have a wider cut at the top to improve the width-to-depth ratio of the sealant. Shorter-than-normal slab lengths were also tested. Preliminary observations indicate that the improved rubberized-asphalt sealant performs better than the conventional-grade material. The cold-poured polymers were found to require careful mixing and handling to obtain satisfactory results. When properly placed, these materials also appear to provide good sealing qualities. The neoprene seals are also performing very well. Preliminary results indicate that the shorter slab lengths offer better joint seal performance and the ½-in. wide × ¾-in. deep joint cut is more satisfactory than the method conventionally used.

•MANY concrete pavement failures can be attributed in some way to poor joint functioning. It is generally necessary to provide joints in a conventional concrete pavement to control shrinkage cracking and allow for thermal changes. For the joint to function properly throughout the life of the pavement, it should be properly sealed to prevent the intrusion of water and debris. Problems that may arise from poorly sealed joints often reduce the life of the pavement and lead to costly maintenance procedures.

The intrusion of incompressible materials into the joint opening is a primary cause of pavement blowups in hot weather (1). This incompressible material restricts pavement slab expansion, resulting in excessive stresses in the slab and eventual buckling or crushing of the slabs at a joint. Slab migration, or translation in a longitudinal direction, has also been attributed partially to joint intrusion. This phenomenon is often called pavement growth and has led to severe damage to structure abutments.

Water intrusion is considered another major cause of deterioration at joints (1). A degradation of subbase support due to infiltration of water may lead to pumping and frost heaving. In the winter, deicing chemicals are also carried into the joint by this water, leading to corrosion of the steel load-transfer devices.

Conventional methods of sealing concrete pavement joints, using hot-poured asphalt sealant, have never been really effective. Bond and cohesion failures generally occur in cold weather, and excessive extrusion of sealant takes place in summer. Annual resealing maintenance is necessary to replace sealant lost by extrusion and snowplowing. Sealant material in the past simply consisted of asphalt with a mineral filler; it

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is still used for resealing in some areas. The most popular material in use at the present time is the hot-poured, rubberized-asphalt sealant. Initially, it gives somewhat better performance than plain asphalt, but it still suffers from the same faults.

Achieving a satisfactory seal of concrete pavement joints has been the goal of many past research projects. Many states have had success with preformed neoprene seals, and their performance has generally been better than other types of joint sealers (2). A variety of cold-poured elastomeric sealers are also available. These more sophisticated materials are generally 2-component mixtures of either the polysulfide or urethane type. A recent summary report of joint seal research in Pennsylvania (3) showed a need for further investigation of improved, hot-poured sealants, neoprene seals, and cold-poured elastomers. Some initial evidence, on the basis of limited research, has also indicated that shorter slab lengths and an improved joint width-to-depth ratio contribute to better performance.

OBJECTIVE

The purpose of this project is to perform a comprehensive evaluation of various types of sealing materials, joint shapes, and slab lengths by using methods that have indicated good results during previous research (4). It is expected that an extensive project such as this will provide meaningful results that may lead to better sealing techniques and performance of the sealed joint. A minimum of 7 years of study is anticipated; interim reports will be prepared as results become apparent.

SCOPE

The test site is located in Pike County on Interstate 84 between Pennsylvania routes 507 and 390. A total of 1,020 joints are involved in the study. Various combinations of material, joint shape, and slab length were used to obtain 51 different test sections consisting of 20 joints each.

Five different types of sealant materials were used:

1. Control—hot-poured rubberized asphalt;
2. Improved—hot-poured rubberized asphalt with upgraded specifications;
3. Urethane—2-component, cold-poured urethane elastomer;
4. Polysulfide—2-component, cold-poured polysulfide elastomer; and
5. Neoprene—preformed, elastomeric seals.

In addition to the normal $^3/_8$-in. (9.5-mm) joint width, three sizes of step-cut joints were used (Figure 1). A fifth size of joint was used for neoprene only. The joint sizes were as follows:

1. Control, $2^1/_2 \times {}^3/_8$ in. (63.5 × 9.5 mm);
2. Step-cut, (A) $2^1/_2 \times {}^1/_8$ in. (63.5 × 3.2 mm), (B) $^1/_2$ in. w × $^3/_4$ in. d (12.7 mm w × 19 mm d);
3. Step-cut, (A) $2^1/_2 \times {}^1/_8$ in. (63.5 × 3.2 mm), (B) $^3/_4$ in. w × $^3/_4$ in. d (19 mm w × 19 mm d);
4. Step-cut, (A) $2^1/_2 \times {}^1/_8$ in. (63.5 × 3.2 mm), (B) 1 in. w × 1 in. d (25.4 mm w × 25.4 mm d); and
5. Step-cut, (A) $2^1/_2 \times {}^1/_8$ in. (63.5 × 3.2 mm), (B) $^1/_2$ in. (12.7 mm) w × seal depth + $^1/_4$ in. (6.4 mm) (neoprene only).

There were 2 shorter slab lengths utilized in addition to the conventional 46.5-ft (14.1-m) joint spacing. They were 38.5 ft (11.7 m) and 31.5 ft (9.6 m).

Figure 1. Step-cut joints.

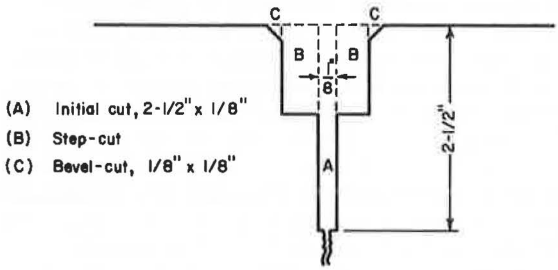

Figure 2. Sealing joint with rubberized asphalt.

CONSTRUCTION PROCEDURES AND EQUIPMENT

Pavement Construction

The concrete pavement was constructed with accepted conventional equipment and procedures. The compacted subbase material was fine-graded with a CMI Autograder that followed alignment and grade set by the offset stringline. The concrete pavement slab was placed on the compacted subbase by a CMI slipform paving train.

Concrete, with an average slump of 1.5 in. (38.1 mm), was placed in slabs 10 in. (254 mm) thick and 24 ft (7.2 m) wide. Pavement slab reinforcement, consisting of wire mesh, was placed in the plastic concrete prior to final strike-off at a depth of 3 ± ½ in. (76.2 ± 12.7 mm). Basket units of slip-bar dowels were staked in place at each transverse joint location for the purpose of load transfer. White membrane curing compound was sprayed on the slab immediately after final finishing of the concrete.

Joint Sawing

All transverse joints were sawed to a depth of 2½ in. (63.5 mm) between 4 and 24 hours after the concrete was placed. The control joints were cut ⅜ in. (9.3 mm) wide and the step-cut joints were initially sawed ⅛ in. (3.2 mm) wide. After the joint was sawed, it was immediately flushed with clean water and a roving material was placed in the saw cut. The longitudinal joint between lanes was sawed within 72 hours after placing the concrete. This joint had a width of 3/16 in. (4.6 mm) and a depth of 2¾ in. (69.9 mm).

The experimental transverse joints consisting of a step-cut joint were sawed in a 2-stage operation. Initially, the step-cut section of the joint was to be sawed immediately after the initial ⅛-in. (3.2-mm) cut; however, the concrete was too green and spalled excessively. The second cut was therefore made after the concrete was allowed to cure for at least 3 days. The wider saw cuts were made with conventional concrete saws. A pair of blades was spaced the required width and centered over the initial cut. All joints of the eastbound lanes had beveled edges, whereas the joints on the westbound lanes were square-cut. These saw cuts were flushed with water and again filled with roving material to keep the joint clean and the joint area moist during the curing period.

Joint Sealing

The joints on the eastbound lanes were sealed within 3 to 7 days after they were sawed. This was done to permit the contractor to use the pavement as a haul road. The joints on the westbound lanes were sealed after the entire section of pavement was placed, because the contractor did not need to use the pavement.

Every joint was cleaned prior to sealing by the following method: First, the saturated rope was extracted from the joint, and any stones, concrete chips, and other debris were removed with a metal-bladed scraper. Next, the joint was blown clean by compressed air, and the saturated rope was returned to most of the ⅜-in. (9.5-mm) joints where the cold-poured sealants were used. The step joints had a bond breaker inserted at the bottom of the channel consisting of either an adhesive-backed sponge-rubber tape or common masking tape. Joint-sealing material was poured immediately after the bond breaker was placed.

Three different types of equipment were used to place the materials in the joints. The control material (hot) and the improved material (hot) were heated in a conventional truck-mounted kettle. This kettle was equipped with a motor-driven pump that forced the material through a hose and hand-held nozzle into the joint (Figure 2).

The 2-component, cold-pour material (polysulfide and urethane) was mixed in a device that blended the material in equal proportions by volume. After the material was mixed, it was pumped through a hand-held nozzle into the joint.

Neoprene seals were installed with a hand-operated machine. Two rollers compressed the seal in front of a third roller that forced the seal into the joint. This third roller was adjustable to any desired depth. Prior to the installation of the seal a lubricant was painted on the edges of the joints.

Instrumentation

Expansion and contraction of the joints are being measured with an Invar bar gauge. Brass plugs were inserted in the plastic concrete on each side of 10 joints in each of 15 test sections. They were spaced 10 in. (254 mm) apart across the joint and 10 in. (254 mm) in from the edge of the pavement.

MATERIALS

Five joint sealers are being evaluated in this project. The conventional, rubberized-asphalt joint sealer conformed to ASTM designations D 1190 and D 1191 and Federal Specifications SS-S-164. The ductility test was performed using molds as described in ASTM D 113 at 77 F and applied at a rate of extension of 5 centimeters per minute.

An "improved", rubberized-asphalt sealer having a higher latex content was used to evaluate material conforming to more stringent specifications. This specification followed ASTM D 1190, with additional requirements and modifications relating to composition and physical properties. The composition was specified as a mixture of virgin synthetic rubber, devulcanized reclaimed rubber, or a combination of the two, with asphalt, plasticizers, and tacifiers that excluded ground cured rubber. The impact test consisted of a swinging hammer that delivered 10 ft-lb of force (13.56 J) to the side of the upper concrete block of a bond specimen turned on its side. This test was performed after the bond test, or, if the specimen failed, a new specimen was conditioned at 0 F for 24 hours and immediately tested. The resiliency test was performed on a sample contained in a 3-oz tin at ambient temperature, 74 ± 2 F (23.3 ± 1.1 C). A steel ball, ⅝ in. (15.8 mm) in diameter, was pushed to a depth of 10 mm and the pressure held for 15 seconds before being released. A new reading was recorded after 30 seconds and subtracted from 100 for the penetration value. The bond test was modified to a ½-in. (12.7-mm) spacing.

Cold-poured, 2-component polymers, one a polysulfide and the other a urethane, were installed to evaluate the performance of these types of materials. The sealants were subjected to the requirements of Federal Specifications SS-S-159b and modified to include water soaking and oven aging. The polysulfide was a thixotropic, nonflowing compound cured with an accelerator introduced and blended into the base compound. The urethane consisted of 2 nonvolatile bitumen-extended elastomers.

Preformed neoprene compression seals $1\frac{1}{4}$ in. (31.8 mm) deep were also installed. The neoprene seals were tested against and conformed to ASTM D 2628.

MATERIAL COST DATA

The costs of the material for a 24-ft lane width are as follows (in dollars):

	Joint Size, in.				
Material	$\frac{3}{8} \times 2\frac{1}{2}$	$\frac{1}{2} \times \frac{3}{4}$	$\frac{3}{4} \times \frac{3}{4}$	1×1	$\frac{1}{2} \times 1\frac{3}{4}$
Control	2.27	0.90	1.32	2.38	—
Improved	3.15	1.24	1.83	3.29	—
Polysulfide	12.07	4.72	7.12	12.63	—
Urethane	5.47	2.14	3.24	5.74	—
Neoprene	—	—	—	—	24.19

These are the costs of the sealant material only. Labor costs involved in the construction of the joints would be additional. Joint cutting and bevel cutting would add to the cost of the step-cut and neoprene joints due to the added sawing operations involved. Another cost involved with the step-cut joints would be the labor required to place the bond-breaker tape in each joint prior to sealing. Cost data for these operations were not available. The primer for the polysulfide sealant and the lubricant-adhesive for the neoprene seals are included in the cost data but not the labor to apply them.

FIELD OBSERVATIONS

Control Material

The control sealant was very susceptible to the intrusion of debris, especially in the step-cut joints. A bond breaker of either masking tape or sponge-rubber tape was used in the bottom of all wide joints.

Improved Material

The improved material was much less susceptible to the intrusion of debris. The installation procedure was the same as for the control. Bond breakers were again used in the wider joints.

Polysulfide Material

The primer was applied by hand with an ordinary paint brush. Occasionally the primer turned white and powdery because of excessive moisture in the concrete and repriming was necessary. The mixing procedure for the polysulfide sealer was critical and required a positive means of thoroughly blending on a 1:1 ratio to ensure proper curing. Some joints with partial cure did result from poor mixing, and these required removal and replacement. The mixing and placing equipment was less than ideal and required close attention to maintain a uniform mixture through the application nozzle. A mechanical failure occurred after the first 3 joints were completed, requiring manual operation of the machine. The proportioning was not satisfactory until a replacement part was obtained. A bond breaker of either masking tape or sponge tape was used in the bottom of the wider joints. In several instances the masking tape turned or became embedded in the sealer and did not provide the necessary seal at the bottom of the joint. This allowed the uncured material to flow under or around the bond breaker and required additional applications to obtain the proper level and seal. The use of masking tape was discontinued in favor of the sponge-rubber tape, which performed satisfactorily when installed correctly. The consistency of the material allowed placement on superelevation with a minimum of flow before "set" occurred (Figure 3). The cure time varied from 10 minutes at 95 F (35 C) to 24 hours at 60 F (15.6 C). Where additional applications were required, the material bonded to itself satisfactorily.

Urethane Material

The mixing of the urethane material appeared to be more critical than with the polysulfide. This judgment was based on the wide range of curing times observed. Some joints had not achieved permanent set even after 3 months. The appearance would indicate that set had occurred, but when the surface film was broken the material underneath was in a semicured condition.

The first portion of the test application was completed with the same equipment used for the polysulfide sealer. After the first half of the project was completed and many of the joints remained in a semicured condition, further use of the machine was discontinued. A concerted effort was made by the supplier of the sealant to determine why the material did not cure. One possibility suggested was that moisture had contaminated the stored material, because it was allowed to remain exposed to the elements. The remainder of the shipment was therefore replaced with new material. A portable mixing device, consisting of a motorized turntable with a mixing paddle to blend the 2 components, was also supplied with this new shipment. This operation, which required hand-proportioning and pouring from "watering cans", gave decidedly poor results. The joints were characterized by a wide range of curing time and excessive bubbling of the sealant. Again, further use of the material was suspended. Another effort was undertaken to determine why the variations had occurred. The supplier felt that the problem was caused by improper mixing and procured another machine.

A small, portable, air-operated machine was supplied. It was capable of high-speed, airless mixing and proportioned the materials prior to application through a small hose and nozzle. A fast cleaning method was also possible if the operation should be halted for any period of time.

The remaining joints were completed with a uniform range of curing time. Sealant placed with this machine was characteristic of typical urethane materials (Figure 4). The sponge bond breaker was again used in most of the wider joints. Roving material was placed in the bottom of the conventional joints and a light application of sealer was applied and allowed to cure partially before the joint was fully sealed. Areas that required resealing did not bond as well as expected to the previously placed material. The urethane material flowed slightly on superelevated sections.

Neoprene Material

The lubricant-adhesive machine was inoperative and hand application was required. The machine used to place the seal was fast, efficient, and provided a uniformly placed seal (Figure 5). When a problem occurred, such as a twisted or deeply placed seal, the machine could be backed up and the nonuniform area could be extracted and reinserted with a minimum of effort. The ease of installation was superior to all other sealers placed during this evaluation (Figure 6).

Figure 3. Polysulfide sealant (½ x ¾ in.).

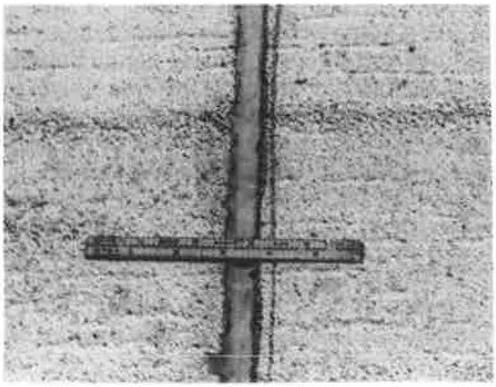

Figure 4. Urethane sealant (¾ x ¾ in.).

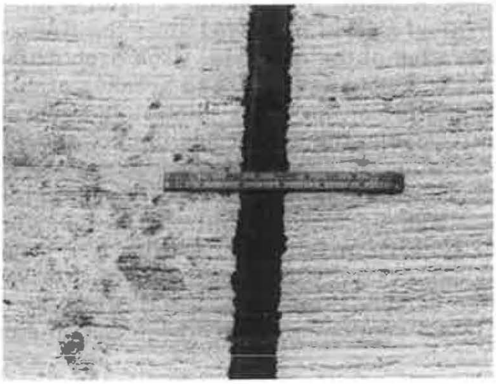

Figure 5. Preformed neoprene seal installer.

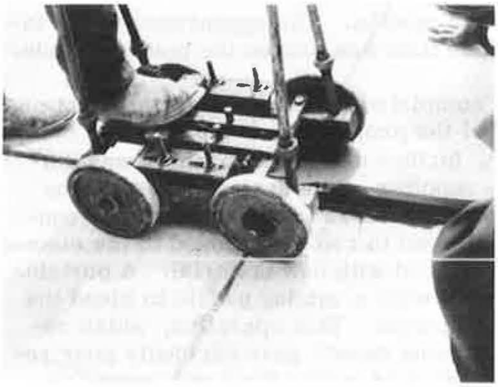

Figure 6. Completed preformed neoprene seal.

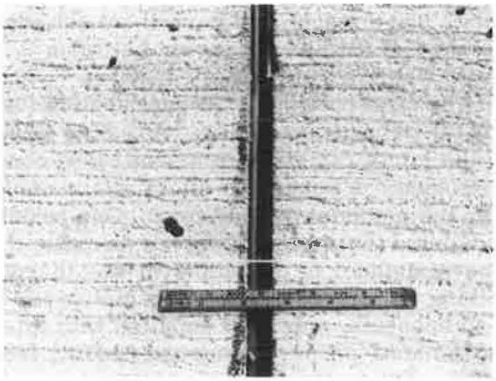

Table 1. Rating levels.

Rating	Degree	Description
Sealing		
5	None	Seal is intact and in the condition as constructed.
4	Slight	Seal has experienced adhesion, cohesion, and/or spalling defects in less than 5 percent of the joint area.
3	Moderate	Seal has experienced adhesion, cohesion, and/or spalling defects in less than 50 percent but more than 5 percent of the joint area.
2	Severe	Seal has experienced adhesion, cohesion, and/or spalling defects in less than 50 percent but more than 25 percent of the joint area.
1	Deteriorated	Seal has experienced adhesion, cohesion, and/or spalling defects in more than 50 percent of the joint area.
Weathering		
5	None	Seal is intact and in the condition as constructed.
4	Slight	Seal surface aged or oxidized.
3	Moderate	Seal surface has weather checking.
2	Severe	Seal surface has alligator cracking.
1	Deteriorated	Seal surface has eroded.
Debris Intrusion		
5	None	Seal is intact and in the condition as constructed.
4	Slight	Seal is intact and in the condition as constructed with debris accumulated but no intrusion.
3	Moderate	Seal has accumulated debris with scattered intrusion.
2	Severe	Seal has accumulated debris with much intrusion.
1	Deteriorated	Seal is broken and eroded by excessive intrusion of debris.

CONDITION SURVEY

A rating system was devised to evaluate the performance of the joint sealing materials. The method requires that each joint be appraised for its sealing, weathering, and debris-resistant characteristics. The level of performance is rated on a scale of 1 to 5, with 5 as maximum, as described in Table 1.

The first field performance evaluation was accomplished on November 15-16, 1973. A summary of the data accumulated is given in Table 2. Each value shown is the average obtained for 20 joints (Figure 7).

The control material was performing poorly in all joint size configurations and in all 3 pavement slab lengths (Figure 8). There was an excessive intrusion of debris into the material in the wider joint openings. Bond failure is prevalent between pavement joint and sealant (adhesion) and to some extent within the material itself (cohesion).

The improved rubberized-asphalt material was much better in appearance. It sealed the joints and rejected the intrusion of debris better than the control material (Figures 9 and 10). There was a minimal amount of debris even in the $^3/_4$- and 1-in. joints.

Both of the poured sealants, polysulfide and urethane, are performing significantly better than the improved rubberized-asphalt material. The polysulfide has a very slight edge over the urethane material in overall performance. The sealing, weathering, and debris-resistant characteristics of these materials are excellent thus far. As mentioned earlier, some difficulty was experienced with curing in some of the urethane joints. These joints will not be discarded from the test, but they have been identified in Table 2 with an explanation of their lower-than-normal performance.

The overall performance of the preformed neoprene seals was rated as superior at this time. Although some intrusion of debris was apparent between the neoprene seal and the joint face, this was attributed to a poor construction practice.

The practice of allowing construction equipment and traffic to use the pavement after the joints are sealed has led to poor joint performance. The shoulder placement and final clean-up operations have spalled the joint walls as a result of aggregate being "bladed" across or from the pavement surface (Figure 11). At the same time, debris is forced into the joint opening, and, if the sealer lacks characteristics to repel this intrusion, "solid" contamination occurs. A high joint failure rate can be expected from this practice. The occurrence of such damage on this job has served to lower the overall performance ratings.

The initial condition survey has indicated the following levels of performance (in decreasing order):

Material	Joint Spacing	Configuration
Neoprene	31.5 ft	$^1/_2 \times {}^3/_4$ in.
Polysulfide	38.5 ft	$^3/_4 \times {}^3/_4$ in.
Improved	46.5 ft	1×1 in.
Urethane		$^3/_8 \times 2^1/_2$ in.
Control		

RECOMMENDATIONS

The data collected and observations made thus far would seem to justify the following statements:

1. The specification for joint sealing material should be upgraded to require the use of a sealer equivalent to the improved rubberized-asphalt material.
2. Consideration should be given to the potential savings that might be achieved in going from the conventional joint (\$2.27 to fill a $2^1/_2 \times {}^3/_8$-in. joint with conventional hot-pour sealer) to a $^1/_2 \times {}^3/_4$-in. joint (\$1.24 to fill with improved rubberized material). It would require some additional cost to make the second step-cut.

Table 2. Summary of joint ratings.

Section	31.5-ft Joints Seal	Weather	Debris	Average	38.5-ft Joints Seal	Weather	Debris	Average	46.5-ft Joints Seal	Weather	Debris	Average
C-1	3.7	3.0	3.0	3.2	3.2	3.0	3.0	3.1	1.4	2.1	1.8	1.8
C-2	3.0	3.0	2.1	2.7	2.3	1.9	2.0	2.1	3.2	3.0	3.0	3.1
C-3	2.0	3.0	1.4	2.1	3.5	3.0	3.0	3.2	2.8	2.0	1.7	2.2
C-4	3.0	2.0	1.4	2.1	2.7	3.0	2.4	2.7	3.8	2.0	2.4	2.7
I-1	3.9	3.0	3.0	3.3	3.4	3.0	3.1	3.2	2.7	3.0	3.1	2.9
I-2	3.5	3.0	3.0	3.2	3.5	3.0	3.0	3.2	3.5	3.0	3.0	3.2
I-3	3.5	3.0	2.9	3.1	3.4	3.1	3.0	3.2	3.9	3.0	2.9	3.3
I-4	4.0	3.0	3.0	3.3	3.3	3.0	3.0	3.1	3.7	3.0	3.0	3.2
P-1	4.1	4.0	4.7	4.3	3.3	4.0	3.8	3.7	2.6	4.0	4.2	3.6
P-2	4.3	4.0	5.0	4.4	3.3	4.0	5.0	4.1	3.1	4.0	4.7	3.9
P-3	4.2	4.0	5.0	4.4	3.7	4.0	5.0	4.2	4.0	3.7	4.0	3.9
P-4	4.2	4.0	4.4	4.2	3.5	4.0	4.9	4.1	3.6	4.0	3.9	3.8
U-1	4.1	4.0	4.4	4.2	3.1	4.0	4.4	3.8	1.7	2.9	2.2	2.3[a]
U-2	3.1	3.5	4.9	3.8	1.5	2.2	2.0	1.9[a]	1.0	3.0	3.0	2.3[a]
U-3	1.9	2.0	2.2	2.0[a]	3.1	4.0	5.0	4.0	4.4	3.0	4.6	4.0
U-4	2.7	2.7	2.1	2.5[a]	1.9	2.1	1.6	1.9[a]	3.7	3.0	3.1	3.3[a]
N-5	5.0	5.0	3.5	4.5	5.0	5.0	3.5	4.5	5.0	5.0	3.3	4.4

[a]Either part or all of the joints in this section suffered from improper curing due to poor mixing of the two components.

Figure 7. Field rating sheet.

46.5-Ft. Joints

Rating \ Joint	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
Sealing																				
Weathering																				
Debris																				

Section____

Date______

Average Rating______

38.5-Ft. Joints

Rating \ Joint	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
Sealing																				
Weathering																				
Debris																				

Average Rating______

31.5-Ft. Joints

Rating \ Joint	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
Sealing																				
Weathering																				
Debris																				

Average Rating______

Figure 8. Control joint showing early failure.

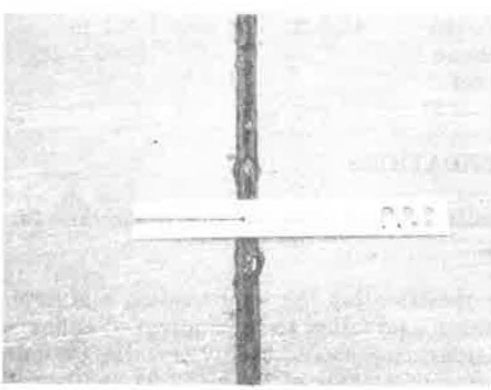

Figure 9. Debris intrusion in control material (1 x 1 in.).

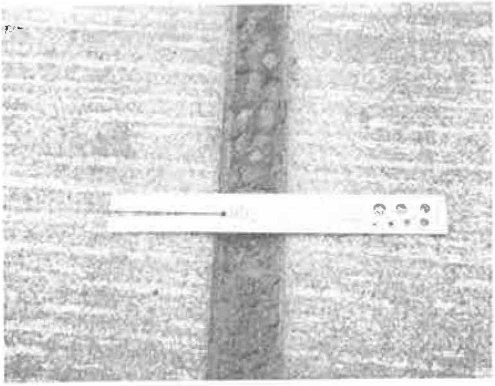

Figure 10. Minimal intrusion in improved rubberized-asphalt material (1 x 1 in.).

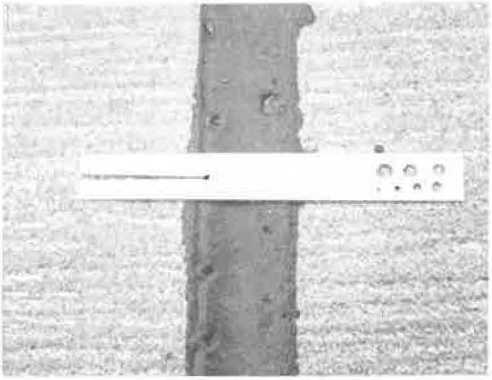

Figure 11. Construction damage to completed urethane sealant.

3. Consideration should be given to specifying the joint sealing operation as the last construction operation on a new project. The use of step cutting would allow the first cut to be made at the required time and roving material could be maintained until the second cut is completed.

4. Inspection of the joint sealing operation should receive a high priority to ensure the construction of a quality joint seal.

REFERENCES

1. J. P. Cook and R. M. Lewis. Evaluation of Pavement Joint and Crack Sealing Materials and Practices. NCHRP Report 38, 1967, pp. 2-3.
2. D. D. Brown. Effects of Various Sealing Systems on Portland Cement Concrete Joints. Highway Research Record 389, 1972, pp. 28-39.
3. R. J. Brunner and D. B. Mellott. A Summary Report on the Installation and Performance of Pavement and Bridge Joint Sealants. Pennsylvania Department of Transportation, Nov. 1973.
4. D. B. Mellott. Experimental Joint Sealants: Five Products. Research Project No. 68-19, Pennsylvania Department of Transportation, Feb. 1970.
5. Joint Sealing: A Glossary. HRB Spec. Rept. 112, 1970.

PERFORMANCE EVALUATION OF UTAH'S CONCRETE PAVEMENT JOINT SEALS

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During the summer of 1972, Utah experienced the first major pavement distress in its concrete pavement Interstate highways in the form of pavement blowups. Subsequent investigation indicated that these resulted from poor construction and repair, which allowed contraction joints to be filled with incompressibles. Growing concern about more widespread pavement distress led to additional visual inspection and joint corings. Six sections were chosen, ranging in age from 6 months to 10 years, from which cores were taken, and it was found that all but the most recently sealed joints had seal failures, even in those that were only $1\frac{1}{2}$ years old. When it was determined that seal failure was so common, the designs were reviewed, and it was found that the present seals are overstressed. It was recommended that either a $\frac{7}{16}$-in. preformed seal be installed in a $\frac{1}{4}$-in.-seal reservoir or a PVC hot-pour seal in a $\frac{3}{8}$-in.-wide joint be used instead of the present design. Other observations showed that the longitudinal joint at the pavement edge was in poor condition and needed resealing but that the longitudinal centerline joint was in good condition.

•DURING the summer of 1972 on I-15, between the 31st Street Exit in Ogden and the Layton Exit, 2 blowups occurred in the concrete pavement. These major pavement distress problems were the first such problems to occur on the state's Interstate highways since the concrete-paved sections were opened to traffic approximately 10 years ago. At the time of construction the contractor had failed to remove the wooden bulkheads laid at the end of each day's paving when he started to place concrete the next morning. When the pavement was nearly completed, the wooden bulks, which had been left in the pavement, were noticed. In removing the bulkheads the pavement was only partially cut and the remaining depth of pavement was broken with a jackhammer. Instead of having a vertical break, the remaining slab was undercut as in Figure 1a. The gap was then filled with expansive concrete containing iron filings. Several years later the expansive concrete had started to deteriorate rapidly due to rusting of the metal filings, leaving a depression in the pavement (Figure 1b).

Plans were made to remove the deteriorated concrete, but before this could be done the pavement blew up at the construction joints (Figure 1c). During the investigation of these blowups it was observed that the joint seals in this area had failed and the joints were being infiltrated by incompressibles. With the joints infiltrated the horizontal stresses resulting from thermal changes could not be relieved. When the forces became large the pavement was pushed up the ready-made ramps at the construction joints.

After these blowups were examined and their cause was determined, additional investigation to evaluate the condition of the concrete pavement in other areas was proposed. The proposed study objectives were to

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Figure 1. Repair, deterioration, and blowup sequence.

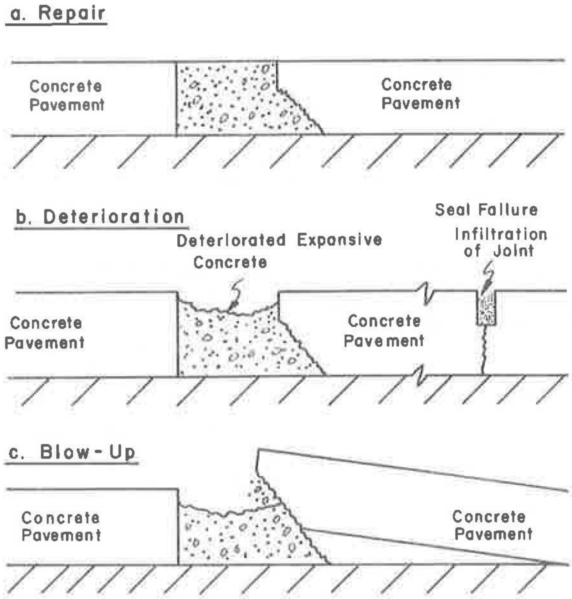


Figure 2. Aggregate interlock.

Figure 3. Joint location for 48-ft roadway.

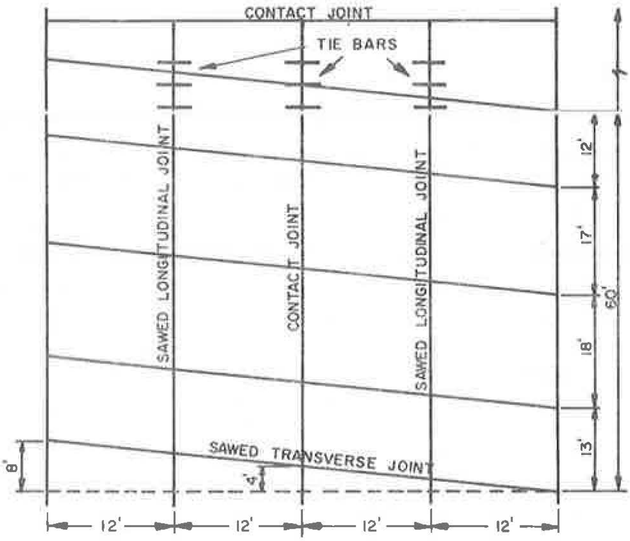
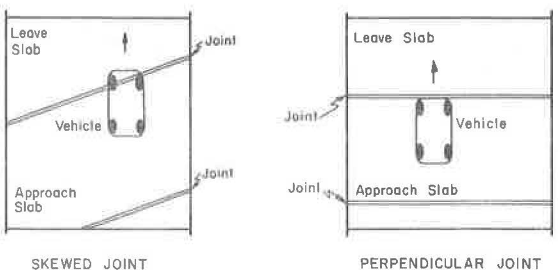


Figure 4. Vehicle crossing skewed joint versus perpendicular joint.



1. Evaluate existing pavement joints and determine if seals are still effective and if the joints are being filled with incompressibles,
2. Determine if there is pavement growth,
3. Determine if bridges are being pushed by the pavement, and
4. Formulate recommendations for action.

This report covers the results of the study.

PROJECT DESCRIPTION AND FINDINGS

The purpose of concrete pavement joints is to control cracking, to accommodate movements caused by changes in temperature and moisture, and to facilitate construction. After the joint is designed and built, its preservation as a working component of the pavement is important. Sealing is the means by which preservation is attempted. If for some reason the seal fails, either through poor design or construction, the joint will begin to be infiltrated with incompressibles and water. Continued infiltration will result in pavement distress in the form of raveling, spalling, faulting, pumping, and blowups.

To evaluate potential future pavement distress through joint seal failure, a literature review, visual inspection, and joint coring program were conducted.

Present Paving Practice

Plain concrete pavements are used in Utah, with tie bars used only along the longitudinal joint.

Aggregate interlock, which is the simplest means of load transfer, is used in transverse joints (Figure 2). The effectiveness of aggregate interlock varies inversely with the joint openings, so the shortest practical slab length is therefore desirable. To keep the slabs at the shortest practical length, random joint spacings of 12, 17, 18, and 13 ft (3.6, 5.2, 5.5, and 4.0 m) are used (Figure 3). This random spacing breaks up the resonance that can be created by vehicles when a uniform 15-ft (4.5-m) joint spacing is used. The randomly spaced joints are cut on a 1:6 skew (Figure 3). Skewed joints have the advantage of reducing stresses in the impacted corner and help reduce the corner cracking that used to be prevalent with older narrow pavements. The impact on the leave slab is reduced by causing the wheel axles to be more gradually applied to the leave slab and one at a time rather than to have the entire axle load "fall" onto the leave slab, as is the case with perpendicular joints (Figure 4). Overall, the skewed joints provide a smoother ride to the traveling public.

The concrete slabs are supported on a cement-treated base material. Figure 5 shows a typical section. It seems significant that western states (1) have been employing cement-treated bases and aggregate interlock joints with success, whereas such joints have not proved durable in other areas where untreated bases were used.

Once the transverse saw cuts of $\frac{1}{8} \times 2\frac{1}{2}$ in. (0.31×6.3 cm) are made, the joint is sealed. The joint is first cleaned with compressed air and then filled with a hot-poured seal meeting Federal Specification FSS-SS-S-164.

Joint and Seal Evaluation

Concerning pavement distress in the form of pavement blowup (one of the most spectacular forms), Stott and Brook (2) describe the mechanism by which the blowups may occur: "This theory may be developed in detail. It supposes that material infiltrates into open joints during the winter months either from the upper surface of the road, from material in the base, or from dislodged material in the joint itself. This material settles at the bottoms of the joints due to gravity. The material creates local points of contact between the opposite faces of the joints when the joints close in sum-

Figure 5. Typical cross section of roadway with treated base.

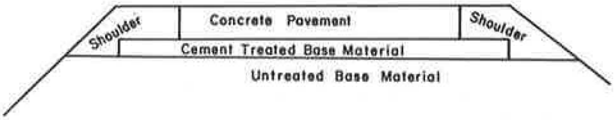


Figure 6. Mechanism for pavement blowup.

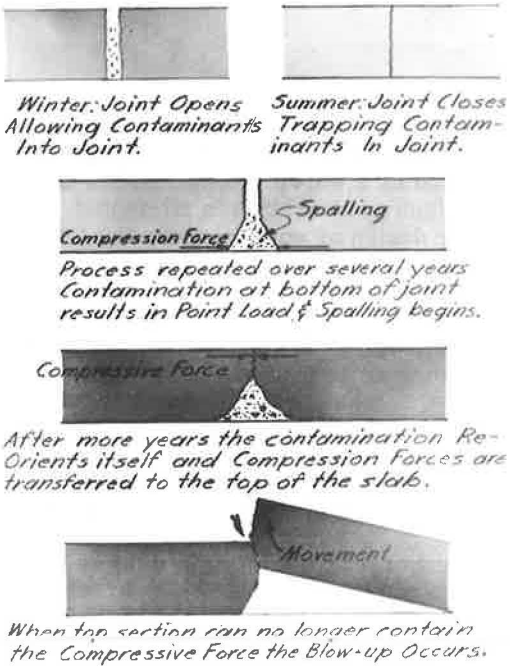


Figure 7. Locations of test sections.

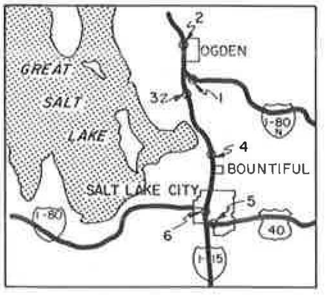


Table 1. Test sections.

Section	Location	Age	Joint Formed by	Comment
1	I-80N, Uinta Jct. and Riverdale	6 months	Sawed	
2	I-15, Sladerville Exit	5 years	Plastic strip	
3	I-15, Roy Exit	9 years	Sawed	Blowups in area
4	I-15, near Centerville Exit	1½ years	Sawed	
5	I-80, 20th East St., Salt Lake City	8 years	Sawed	
6	I-15, 2nd South St., Salt Lake City	10 years	Sawed	

Figure 8. Core from section 1.

mer and therefore local concentrations of compression arise which spall the joints. The spalled material is added to that already at the bottom of the joint and the process is repeated over several years with progressive spalling. After some years, the situation changes and the compression is transmitted through the relatively sound tops of slabs. This may happen because the infiltrated material reorients itself in the joints so that it will no longer transmit compression between the bottoms of slabs. The relatively sound tops of the slabs present a reduced area to the compression force and an upwards eccentricity so there is a much greater liability to blow-up than in the original sound slab." This phenomenon is shown in Figure 6. The deterioration of the pavement at the bottom of the joint is also a contributing factor in the faulting, cracking, and pumping of pavement slabs (4).

One major problem with deterioration on the underside of the pavement is that it is very difficult to detect before the pavement failure occurs. In many cases a visual inspection of the top of a joint does not disclose potential problems at the bottom. To evaluate deterioration at the bottom of the joints, cores were taken. Six areas (Figure 7) were chosen for sampling that would represent pavements of different ages and conditions, as listed in Table 1.

A total of 45 cores were taken in these test sections. Before coring, an epoxy was poured into the joint to set the contaminants so they would not be removed while coring.

The pavement in section 1 was 6 months old at the time of coring. No pavement deterioration was evident from the top of the slab. During the coring process the water used in coring ran along the top of the joint, indicating that it was watertight. Of the 3 joints sampled, only 1 had cracked. The core from the joint that had cracked is shown in Figure 8. The seal in this core was in excellent condition as evidenced by the amount of cohesion. There was no deterioration of the joint walls or bottom. Only a slight discoloration was apparent on the joint walls.

The joints in section 2 were formed by use of a plastic strip and were 5 years old. A visual inspection of the joint surface indicated that the joint was open and being infiltrated with contaminants. The longitudinal joints, which were also formed with plastic strips, were displaced at each transverse joint intersection (Figure 9). This displacement has been noted in another report (3) and is a construction problem with plastic strips. The cores revealed that a dense layer of granular material 2 in. (5 cm) deep lay at the bottom (Figure 10). All of the joints sampled had cracked, affirming that the plastic strips do indeed produce an effective weakened plane in the pavement, giving a controlled crack location.

Section 3 had the most deteriorated joints of any test section. This section was also the section in which the pavement blowups had occurred. In the wider joint openings large aggregate was noted at the surface. Some grain dropped by passing trucks was also observed in the joints (Figure 11).

A temporary bituminous filler had been used to repair the blown-up section and a small amount of slab migration was evident in the joint widths approaching the patched sections. The bituminous mix filler acted as a pressure relief joint, permitting the slabs to migrate. To maintain the load transfer between slabs the aggregate interlock must be maintained. Therefore, because of slab migration a bituminous slab filler must only be used as a temporary repair measure.

The cores taken in this area (Figure 12) revealed 3 layers of contamination. These layers indicated that a crushing action was taking place. The layer at the top of the joint was mainly large aggregate whose size depended solely on the opening of the joint. The middle layer contained a fine-grained material, and coarser material was at the bottom. In several cores the coarse-grained material was $\frac{1}{4}$ in. (0.63 cm) thick and 5 in. (12.7 cm) deep. One of the cores had a sprouted seed midway down the core (Figure 12). The joints close to the blowup showed little evidence of contamination layering. While the cores were being taken it was noticed that little water was coming to the surface. It was felt that there was a void under the joint. If this was true, then the coarse aggregate would have dropped into the void, forming no aggregate layer in the joint.

Section 4 was $1\frac{1}{2}$ years old and the surface of the joint showed some deterioration (Figure 13). Adhesion between the seal and the top of the joint walls had started to

Figure 9. Longitudinal joint displacement in section 2.

Figure 10. Core from section 2.

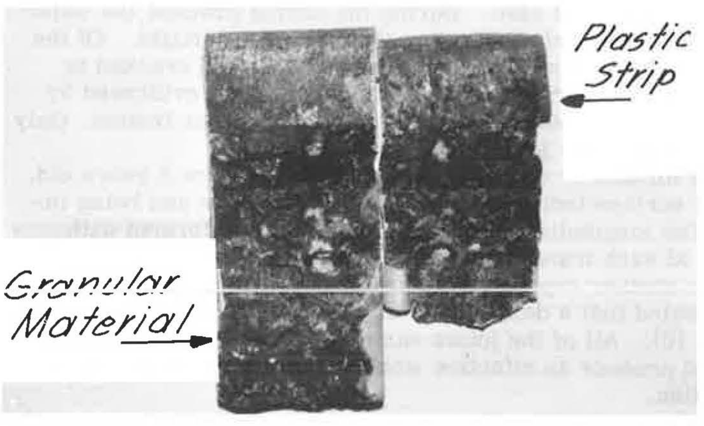


Figure 11. Joint containing grain seeds.

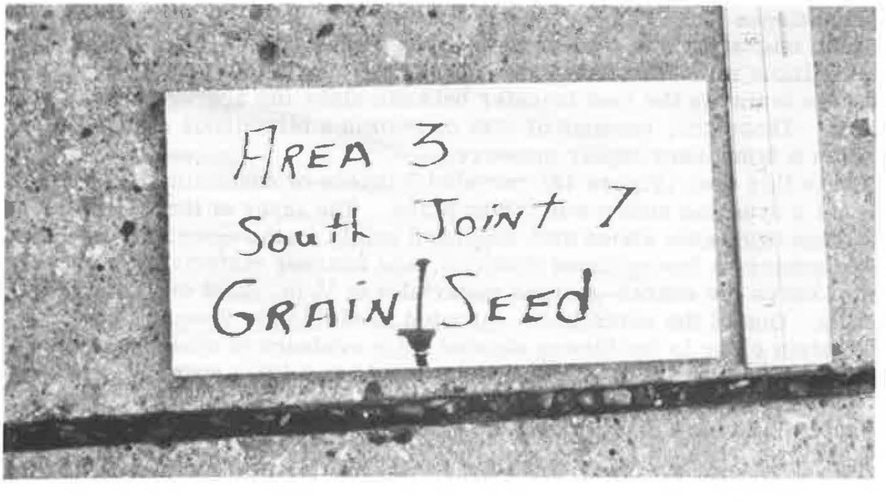


Figure 12. Cores from section 3.

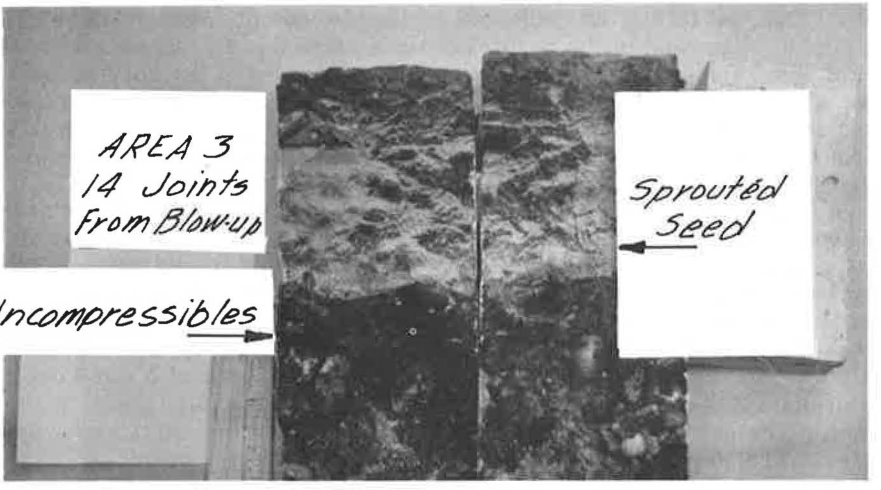
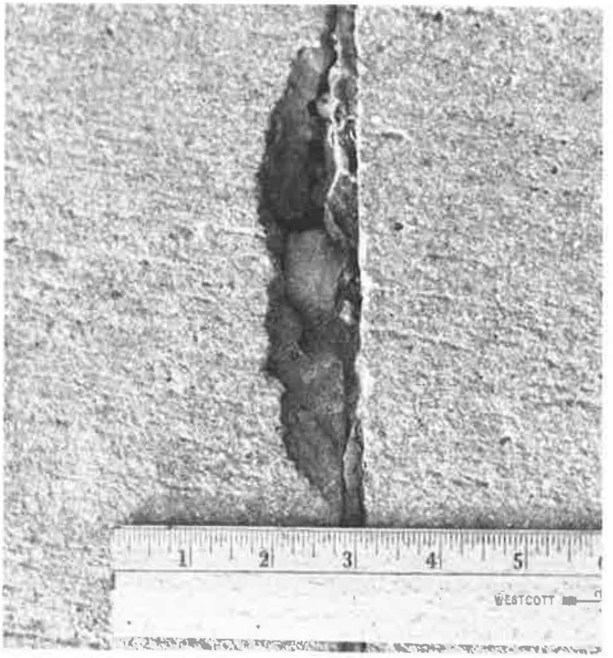


Figure 13. Spall at joint in section 4.



fail (Figure 14). The cores revealed that some seals had failed where others had not. The 3 cores shown in Figure 15 are at different stages of failure. The core at the left contains a seal that has not failed, as evidenced by the absence of epoxy or other contaminants in the joint. The seal of the center core has started to fail as shown by the limited amount of epoxy that was able to pass the seal. The seal of the core at the right has failed, as shown by the epoxy and incompressibles found at the base of the core. In Figure 16 the problem of adhesion is apparent. The seal was adhering to the aggregate but not to the cement mortar. Cleaner joint walls may solve this type of adhesion problem.

Sections 5 and 6 are respectively 8 and 10 years old. The surface conditions of the joints in these areas were about the same, with no apparent excessive deterioration. The lack of epoxy on the walls of the cores indicates that the joints were closed when the epoxy was poured (Figure 17). In the laboratory all of the cores separated easily, indicating there was no adhesion of the seal. The contamination in the joint was different from those joints sampled in sections 2 and 3 (Figure 18). In sections 2 and 3 the contamination formed layers, whereas the contaminants in sections 5 and 6 displayed a fairly even coating of very fine, clay-like material on the joint walls, with no apparent layering. The presence of contamination indicates that the joints are subject to infiltration, even though at the time of coring no infiltration was evident.

One possible reason for the difference in the type of incompressible in sections 2 and 3 and 5 and 6 is the winter maintenance procedures in these areas. In the Ogden area, where sections 2 and 3 are located, sand and salt are used. In the Salt Lake City area, where sections 5 and 6 are located, only salt is used.

The possibility that the contamination in the joint is material from the base was considered, but because of the treated base it was felt that any migration of this material would be minimal.

Deterioration at the joint slab bottom was detected in all but the newest pavement. The bottoms of the joints in section 1 were square, with no spalling (Figure 19). Cores from the oldest sections (3 and 6) had spalling that was around 2 in. (5 cm) in height (Figure 20). The rate at which this spalling occurred was not investigated.

During inspection of the transverse joints it was observed that the longitudinal joint between the pavement and asphalt shoulder was in a bad condition. In many cases a ½-in. (1.3-cm) horizontal gap existed between the 2 surfaces. Also, there was a depression forming a trough at the concrete pavement edge (Figure 21). Any water falling on the roadway would run off the pavement and into the longitudinal edge joint (Figure 22) instead of running off the pavement and over the shoulder. With the longitudinal edge joint badly in need of repair, the quantity of water entering the base material via the transverse joints is insignificant compared to the amount entering via the pavement edge joint. If watertightness was a serious problem, the longitudinal edge joint should be repaired before attending to the water-tightness of the transverse joint. To alleviate the pavement edge joint problem in the future, the concrete pavement will be widened so that the shoulders will be of concrete instead of asphalt.

A limited investigation was conducted to determine if any bridge pushing from pavement growth had occurred. The only evidence of possible pushing was found on the Bluffdale overpass on I-15. The abutment joint on the bridge had been closed and spalling on the abutment had occurred (Figure 23).

The longitudinal joints in the pavement were found to be in good condition in all test sections. Movement experienced by the longitudinal joint is restricted due to the tie bars.

ANALYSIS OF FINDINGS

After finding that all but the most recently sealed joints had failed, a further literature review was conducted to determine if there were problems with the type of seal used or possibly the joint design.

The expected movement for an 18-ft (5.5-m) slab over a 130 F (54.4 C) temperature range is 0.112 in. (0.28 cm):

$\Delta L = \Delta T \alpha L = 0.112$ in. (0.28 cm)

where

$\Delta T = 130$ F (54.4 C),
$\alpha = 4 \times 10^{-6}$ in./in./deg F (7.2×10^{-6} cm/cm/deg C), and
L = 18 ft (5.5 m).

The coefficient of thermal expansion for ordinary concrete has commonly been assumed to be about 5 to 6×10^{-6} in./in./deg F. Because of subgrade restraint a practical calculation for anticipated joint movement for either plain or reinforced slabs can be made by modifying the expansion coefficient to 4×10^{-6} in./in./deg F (7.2 cm/cm/deg C) to compensate for restraint (1). A slab length of 18 ft was used to determine the expected movement instead of the average joint spacing of 15 ft so that the proposed seal design would perform in the worst condition. Thus, for a seal to be effective it must be able to extend a minimum of 0.112 in. (0.28 cm) or W + Δ L (Figure 24).

Tons (5) investigated the effect of the width-to-depth ratio on the sealant. If a straight-line extrapolation of Tons' Figure 14 is used, Utah's present joint design would result in strains along the parabolic curve of 1,780 percent when extended to the calculated maximum width. Strains of this magnitude far exceed the strain limits of any seal in use, and this gives reason for the universal seal failure that was found. Therefore, a new joint design will be needed to ensure that the joints will be properly sealed.

NCHRP Synthesis 19 (1), in discussing durability and working range of joint seals, states that hot-poured seals have had a service life of about 2 years and preformed materials between 5 and 10 years. Part of this limited life is due to faulty installation, improper design, or excessive spacing.

When these hot-pour materials are used in transverse contraction joints, the reservoir must be wide enough to keep extension of the sealant within its capabilities (usually less than 20 percent).

The recommended working range of the preformed seal was suggested to be 30 percent of the seal width (7, 8).

A polyvinyl chloride (PVC) hot-poured elastomeric sealant is now on the market with a 10-year service life warranty (6). A minimum joint size of ⅜ in. (0.95 cm) by 1¼ in. (3.7 cm) is recommended for an average joint spacing of 25 lineal feet.

If the 20 percent capability limit for a hot-pour seal is used, the required reservoir width (R.W.) would be 0.56 in. (1.4 cm):

$$\text{R.W.} = \frac{\Delta L}{C_H} = \frac{0.112 \text{ in. } (0.28 \text{ cm})}{0.2} = 0.56 \text{ in. } (1.4 \text{ cm})$$

where C_H = 20 percent extension limit.

If the 30 percent capability limit for a preformed seal is used, the required working range (W.R.) would be

$$\text{W.R.} = \frac{\Delta L}{C_p} = \frac{0.112 \text{ in. } (0.28 \text{ cm})}{0.3} = 0.37 \text{ in. } (0.94 \text{ cm})$$

where C_p = 30 percent extension limit.

The size of preformed seal that would be compatible with the expected movement would be one $\frac{7}{16}$ in. (1.1 cm) wide. The seal would be installed in a joint ¼ in. (0.64 cm) wide. The working range of the preformed seal is shown in Figure 25, a force-deflection curve for a typical $\frac{7}{16}$-in. (1.1-cm) seal. For a preformed seal the working

Figure 14. Sealing material in section 4.

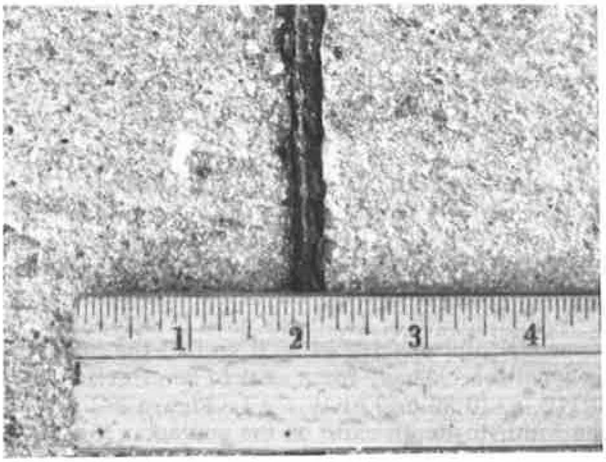


Figure 15. Cores from section 4.

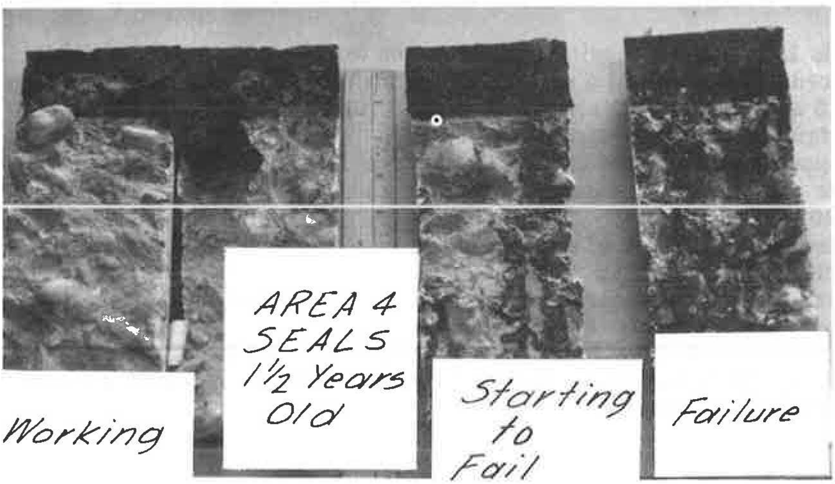


Figure 16. Seal failing in adhesion in section 4.

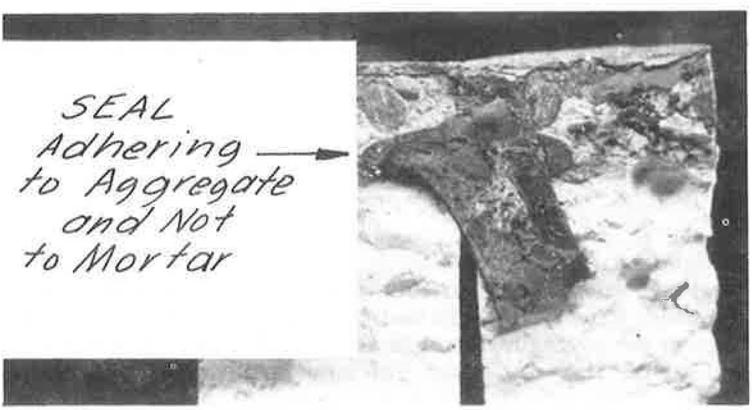


Figure 17. Core from section 6.

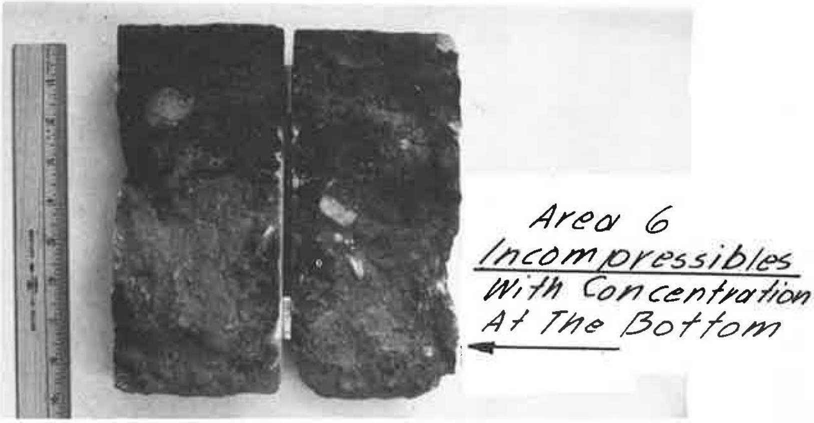


Figure 18. Cores from sections 3 and 6 showing different types of contamination.

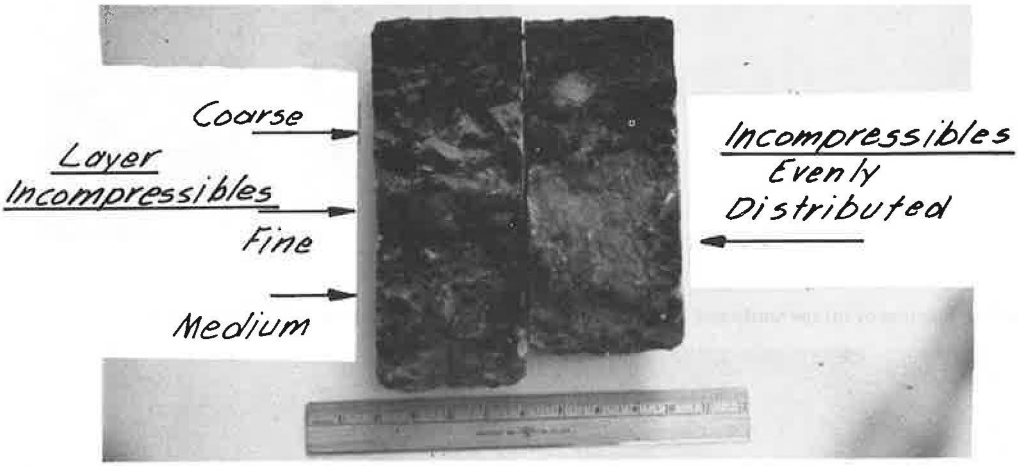


Figure 19. New pavement core with no spalling at the bottom.

Figure 20. Cores from 9- and 10-year-old pavements with spalling at bottom of joint.

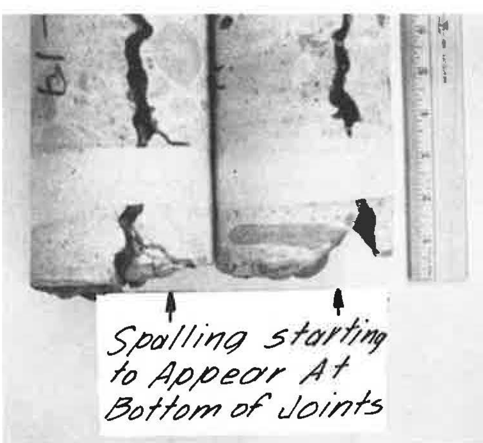


Figure 21. Trough formed between concrete pavement and asphalt shoulder.

Figure 22. Water in depression between pavement and shoulder.

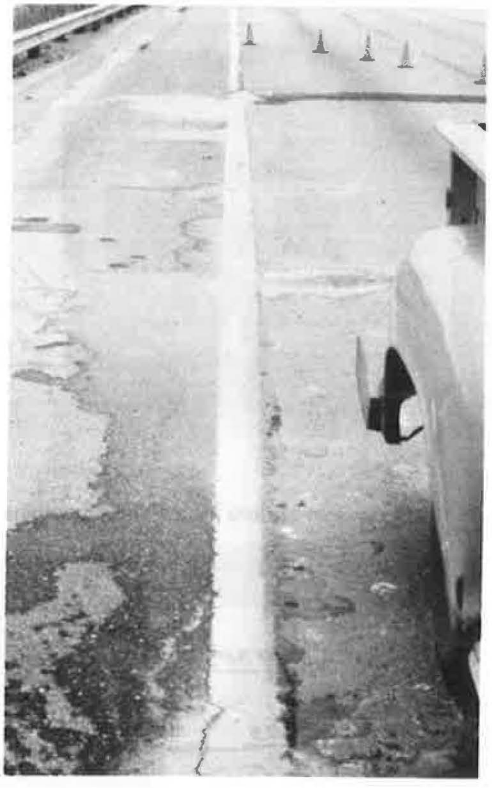


Figure 23. Spalling of bridge abutment joint.

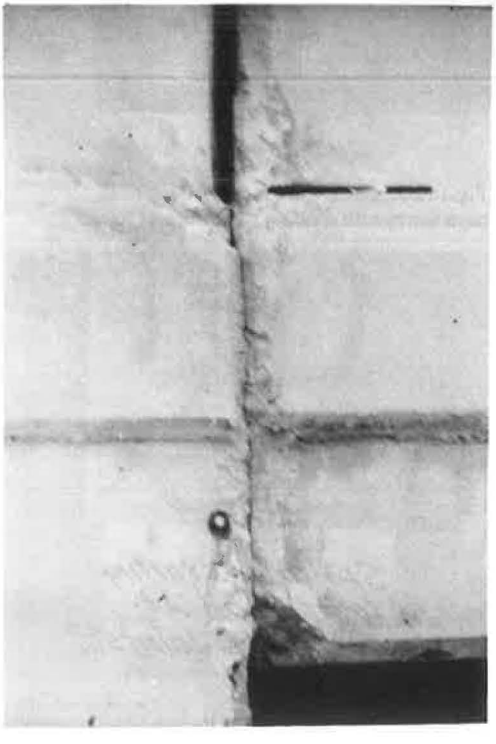


Figure 24. Estimated joint movement.

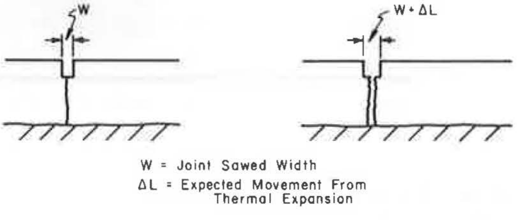
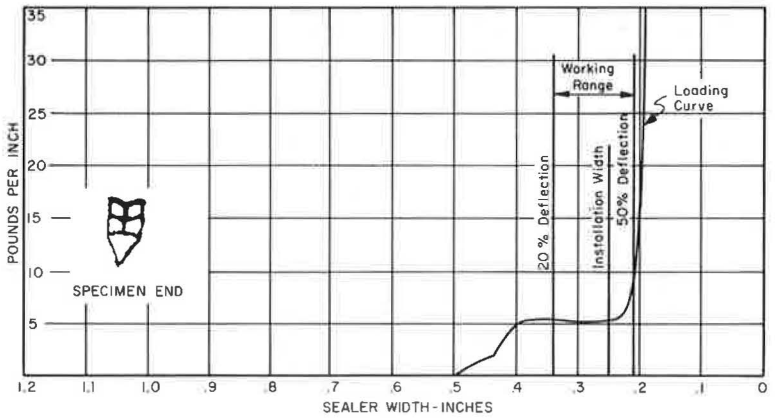


Figure 25. Force-deflection curve for typical 7/16-in. seal.



range is considered to be between 20 percent and 50 percent of the deflection of the seal; 20 percent deflection is used so that enough sidewall pressure is exerted by the seal to keep it in the joint if failure of the lubricant adhesive occurs. At 50 percent deflection the seal has not reached a rubber-on-rubber condition, which would eventually lead to web adhesion and failure of the seal. A sawed reservoir of $\frac{1}{4} \pm \frac{1}{32}$ in. (0.63 ± 0.07 cm) was chosen because the seal is to be installed during the summer months when in the future the joint would be closed, thus ensuring that the seal would not be overly compressed. Also, the ¼-in. (0.63-cm) saw cut allows the contractor to use two ⅛-in. (0.31-cm) blades, which would save on sawing costs.

It was determined from the study results that during the colder months of the year the pavement joints were open and being infiltrated with incompressibles. However, the overall past performance of the pavements has been excellent. At the present time, joint infiltration has not adversely affected the overall pavement performance. This may not be the case in the future, because there was spalling at the bottom of the joints. Spalling in the joints is one of the steps in the mechanism for blowups. To correct the problem, the presently sealed joints should be cleaned and resealed and the present sealing practice should be changed to ensure a more durable sealing system.

Regarding the possible cause for the blowups in Utah, Patrick R. Nolan of Portland Cement Association stated the following concerning the importance of the narrow joint width used in Utah: "If joint seals are effective so that sand and other incompressible materials do not infiltrate the joints, the contraction joints will easily provide for temperature expansions. Blow-up problems have been nearly nonexistent in pavements with short joint spacing (20 ft or less). First, joint sealants perform better at joints with less total movement. Secondly, infiltration of unwanted material is minimized with smaller openings. Blowups are much more common in pavements utilizing mesh dowel design with joint spacing of 40 ft or more. In older pavements, blowups were common at expansion joints where sealants failed, allowing large rocks to enter the joints." To maintain the present pavement design, the smallest possible joint opening that is compatible with the expected joint movement and seal capabilities is desirable. The hot-pour seal, with 20 percent extension capabilities, would be undesirable because of the large joint opening required.

The estimated additional costs for installing the 7/16-in. (1.1-cm) preformed seal and PVC sealant would be as follows:

Preformed seal

Second saw cut, $\frac{1}{4} \times 1\frac{1}{8}$ in. (0.63×3.2 cm)	14 cents
Materials (28 cents) and installation (15 cents)	43 cents
	57 cents
Less cost of present sealing	07 cents
Additional total cost per foot	50 cents

PVC hot-pour

Second saw cut, $\frac{3}{8} \times 1\frac{1}{4}$ in. (0.95×3.7 cm)	16 cents
Material (7.6 cents) and installation (7 cents)	14.6 cents
	30.6 cents
Less cost of present sealing	07 cents
Additional total cost per foot	23.6 cents

The additional cost based on square yards of surface concrete (using an average slab length of 15 feet) would be 30 cents for preformed seals and 14 cents for PVC hot-pour per square yard.

If a figure of \$7.00 per square yard of concrete is used, the percentage increase in construction cost would be 4.2 percent for preformed seals and 2.0 percent for PVC hot-pour.

The cost per service year over the entire life of the pavement (20 years) would include removal of the old seal and resealing at the end of 10 years for each type of seal. Resealing with a preformed seal would require the seal to be pulled out, the joint cleaned by a wire brush or sandblasting, spalled areas repaired, and seal replaced, at a cost of 48 cents per foot. Replacement of PVC hot-pour sealing requires that the old sealant be plowed out, the joint widened to $\frac{1}{2} \times 1$ in. (1.27×2.54 cm), joints cleaned by sandblasting, and seal replaced, at a cost of 28 cents per foot.

Over the 20-year design life of the pavement the cost per year would be 2.9 cents per square yard per service year for preformed seal and 1.5 cents per square yard per service year for PVC hot-pour seal.

CONCLUSIONS

Prior to the pavement blowups that occurred during the summer of 1972, little thought was given to the concrete pavement because the overall performance had been excellent. Following the blowups an investigation of the potential for future blowups in the existing pavements was conducted. The study evaluated the existing pavement joint sealing systems, determined if there was any pavement growth resulting in bridge pushing, and formulated recommendations for actions.

In the evaluation of the existing pavement joint sealing system, 6 pavement sections were inspected and cored. The ages of the pavements ranged from 6 months to 10 years. Even though the general performance of the pavement has been excellent, the joint cores revealed that the seals in all but the most recently sealed contraction joints had failed. Seal failure was noted in joints that were only $1\frac{1}{2}$ years old. In the longitudinal pavement lane joints that were sealed at the same time as the contraction joints, no failures were found. After measuring the widths of the joints and evaluating the extension limits of the seals, it was found that the seals were being overextended. To correct the seal failure problem, wider joints would be required so as to keep the movements within the capability of the sealing material.

The narrowest possible joint width that is compatible with the expected movement and seal capability is desirable when using Utah's pavement design system. Hot-pour materials presently being used would require a joint width increase to 0.56 in. (1.4 cm). This width of joint is undesirable. A $\frac{7}{16}$-in. (1.1-cm) preformed seal installed in a joint $\frac{1}{4}$ in. (0.63 cm) wide or a PVC hot-pour seal in a $\frac{3}{8} \times 1\frac{1}{4}$-in. ($0.95 \times 3.17$-cm) joint was recommended.

The cost per service year over the life of the pavement to change to either a preformed seal or a PVC hot-pour would be comparable.

During the coring of the joints it was noted that the pavement edge joint between the concrete and the asphalt shoulder was in very poor condition. Any runoff water would be funneled into the longitudinal edge joint instead of over the shoulder. This water entering the base and subbase could cause damage to the roadway foundation, eventually resulting in pavement distress problems. Resealing of the joint is important to retaining the integrity of the pavement.

The visual inspection for pavement growth and bridge pushing found that no serious problem of this type existed. Only one possible example was found but this case was not serious.

In conclusion, this study found that even though pavement distress problems have been minor, the existing sealing system has failed. With joints being freely infiltrated, there is good reason to believe the pavement distress problems will become common if action to correct the problem is not taken.

REFERENCES

1. Design, Construction, and Maintenance of PCC Pavement Joints. NCHRP Synthesis of Highway Practice 19, 1973.
2. J. P. Stott and K. M. Brook. Report on a Visit to U.S.A. to Study Blow-Ups in Concrete Roads. Road Research Laboratory, Crowthorne, Berkshire, England, 1968.
3. A Comparison of Transverse Weakened Plane Joints Formed by Sawing and by Plastic Insert. California Dept. of Transportation, Research Report No. CA-DOT-TL-5254-I-73-26.
4. E. J. Barenberg, C. L. Bartholomew, and M. Herrin. Pavement Distress Identification and Repair. Construction Engineering Research Laboratory, U.S. Army, Technical Report P-6, March 1973.
5. Egons Tons. A Theoretical Approach to Design of a Road Joint Seal. HRB Bull. 229, 1959, pp. 20-53.
6. Concrete Pavement Joints Resealed for Long Life. Roads and Streets, Aug. 1974.
7. Robert D. Carlson. Transverse Joint Construction and Sealing Practices—1968-72. Research Report 20, Engineering Research and Development Bureau, New York State Dept. of Transportation, 1974.
8. Evaluation of Preformed Elastomeric Pavement Joint Sealing Systems and Practices. NCHRP Research Results Digest 35, 1972.

DISCUSSION

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I agree with the authors that proper and positive sealing of pavement joints is necessary to reduce pavement distress. They have shown that premium sealant materials are available to perform that function.

The paper compares costs per square yard per service year. I would like to suggest that the final figures may be further refined. Consider the following:

1. A longer service life than 10 years is being reported for the PVC hot-poured

sealant. Annual field surveys confirm continued excellent performance in sizable installations since 1963, which totals 12 years without failures. No loss of their rubber-like properties is apparent after 12 years in the field subject to exposure from -40 F to 125 F ambient temperatures. It seems certain they will go 15 years, possibly 20 years. In reference to neoprene compression seals, there are published highway department data to substantiate that compressive force decreases up to 70 percent of the initial force after 2 years of service. It is now established that neoprene compression seals after a relatively short time in service (several years) take a "set" and do not maintain their original compression recovery force and, more important, do not maintain initial dimension widths.

2. The resealing of joints with preformed neoprene compression seals may be more costly than estimated because the seals will have to be individually sized for each joint and generally larger than the original width seal used in the joint.

I am suggesting these items be considered for revision because PVC hot-poured sealant is warranted for a 10-year period as a minimum life rather than a maximum. A 15-year life would reduce the cost per year for the PVC sealant. In reseal work, nonuniform slab movements, infiltration of fines, and loss of concrete on joint faces from cleaning, shrinkage, and other factors make the joint widths vary. It is also established that preformed seal extrusions vary considerably in width, as there is a variability of $\pm^1/_{16}$ in. in the extrusion process that must be considered. These factors require the individual sizing of preformed seals for each joint when being considered for resealing.

We have recently surveyed a number of state and provincial highway departments by written questionnaire on this subject. Several have done some resealing of joints originally sealed with preformed seal and have found that individual sizing of preformed seals for each joint is required. This would add to the cost per year for preformed seals.

Incorporating these items in the original paper would show an even greater cost advantage accruing from the use of the PVC hot-pour than that shown originally.

AUTHORS' CLOSURE

The purpose of the study was to evaluate the performance of the existing sealing design used in Utah. After finding that the present design was inadequate, a change to a preformed neoprene compression seal or PVC hot-pour sealant was recommended. The recommendation was based on the expected joint movement and the reported service life of each material. Since completion of the report a longer service life of the PVC material is reported by Knoblock. This longer life would indeed reduce the cost of the material over the life of the pavement. How this cost reduction would affect the cost comparison between PVC hot-pour material and neoprene compression seals is unclear because during the same time since completion of the report the service life of the neoprene seal has also been extended.

It is well established that neoprene compression seals do take a permanent set with age. During early design work with compression seals this set was not taken into account and the seal was underdesigned. As stated in the report, a 30 percent working range is now recommended by several states. This working range allows for permanent set, thus guarding against early seal failure through permanent set.

In reference to resealing with preformed neoprene seals, results reported in "Thermal Expansion and Contraction of Concrete Pavements in Utah" indicate that there is not enough variation in joint widths to require individual sizing of each joint resealed or to require a larger seal.

SOME REFINEMENTS IN EXPANSION JOINT SYSTEMS

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Testing and performance inadequacies of the widely used compression seals can be eliminated to a large degree by the use of mechanically locked seal elements. Rehabilitation techniques for expansion joints on older structures utilizing strip seals, low-stress sealing glands, and special low-profile elements are discussed. Major improvements in the rubber cushion concept are advanced, such as armoring for protection against snowplows and attrition, new curb and gutter techniques utilizing weldments for multidirectional changes, and lowering of detrimental force transmission. Improved modular systems for very large movements, methods of raising existing systems to meet asphalt overlay requirements, field splicing techniques utilized in lane-at-a-time reconstruction, and important noise reduction developments for urban environments are evidenced in case history evaluation.

WIDE experience with preformed neoprene compression seals over the past decade on many thousands of bridges throughout the world has left no doubts regarding the advisability of mechanically locking the seal element between armored interfaces and that this refinement is strongly indicated as the way for bridge design engineers to go in the direction of seal design.

For an extensive period of time the writer has served as chairman of a special task group in ASTM that has had the mission of producing an acceptable national specification for the bridge compression seal industry. This task group includes the major producers, users, installers, and state highway department testing authorities, all knowledgeable in the testing of these compression seals. After years of work and a great many meetings, drafts, and redrafts, the committee is still not in agreement because of the vagaries of the recovery tests and the complete lack of a sound method of calculating the true limit of safe compressibility of a seal configuration.

What this means is that we apparently have been seeking something very hard to find: We expect a bridge compression seal, extruded from an organic material and subjected to long-term compression, to continue to recover and retain a functional level of residual compression when we already know there will be a time-dependent, gradually increasing loss of pressure. The complexities of different seal configurations, unavoidable changes in web thickness, and the need for a sufficient amount of internal web structure to resist traffic-activated vertical forces all complicate the problem for testing engineers.

By adding the fail-safe ingredient of mechanically locking the seal element to the joint interface, engineers can now specify with confidence the seal configuration shown in Figure 1 because most problems currently symptomatic of neoprene compression bridge seals would be eliminated. Specifically, mechanical locking

1. Eliminates concern for the problem of long-term pressure decay of the seal element;
2. Eliminates the argument over a seal's limit of safe compressibility;
3. Handles movements in excess of its design stroke, free of overstress;
4. Continues to perform successfully regardless of time-dependent, unanticipated joint width changes;

Publication of this paper sponsored by Committee on Sealants and Fillers for Joints and Cracks.

5. Ends forever any possibility of intrusion of foreign material by establishing and maintaining a permanent, impervious locking contact with both joint interfaces;
6. Ensures performance against unusual dynamic vertical forces due to live loading, hydrostatic pressures, etc.;
7. Offers performance in near-cryogenic environments, since the practical limit of performance of neoprene stops at -20 F when used as a compression seal;
8. Facilitates much narrower joint widths than any other sealing concept, including basic compression sealing, which requires a 20 percent safety factor in width not required in the corner locking systems; and
9. Provides much lower overall cost.

REHABILITATION OF BRIDGE DECK FINGER JOINTS WITH STRIP SEALS

Finger or comb joints can be restored in a leakproof manner using strip seals. The old fingers are sawed out by means of a moving guided torch, after which the steel extrusions are welded at the desired elevation where the protruding fingers have been cut off. Figure 2 shows one lane with the steel extrusions welded in place. The adjacent extrusions will then be butt-welded in an impervious manner, end to end. The strip seal element then is placed a lane at a time in a continuous strip across the joint and up through the curb in a leakproof installation (Figure 3).

Bolting the Strip Seal Extrusions

A method of bolting the steel extrusions to the deck is shown in Figure 4. Bolt-hole tabs are welded to the steel extrusions, extending back into the deck as far as is necessary for good structural practice. The seal element is installed after both steel interfaces are securely fastened down, after which the asphalt overlay is placed using the steel extrusions as an end dam.

Rehabilitation of Bridge Deck Slider Plate Joints With Strip Seals

Strip sealing systems are now being widely used to repair sliding-plate joints where a new asphalt overlay is being installed over the old decks.

The slider plate is cut off by means of a moving torch and then rewelded in place to re-form the desired joint opening. The strip seal extrusions can then be welded or bolted to the reconstructed steel joint opening a lane at a time, after which the neoprene strip seal element can be installed in one piece across the deck, making it a leakproof installation.

Field Installation of Strip Seal Element

Figure 5 shows field installation of the strip seal element after all construction work is complete. This is accomplished by using a tool shaped like a tire iron, which rotates the locking lugs of the gland into their mating receptacles. This is an important feature of a sealing system because replacement of damaged glands could become necessary in case of fire or vandalism.

Skewed Joint Movements With Strip Sealing Systems

Strip sealing systems offer good performance for even the most extreme angles of skewed joints. Many hundreds of installations of strip sealing systems are now operating

Figure 1. Typical mechanical locking of the seal element.

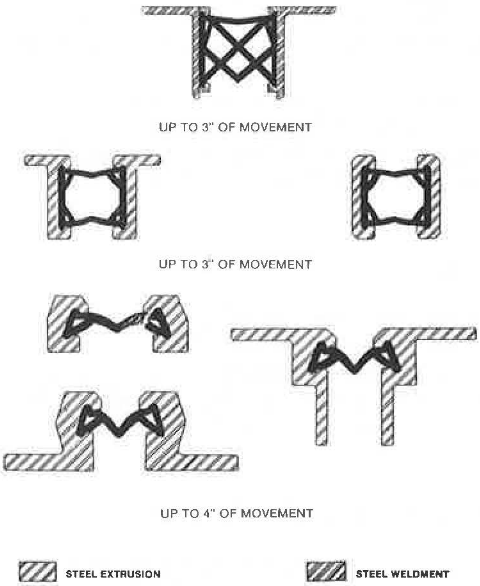


Figure 2. Seam-welded steel extrusion replacing old finger joint.

Figure 3. Installing strip seal element in one piece, a lane at a time, for 100 percent leakproofing.

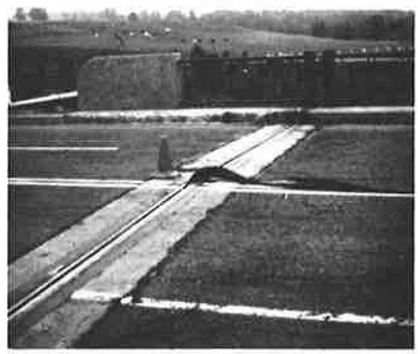


Figure 4. Bolting strip seal armor to bridge deck.

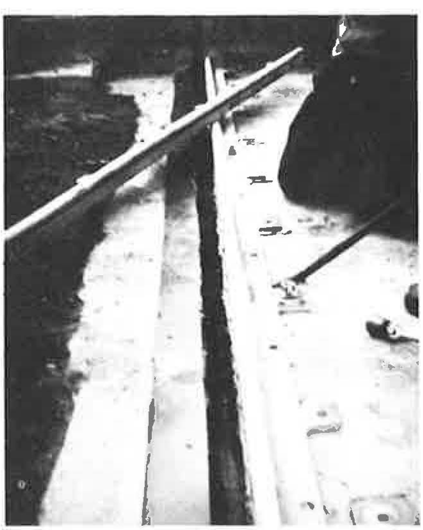


Figure 5. Field installation of strip seal gland.

successfully in service on new and rehabilitated bridge decks around the world—Germany, France, Holland, Belgium, England, Austria, Africa, portions of the new Bosporous Bridge in Turkey, as well as a number of installations in Canada and, more recently, in the United States.

As an example of the low forces attainable in a skew, at an angle of 45 deg, tests show the ability to absorb ±1 to $1\frac{1}{2}$ in. (25 to 38 mm) vertically to its axis and ±1 to $1\frac{1}{2}$ in. in the direction of its axis. The shear force occurring in the longitudinal direction of the seal at a deformation of $1\frac{3}{16}$ in. (30 mm) will be 235 lb/ft (3.33 kN/m).

Differential Height Problems at Opposing Interfaces

There are occasions where opposing interfaces are constructed at different times and under differing conditions of vertical load, creep, time-dependent variations in crown, etc., that could result in an unpredictable difference in height (Figure 6). The strip seal system, with the seal element capable of being installed at any time, coupled with very low working stress, is an ideal solution for sealing problems of this type.

Interchangeability of Strip Seal Elements for Unanticipated Movement

Should unanticipated permanent movements occur in a bridge structure far in excess of design calculation, one merely has to change the size of a strip sealing gland. This not-too-infrequent problem, whose principal cause is excessive creep-shrink in post-tensioned structures, has often been solved with strip seals in this manner by using the next-larger gland.

A NEW ARMORED RUBBER CUSHION SEALING SYSTEM

Confirming the necessity for armor-plating of rubber surfaces that will be subject to traffic loadings is the newest design of low-stress gland-type rubber cushion sealing shown in Figure 7. Significant features of this design are

1. A one-piece sealing gland that extends the full width across the joint, compared to conventional 4-ft (1.22-m) lengths;
2. A low-cost, replaceable sealing gland;
3. Movement ranges up to 3 in. (76 mm);
4. Total armoring of the rubber cushion; and
5. A very low cyclic stress transmission to the structure.

New Low-Profile Mechanically Locked System

Figure 8 shows a compartmented, mechanically locked seal element with a low profile specifically designed for low height requirements such as are necessary for overlaying old existing bridge decks.

The system, when subjected to an extreme movement test, does not pull out of engagement at 7 in. (178 mm) of elongation (Figure 9).

Very low forces are transmitted to the structure using low-profile seal elements compared to most heavy-duty bridge compression seals.

PROTECTION AGAINST SNOWPLOWS, STUDDED TIRES, AND OTHER WINTER MAINTENANCE CONDITIONS

Lowering the System Below the Riding Surface

Typical examples of snowplow, studded tire, and tire chain winter damage are shown

Figure 6. Strip seal system has capability of accepting extreme changes in interfacial elevation.

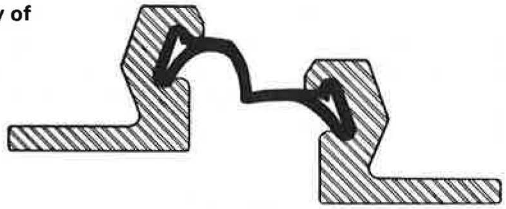


Figure 7. Armored rubber cushion sealing system.

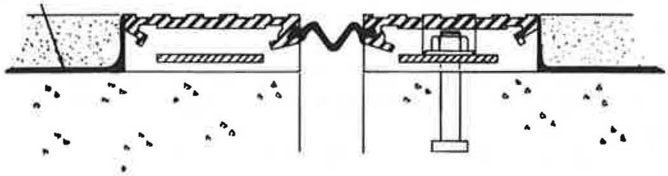


Figure 8. Low-profile mechanically locked sealing element.

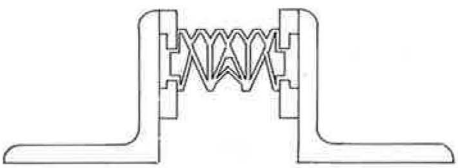


Figure 9. Low-profile system subjected to extreme elongation.

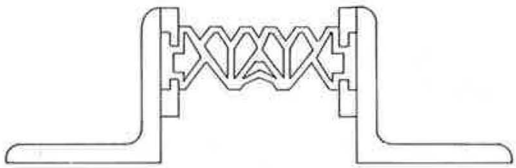


Figure 10. Damage to seals incurred during winter conditions.

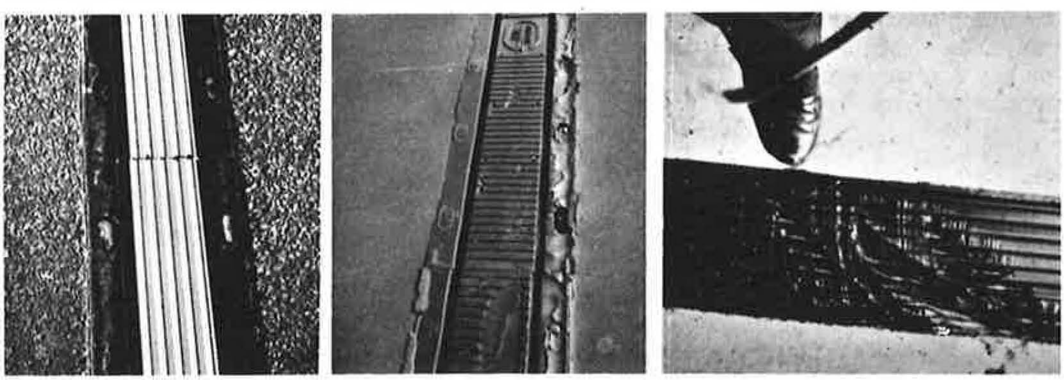
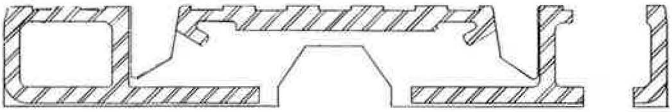


Figure 11. New fully armored rubber cushion system offers 100 percent protection of neoprene against snowplows and studded tires.



in Figure 10. The normal wearing away of the asphalt riding surface adjacent to the expansion joint exposes the edges of the jointing system to the damaging effects of plows, and synthetic rubber at low temperatures is highly susceptible to attrition. Because of this, rubber cushions lacking snowplow protection should be installed below the riding surface to anticipate this time-dependent change in elevation.

Protecting or Armoring of the Rubber Cushion

Most manufacturers of rubber cushion sealing systems are alert to the problems of winter damage, and a number of methods of incorporating snowplow protection are now becoming available. Since there are few environments in North America where snow is not encountered at least occasionally, it appears that it would be good business judgment for all of these systems to encompass at least some snowplow protection in their design.

Figure 11 shows a new design using armor to protect the center portion of the neoprene, which at low temperatures takes on the consistency of brick cheese insofar as its resistance to attrition is concerned. Its entire wearing surface is protected by the addition of a high-strength, corrosion-resistant aluminum extrusion that serves as both a wear plate and a fastening component.

IMPROVEMENTS IN THE RUBBER CUSHION SEALING CONCEPT

The use of molded rubber cushion expansion joints, various types of which have existed for the past 30 years, has been greatly impeded and their general acceptance by bridge engineers around the world discouraged to a large degree by the difficult problem of their high incidence of leaking, particularly at the curb and gutter area.

Most designs of molded sections, by reason of conventional practical rubber production techniques, are restricted to sections 4 to 6 ft (1.2 to 1.8 m) long. Furthermore, there are probably no 2 curb and gutter configurations that incorporate the same design on any 2 adjacent bridges. As a result, there are a great many leaking rubber cushion expansion joint systems.

Prefabricating Curbs by Means of Welding

Advances in design of curb and gutter treatments are shown in Figures 12 and 13, where the units are joined by a series of welded connections.

The appropriate solution is detailed by shop drawings, then factory prefabricated by means of miter cutting and welding them together to fit the exact contours of the curb. Figure 13 shows a factory-prefabricated unit incorporating 3 vertical and horizontal bends.

Greatly Reduced Cyclic Stresses in New Designs of Rubber Cushion Systems

The new rubber cushions shown in Figure 14 have been intentionally redesigned to achieve the lowest possible cyclic working stress attainable in keeping with good structural design. Tests have validated that the forces at full tension-compression have now been lowered to approximately 500 lb per linear foot (7297 N/m), a highly desirable reduction from earlier designs of these rubber cushion systems for this 4-in. (102-mm) range of movement.

Ideally, a sealing system should transmit zero stress to the structure, since the designer of a bridge often has no control over the selection of the device to be used on

Figure 12. Completed installation of factory-prefabricated curb section.

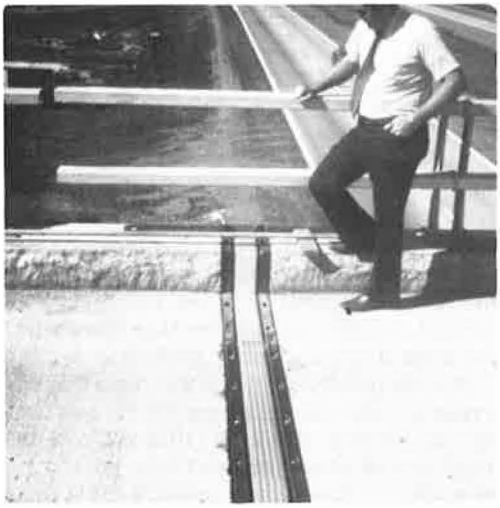


Figure 13. Factory-prefabricated unit incorporating vertical and horizontal bends.

Figure 14. Improved rubber cushion system incorporating low cyclic stress.

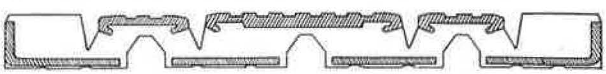


Figure 15. Urato Bay Bridge at Kochi, Japan, incorporates modular system at main-span joint.

Figure 16. Urato Bay Bridge 4-tube system located at midspan for 10.4-in. (264-mm) movements.

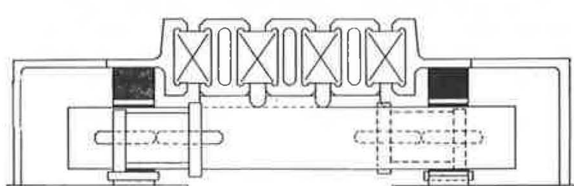
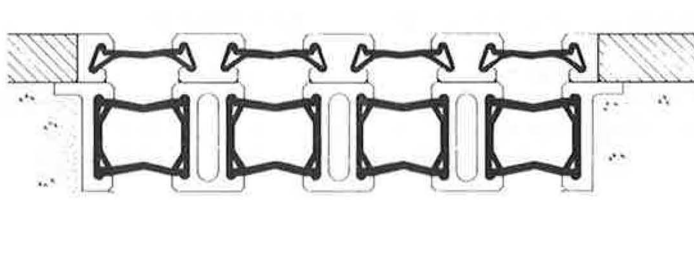


Figure 17. Using strip seal component to elevate existing systems for new overlay.



the bridge. Systems that produce very high cyclic stress can be harmful to the structure, and therefore the designer should be aware of the availability of armored rubber cushion systems in the larger movement category of 4 in. (102 mm) and over that incorporate this most desirable stress reduction.

IMPROVEMENTS IN MODULAR SEALING SYSTEMS FOR LONG-SPAN POST-TENSIONED BRIDGES

The use of improved modular systems as a solution for accommodating large movements and rotation between the span ends on very long, free-standing, post-tensioned concrete box-girder bridges has begun to attract the attention of bridge designers.

The recently completed record-shattering 754.5-ft (229.97-m) box-girder span soaring over Urato Bay in southern Japan (Figure 15) incorporates in the juncture of the 2 cantilevered sections at midspan an expansion jointing system like that shown in Figure 16. This system permits horizontal movement along the axis, providing for the expansion and contraction plus concrete creep. It also positively permits shear movements and rotations and maintains very low friction sliding values since PTFE to stainless steel sliding surfaces under permanent precompression of 4 kips (17.8 kN) are incorporated in the spring-loaded mechanism at each end of every support bar joist.

Some designers have previously used drop-in spans or continuous prestressing in the center. However, an expansion joint system of this type permits the superstructure to accommodate all types of movements, thus eliminating the need for complicated and expensive bearing devices over the main piers. It also results in no moment at midspan, thus making the structure as light as possible.

In view of the success of the design of Urato Bay Bridge, it is the opinion of the writer that span lengths can go much further, possibly 1,000 to 1,500 ft (300 to 450 m), assuming the inevitable development of the durable lightweight concrete that will be necessary to their becoming economically and technically feasible. Urato Bay Bridge utilized conventional concrete, with only 10 percent of the design load over the main piers being live load. Reducing the dead load would substantially cut the amount of material and greatly simplify the design of these attractive and welcome longer spans.

Adjusting the System for New Overlay Requirements

With so many thousands of modular systems being placed in service during the past decade and their obvious ability to outlast not only the deck surfacing but the deck itself, there is often a requirement to raise the system to accommodate the height of a new concrete or asphalt overlay.

Figure 17 shows the method of superimposing small steel extrusions on top of the original separation and edge beams by means of welding and installing strip seal extrusions over the existing tubular seal elements. This can be done very inexpensively and quickly, 1 lane at a time, using standard strip seals. The steel extrusions can be simply and securely fastened a lane at a time under heavy traffic, followed by the installation of the strip seal in the same manner. The writer cannot envision a situation where material and labor could be more effectively used to produce such an ideal solution as is available in the method shown in Figure 17.

Provisions for Future Widening of the Bridge Deck

Increasing traffic often dictates that existing in-service bridges be extended or widened. Provision should be made in the basic design of expansion jointing systems for this future possibility.

The hollow steel separation beams that are components in these improved systems lend themselves nicely to tying together in a lane-at-a-time manner, with the incorporation of steel dowels making a rugged splicing tie between the opposing members

Figure 18. Splice components.

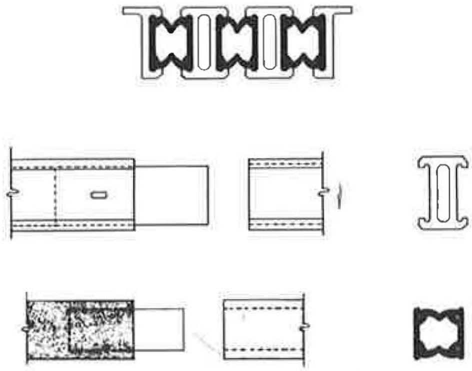


Figure 19. Sliding spring and bearings in normal position under precompression.

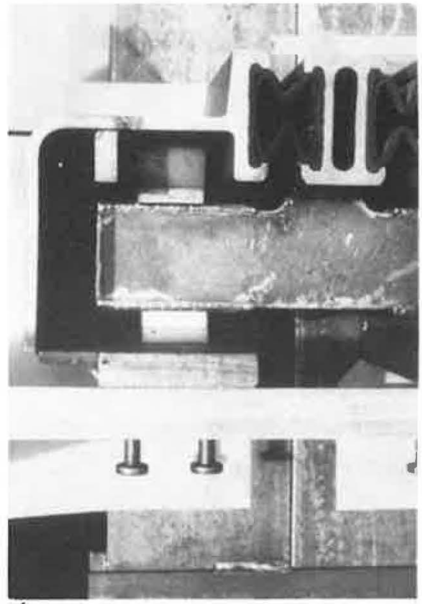


Figure 20. Spring-bearing mechanism performing satisfactorily under rotation.

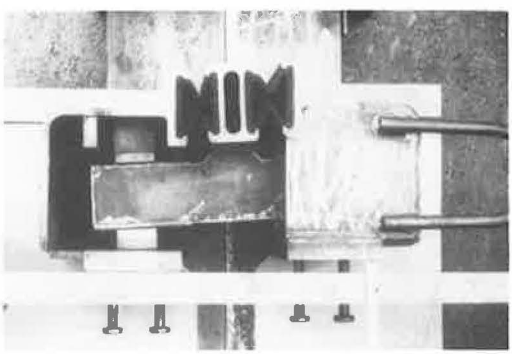


Figure 21. Unusually curved joint in Cologne-Bonn Airport upper departure level structure sealed with single-tube system.

Figure 22. Old seal element.

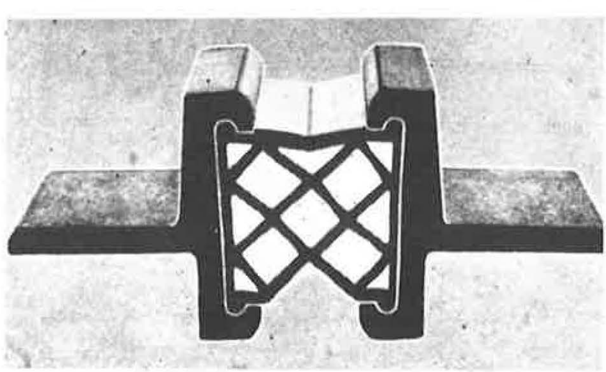


Figure 23. Improved seal element.

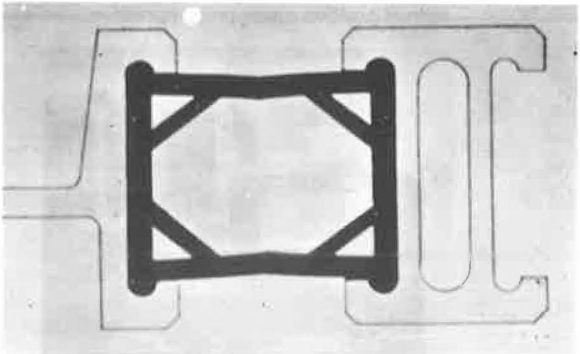
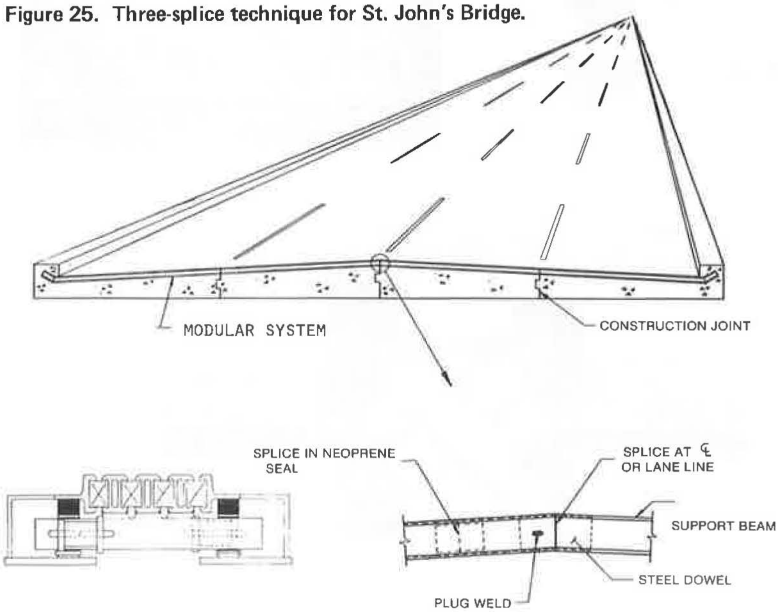


Figure 24. St. John's Bridge in Portland, Oregon, received new expansion joints.

Figure 25. Three-splice technique for St. John's Bridge.



(Figure 18). The seal elements can either be removed and replaced with longer lengths to be continuous across the deck or they can be effectively spliced by vulcanizing.

Rotational Capability

The new longer span, thinner, and more elastically responsive structures coming off the drawing boards of the current generation of bridge designers have posed a challenge to expansion joint designers. Because of this design trend, the ability to withstand extreme rotation of the span ends is a must. This capability is incorporated in a sliding urethane spring-bearing mechanism located at either end of the support bar joists, as shown in a test situation in Figures 19 and 20.

Figure 19 shows the sliding spring in 4 kips (17.8 kN) of compression at the top of the support joist and the sliding bearing in normal support position underneath. It can be seen from the type of test in Figure 20 that the flexibility of the spring-bearing arrangement easily accepts the shifts in forces without losing contact and in a manner that is in no way deterimental to the performance of the system.

Unusually Curved Joints Can Be Effectively Sealed

A typical example of the excellent lateral design flexibility of these systems is a severely curved joint that reverses its direction at the ends of 2 spans of the upper-level structure of the Cologne-Bonn airport (Figure 21). The spans were post-tensioned, and because of structural economics this unusual joint configuration was the best possible solution, all things being considered.

At an early period in its design life, a webbed seal element such as shown in Figure 22 was used. However, the very high working stress in the seal under full compression, high force transmission to the structure, and actual extrusion of the seal in some cases during full joint closure resulted in switching over to a relatively hollow seal (Figure 23). The need for internal webbing is no longer present when a mechanically locked seal is used and is, in fact, a deterrent to the service life of the seal element.

Typical Rehabilitation of Venerable Old Structure

The majestic and colorful St. John's Bridge in Portland, Oregon (Figure 24), designed by the late D. B. Steinman and opened to traffic over 40 years ago, recently had its old finger expansion joints removed and replaced with systems that incorporate a movement capacity of up to 10.4 in. (264 mm). The passage of time and increasing traffic loads had taken their toll to the point that county engineers made the decision to extend its service life by incorporating sound, maintenance-free expansion joint practices.

Because of heavy traffic, a design requirement was that the system be installed in lane-at-a-time widths (Figure 25), which complicated the reconstruction but was an absolute necessity. Connection details have now been engineered that will make the finished system perform similarly to a single-length assemblage.

PRESTRESSED PAVEMENT DEMONSTRATION PROJECT

Raymond J. Brunner, Bureau of Materials, Testing and Research,
Pennsylvania Department of Transportation

This report describes the installation of the first full-scale prestressed concrete pavement in Pennsylvania. A detailed account of the construction and placement methods used and pertinent cost data are included. The purpose of this project was to employ pavement prestressing techniques on a production basis. A variety of construction problems encountered provided valuable experience as to what can be expected on future projects of this nature. Some 23 post-tensioned slabs were placed on the main line, each with a nominal length of 600 ft (183 m). The concrete was placed with a slipform paver that also guided the prestressing strands into the pavement by feeding them through metal tubes on the paving equipment. A unique method was used to construct the joints where the slabs meet. During paving, a 3-ft (0.9-m) jacking space was provided between slabs. This space was later filled with concrete, which was prestressed by transferring the load from the strand anchors through the joint concrete. The slab ends were keyed together with an interlocking beam system to prevent faulting at the joints. Paving work was completed in December 1973. The riding quality over the pavement joints is rated as excellent. The joints are functioning well and are providing for slab length changes caused by temperature variations. The project is being closely observed and its serviceability will be documented in future reports.

•THE use of prestressing techniques for highway pavement has the potential for providing a longer lasting roadway than conventionally reinforced concrete. Prestressing eliminates cracking and transverse contraction joints, both of which are major causes of distress and failure in concrete pavements. Recent experimental installations have shown that prestressed pavement will perform well (1, 2). Excellent riding quality should be obtainable because of the presence of joints only at 600-ft (183-m) intervals. A significant reduction in materials is also achieved because of the substantially reduced pavement slab thickness and reduction in the amount of reinforcing steel.

SCOPE

The main-line pavement consists of 23 post-tensioned slabs with a total length of 13,232 ft (4033 m). The slabs are all 24 ft (7.32 m) wide, 6 in. (152 mm) deep, and nominally 600 ft (183 m) long. Each slab contains 12 prestressing strands, as shown in Figure 1. Maximum grade on the main-line pavement was 1.50 percent, and the maximum curvature was 3.0 deg.

MATERIALS

The pavement is supported by an aggregate bituminous base course 6 in. (152 mm)

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thick and an aggregate subbase course 6 in. (152 mm) thick. Concrete used in the pavement was a 6.25-bag/yd^3 (8.17-bag/m^3) mix conforming to PennDOT Class AA specifications for normal paving concrete. Prestressing strands were 7 wire strands of high-tensile steel in a polypropylene conduit, prepacked with corrosion-inhibiting grease (Table 1).

CONSTRUCTION PROCEDURE

Slipform Paving

Concrete placement was performed by a conventional CMI slipform paving train. Since no transverse reinforcing steel was used, a method was developed for feeding the prestressing strands into the concrete and positioning them at the design depth of 3.5 in. (89 mm). Steel tubes were attached to the spreader machine and the strands passed through these into the concrete. The 12 reels of strand were carried ahead of the paving train on a modified flatbed truck from which they were unwound onto the base, to be picked up by the placement tubes on the concrete spreader (Figures 2 and 3).

The design of prestressed pavement requires that the coefficient of friction between the slab and base be less than 0.6 (3). A friction-reducing membrane was therefore provided by using 2 layers of 4-mil (0.10-mm) polyethylene sheeting between the concrete and base course. To ensure a smooth surface on the base course, sand was swept over it prior to paving. The plastic sheeting was unrolled onto the base ahead of the concrete spreader.

A slab length of 600 ft (183 m) was chosen as the optimum length. Shorter slabs would have increased costs due to placing more joints, whereas longer slabs would have had insufficient resistance to cracking at midpoint due to friction between the slab and base.

The remainder of the slipform paving operation was done by conventional methods. Concrete curing was accomplished with wet burlap, straw, and plastic sheeting for the first 11 slabs. After that, white membrane curing compound was used.

Joint Placement

The continuous placement of prestressed pavement required that a jacking space of at least 3 ft (0.9 m) be left between slabs. A method of placing these joints was devised to prevent interruption to the slipform paving operation. The sequence of operations is shown in Figures 4 through 11 and is described in the following.

Blockouts, consisting of wooden boxes, 3 ft (0.9 m) in length, were staked out in a row on the base at the desired joint location. As the concrete spreader passed over the joint, the tubes that placed the strands passed between the boxes. Paving and finishing were then completed normally over the boxes. After the paving train passed the joint area, the boxes were lifted out of the concrete by a crane. To lift them out they were hooked onto a framework with ropes and lifted in unison. The remainder of the concrete in the joint void was removed by hand, leaving a void slightly longer than 3 ft (0.9 m).

One of the 2 slab ends at the joint was formed by placing a permanent steel beam that distributes the load of the strands across the end of the slab. This beam is known as the female beam and will be discussed later. A steel bulkhead was placed at the other slab end to form a smooth concrete face. Concrete to fill the joint would later be poured against this face after the jacking was completed. The female beam and bulkhead were lowered in place by the crane and positioned properly, after which concrete was placed against them, vibrated, and finished. Slots in the female beam and bulkhead allowed them to be placed over the strands, which were not cut until the following day. This completed the joint-forming operation, and the crew moved ahead to repeat the sequence when the paving train reached the next joint.

Figure 1. Pavement cross section.

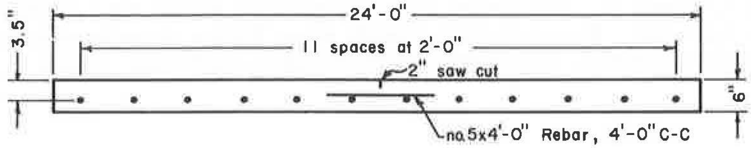


Table 1. Strand data.

Item	Dimension
Strand diameter	0.6 in.
Steel area	0.215 in.^{2}
Length per pound	1.36 ft
Modulus	28×10^6 psi
Ultimate strength	58.6 kips
Temporary force maximum (80 percent of ultimate)	46.9 kips
Stressing load (70 percent of ultimate)	41.0 kips
Design load (60 percent of ultimate)	35.2 kips

Figure 2. Paving operation.

Figure 3. Concrete spreader.

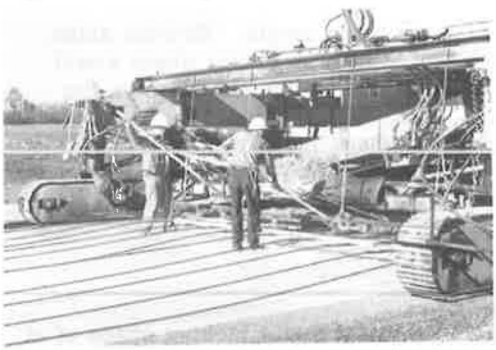


Figure 4. Joint blockout boxes.

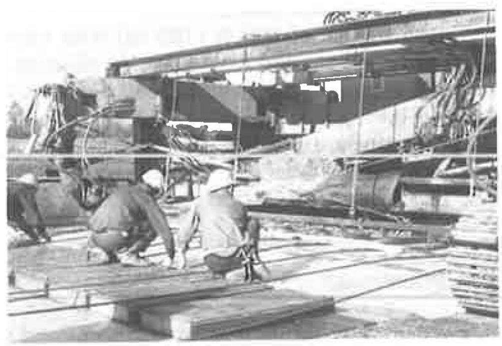


Figure 5. Preparing lifting ropes.

Figure 6. Hooking ropes on lifting frame.

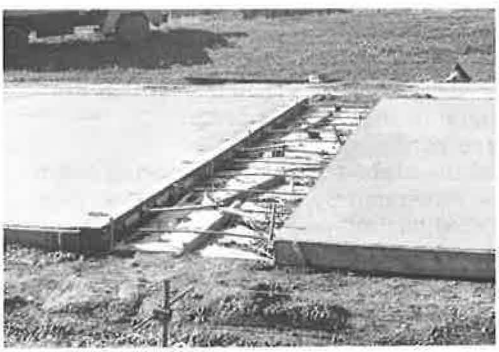
Figure 7. Lifting of blockout boxes.

Figure 8. Placing female beam and bulkhead.

Figure 9. Placing concrete at joints.

Figure 10. Finishing concrete at joint.

Figure 11. Completed joint placement.



A time limit was placed on the joint-forming operation to prevent the possibility of a cold joint when concrete was placed back against the bulkhead and female beam. The contractor was allowed 14 minutes after the paving train passed the joint to remove the wooden boxes and place the beam and bulkhead. Placing and screeding the concrete behind the beam and bulkhead were to be completed within another 6 minutes, or a total of 20 minutes after the paving had passed the joint.

Jacking

A 2-step loading was specified; the initial load of 10 kips (44.5 kN) was applied when the concrete reached a strength of 1,000 psi (6895 kPa) and the final loading of 46.9 kips (207.6 kN) was applied at 2,500 psi (17 238 kPa). Concrete strengths were determined from field-cured cylinders molded when the end of the slab was placed. Prior to placing the initial load, it was necessary for the contractor to sever the strands at the joints and place the strand anchors.

After several slabs were placed, a decision was made to allow an initial load of 20 kips (89 kN) to be placed if the concrete strength was over 1,500 psi (10 343 kPa). This decision further ensured that no shrinkage cracking would occur in the slabs if a sudden drop in temperature or a period of rapid hydration of the concrete occurred.

Jacking was done with portable hydraulic ram jacks capable of loading 1 strand at a time (Figures 12 and 13). The loading sequence used was to jack the center strands first and then jack on alternate sides out to the edges of the slab. The jack rams had a throw of only 10 in. (254 mm) and it was necessary to reposition the jack after each 10 in. (254 mm) of strand elongation was achieved. Jacking was done at both ends of each slab to reduce friction losses in the strands.

Joint Construction

After jacking was completed, the joints were prepared for concreting. A unique method was devised whereby the prestress would be applied to the joint concrete, resulting in movement only at 1 joint gap. To accomplish this, the load from the anchors at 1 end of the slab was transferred to permanent strand anchors cast into the joint concrete. The load transfer was accomplished by releasing the anchors at the end of the slab against which the joint concrete was placed. To release the load, a jacking bridge was used behind each strand anchor (Figure 14). This bridge was later cut after the joint concrete had cured sufficiently. The top and bottom horizontal bars were cut at one side of the strand to allow the load to transfer to the permanent anchor. To cut the jacking bridges, voids (Figure 15) had to be formed in the joint concrete. These were filled with concrete after the jacking bridges were removed.

The completed joint provides for movement of the slabs with an interlocking beam system at the joint. This system is a product of Pavement Systems Inc. and carries the trade name "PAJO". The female beam, placed during paving, becomes integral with the slab and is held to it by the force of the strands. Anchors in the female beam were set into anchor pockets so that they would not protrude from the beam. An anchor pocket is shown in Figure 16. These were filled with cement mortar after jacking.

Male beams were placed inside each female beam prior to placing the joint concrete. It was necessary to fabricate the male beams in short sections due to the interference of the vertical ribs that are part of the female beam (Figure 17). When placing the male beams, a ¼-in. (6.4-mm) spacer of expanded foam sheeting was placed between each male beam and the female beam to allow for expansion of the slabs. A piece of foam was also clipped onto each vertical rib of the female beam to maintain the same spacing at this point. To prevent spalling when the joint closes, a hand-tooled edge was made where the joint concrete meets the top plate of the female beam.

After the permanent anchors were placed on the strands, it was necessary to "set" the wedges on each to ensure that the strand would be held when the jacking bridge was cut. This was done by placing the jack between the temporary anchor and the permanent

Figure 12. Hydraulic jack.

Figure 13. Jacking setup.

Figure 14. Jacking bridge.

Figure 15. Jacking bridge after cutting.

Figure 16. Anchor pocket in female beam.

Figure 17. Placing male beams.

anchor and jacking against them to a load of 30 kips (13.3 kN). The permanent anchors were then tied to 2 transverse reinforcement bars to suspend them at the proper height (Figure 18). A mat of reinforcement bars was placed in the joint and supported by chairs (Figure 19). To prevent bonding of the strands to the joint concrete, pieces of conduit were slipped over them.

After preparations were complete, the joints were filled with class AA concrete, the same as used for conventional paving but with a higher slump. Jacking bridges were not cut until the joint concrete reached a strength of at least 3,000 psi (20 685 kPa).

Terminal joints were used at each end of the job and at the approaches to the structure (Figure 20). The design is applicable to any slab end that must adjoin conventional pavement. Expansion joint material was omitted at the south end of the job where the adjoining pavement is bituminous concrete.

COST DATA

The costs given in Table 2 are those of the original contract, bid for 9-in. (229-mm) reinforced concrete pavement on 9 in. (229 mm) of subbase, versus the negotiated price to substitute 6-in. (152-mm) prestressed pavement on 6 in. (152 mm) of aggregate bituminous base course and 6 in. (152 mm) of subbase.

OBSERVATIONS

Slipform Paving

The system of guide tubes on the concrete spreader proved to be a successful method of placing the strands at the desired depth and alignment. Depth checks were made periodically to determine whether there was any difficulty in achieving the specified 3.5-in. (89-mm) placement depth. The strands were generally within ± ¼ in. (6.4 mm) of the specification. A wider variation in strand depth occurred at the joints, however, because of the disturbance caused by placing the joint. Both low and high strand placement resulted at the joints on several occasions. Other aspects of the slipform paving, such as concrete mixing and delivery, vibrating, screeding, finishing, and curing, proceeded in conventional fashion.

Although the maximum length of pavement placed in 1 day was 2,529 ft (771 m), there is every reason to believe that runs of 3,000 ft (914 m) or more could easily be achieved. The low daily production for this slipform operation can be attributed to difficulties in placing the plastic sheeting on windy days, plant breakdowns, limited batch plant output, delays caused by winter curing, and late morning starting because of low temperatures.

Plastic Sheeting

The major problem encountered during paving was the placement of the plastic sheeting. The operation began by unrolling 2 overlapping rolls, 12.5 ft (3.81 m) wide, of double-layer polyethylene sheeting behind the strand pay-off truck and ahead of the spreader, allowing the strands to hold the plastic down. But, because the spreader dumped the concrete off a conveyor at the centerline and pushed it toward the edges with rotating augers, the plastic was pulled along with the concrete. This caused a space of up to 1 ft (300 mm) at the centerline without plastic. After trying various methods, taping the 2 rolls together proved to be the most successful. Rolls of plastic 25 ft (7.62 m) wide would have been more suitable but were not available.

Another problem with the plastic was its tendency to fold up as it was pushed by the concrete under the spreader. It was feared that these folds were becoming trapped in the concrete slab, causing potential weak planes. Whether this actually occurred or

Figure 18. Permanent anchor and joint reinforcement.

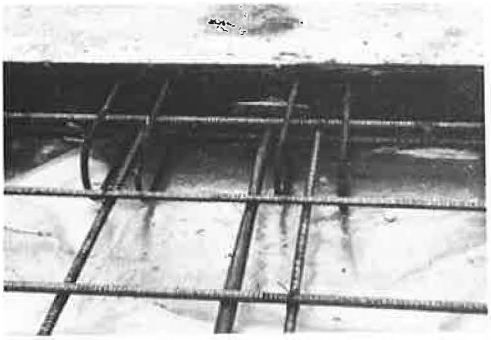


Figure 19. Joint ready for concrete placement.

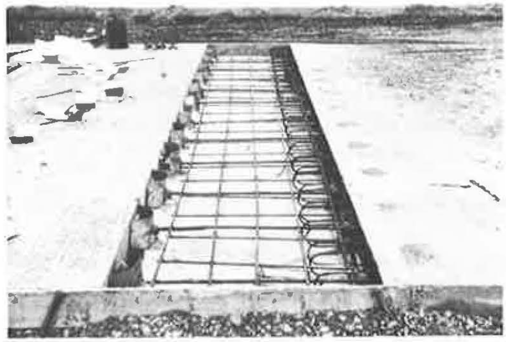


Figure 20. Terminal joint.

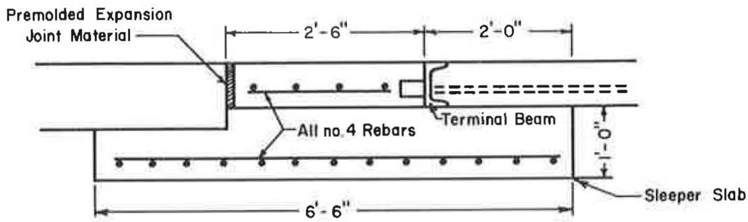


Table 2. Cost data.

Original Bid

Item	Quantity (yd²)	Price per yd² ($)	Total Price ($)
9-in. reinforced concrete pavement	37,608	8.80	330,950.40
9-in. subbase	73,796	1.58	116,597.68
10-ft shoulder, 9 in. (outside)	14,873	3.00	44,619.00
10-ft shoulder, 9 in. (median)	14,873	3.00	44,619.00
Total			536,786.08

Adjusted Price

Item	Quantity (yd²)	Price per yd² ($)	Total Price ($)
6-in. prestressed pavement	38,268	9.40	359,719.20
6-in. aggregate bituminous base	41,650	2.90	120,785.00
6-in. subbase	73,796	1.18	87,079.28
6-in. × 8-ft subbase under shoulder	23,800	1.50	35,700.00
10-ft shoulder, 6 in. (outside)	14,873	2.50	37,182.50
10-ft shoulder, 6 in. (median)	14,873	2.50	37,182.50
Total			677,648.48
Difference: Total			+140,862.40
Per square yard			+3.68

Table 3. Sample joint movements.

Date	Temperature (F) Air	Temperature (F) Concrete	Reading (in.)[a]
11-27-73	54	47	10.061
11-29-73	44	44	10.109
12-6-73	50	47	10.084
12-7-73	39	39	10.220
12-12-73	35	29	10.444
12-14-73	44	38	10.322
12-27-73	57	50	10.154
1-28-74	46	47	10.270
2-5-74	12	7	10.643

Note: Pavement placed 11-2-73, joint placed 11-21-73.

[a]Distance between measuring points.

not is a matter of speculation, but no adverse effects have been noted. This problem was never adequately solved. A concrete spreader that drops concrete from a moving hopper rather than pushing it with augers would have been more suitable.

Wind proved to be the most adverse problem with the plastic. The sheeting tended to billow up and fold over at the edges with only a moderate breeze. On several occasions when the wind was quite strong and gusty, a crew of 10 or more men was required to handle the sheeting. When the conditions were calm, 2 men were sufficient.

Joint Placement

A crew of about 6 men was required for the joint-placing operation. With some practice the crew could complete the operation in about 30 minutes. This included removal of the blockout boxes and concrete, placing the female beam and bulkhead assembly, placing concrete back against them, and finishing the concrete.

Construction Joints

Interruptions in slipform paving can occur at any time for a variety of reasons such as a plant breakdown, equipment failure, or sudden rain. The logical place to cease paving for the day was at a joint location, but the possibility of having to stop part way through a slab was always present. This occurred on 2 occasions, once due to a plant breakdown and once due to an unexpected cement shortage at the concrete plant. The choice became that of placing a joint wherever the breakdown might occur or making a construction joint or "cold" joint.

The decision was made to make a construction joint at the first breakdown because of the added expense of placing an extra full joint. Only 204 ft (62.2 m) of the second slab had been placed when a plant breakdown occurred. A bulkhead was placed and paving commenced the following day, placing concrete against the previous day's pavement. As the concrete cured, the construction joint opened up to a width of about ⅛ in. (3.2 mm). This was caused by shrinkage of the concrete. The slip plane provided by the plastic and the initial gain in strength of the concrete are apparently sufficient to prevent cracking initially in a slab 600 ft (183 m) long; however, the addition of a cold joint caused the 2 parts of the slab to behave independently of each other and move apart. The initial 20-kip (89-kN) tensioning of the strands failed to close the joint. Only after the final load was placed did the strands have sufficient force to pull the 2 slabs together. The joint closed shortly after final jacking and remained closed. This indicated that construction joints could be successfully placed in post-tensioned pavement slabs. The same procedure was followed on the second breakdown.

The construction joints both closed to an eventual opening of 0.005 in. (0.13 mm), where they remained. Full closure was prevented by the intrusion of incompressible material. On February 5, 3 months after placement, the temperature dropped to 7 F, at which time the construction joints were open to a width of 0.015 in. (0.38 mm). Temperatures had been dropping gradually over a 3-day period prior to reaching the low on February 5. The joints closed to their former size when the temperature rose again.

Jacking

The jacking operation proved to be time-consuming with the available equipment. Initial tensioning of the strands was normally required the day following paving, so jacking had to be done regardless of weather conditions. The joints were cleaned out and the anchors placed the morning after paving; jacking did not begin until afternoon. Approximately 2 hours or more were required for the initial jacking on both ends of each slab, using a 2-man crew and a single jack. Final jacking required about 4 hours per slab because the jack had to be reset for each 10 in. (254 mm) of strand elongation. Only 1 jacking crew was available most of the time, and the jacking fell behind schedule as the job progressed. Final strand tensioning was applied as long as 13 days after paving some slabs. Fortunately the 20-kip (89-kN) initial strand load on these slabs was sufficient to prevent shrinkage cracking.

Strand Elongation

The theoretical elongation for the strands is calculated as follows:

$$\text{Percent elongation} = \frac{P}{AE} \times 100 = \frac{46.9 \text{ kips}}{(0.215 \text{ in.}^2)(28 \times 10^3 \text{ ksi})} \times 100 = 0.78$$

Measurements of actual strand elongations were taken by inspection personnel. The average percent elongation for all strands tensioned was 0.68. This is 87 percent of theoretical, indicating a loss of 13 percent due to friction and take-up at anchorages. Further losses would be expected due to the load transfer at the joints, concrete shrinkage, and creep.

Jacking Bridges

The use of the jacking bridges to transfer the load on the strands through the joint concrete was successful, although several problems arose during construction that led to improvements in design and procedure. Shortly after final jacking began, the steel bars used as bridges were bending under the load of the anchors. The bending was more severe where the strand was not exactly perpendicular to the slab end. It appeared that adequate consideration had not been given to the relation between the span of the bridge and the strength of the materials. When a shorter span for the bridge was put into use, the problem was eliminated.

The original plan for severing the jacking bridges to transfer the load through the joint specified flame cutting. This idea was abandoned in favor of cutting them with an electric arc welder. In this way, much less heat was applied to the strand because the total cutting time for each bridge was only about 15 seconds. The amount of heat on the strands was not even sufficient to melt the polypropylene conduit. A gradual release of the load on the temporary anchors occurred as the force was transmitted through the joint concrete to the permanent anchors. The temporary anchor moved approximately ¼ in. (6.4 mm) toward the slab from its original position, due to elongation of the strand in the joint.

Slab Length Change

The movements of the slabs at each joint are being evaluated to determine the changes due to thermal expansion and contraction as well as other slab behavior such as shrinkage and migration. An invar bar equipped with a dial gauge indicator is used to record movements at the joints. Measurements are taken between a mark on the female beam and a brass plug embedded in the joint concrete and between brass plugs in the terminal joint and adjoining pavement. Indications are that the slab length change is very sensitive to ambient temperature changes. Sample movements for one joint are given in Table 3. It is evident that slab shrinkage has accounted for an increase in joint opening since the joint was placed.

Total shrinkage of the slabs was measured by recording the distance between slab ends at the time of placement and later at about 100 days. The differences in measurements were averaged (after adjustment for temperature at time of measuring) to determine an average shrinkage of 1.56 in. (39.6 mm) per slab or 217×10^{-6}. A high initial rate of shrinkage, as reported by Friberg and Pasko (4), occurred prior to placing the joints.

CONCLUSIONS

1. Prestressed concrete pavement can be placed successfully on a production basis using conventional slipform paving equipment. Paving rates equal to those for conventional reinforced concrete pavement can be achieved.

2. The need for a treated base course is the principle reason for the higher cost of prestressed pavement on this job. Average construction costs in this area for 9-in. (229-mm) reinforced concrete pavement are presently \$9.38 per yd², nearly equal to the cost of 6-in. (152-mm) prestressed pavement on this job (5). Continuously reinforced concrete pavement presently averages \$10.19 per yd² for 9-in. (229-mm) thickness. The following factors must be taken into account when considering the prices on this job: The price was negotiated rather than bid competitively; initial investment in equipment and modifications to the paving train were required; and there was probably apprehension on the part of the contractor due to the uncertain amount of construction time involved and the need for on-the-job training of labor inexperienced with prestressing work.

3. Prestressed pavement has the potential for longer pavement life with less maintenance because of the elimination of transverse contraction joints and the elimination of cracks, both major causes of pavement deterioration.

4. Prestressed pavement may be the answer to potentially severe shortages of reinforcement steel and cement in the future.

5. Construction joints can be placed in prestressed pavement slabs at desired locations to accommodate the slipform paving operations. No long-term adverse effects are anticipated, although the joints should be grooved and sealed with a suitable joint sealant to prevent intrusion of debris.

6. The opening of the construction joints at very low temperatures indicates that the coefficient of friction between the base and slab is not as low as anticipated. This could possibly be caused by the less-than-satisfactory methods used to place the polyethylene sheeting. It also creates concern over whether there is compression in the other slabs to resist cracking near midpoint at low temperatures or during sudden temperature drops.

7. Transverse reinforcement steel in prestressed pavement can be eliminated.

8. Load-transfer devices can be used to prestress the joint concrete, allowing all pavement concrete to be in compression.

REFERENCES

1. Prestressed Concrete Pavement Construction: Final Report. Federal Highway Administration, Region 15, FHWA-RDDP-17-1, Feb. 1973.
2. R. J. Brunner. Post-Tensioned Concrete Pavement. Pennsylvania Dept. of Transportation, Bureau of Materials, Testing and Research, Research Project 72-22, Oct. 1972.
3. B. F. Friberg and T. J. Pasko. Prestressed Concrete Highway Pavement at Dulles International Airport: Research Progress Report to 100 Days. Federal Highway Administration, Report FHWA-RD-72-29, Aug. 1973.
4. Bengt F. Friberg and Thomas J. Pasko. Prestressed Concrete Highway Pavement at Dulles International Airport. Highway Research Record 466, 1973, pp. 1-19.
5. Construction Cost Catalog. Pennsylvania Dept. of Transportation, Bulletin 50, March 1973.

DEPTH OF CONCRETE COVER OVER BRIDGE DECK REINFORCEMENT

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Fifty concrete bridge deck spans in New York State were surveyed with a pachometer for depth of clear concrete cover. Compliance with a design requirement for a minimum of 2 in. (51 mm) occurred at 77.3 percent of the locations measured. Spans having a high degree of compliance also tended to have relatively uniform cover depths. The degree of compliance appeared to be related to construction practices. The distribution of cover depths on individual spans was generally not normal, and a construction tolerance of $\pm\frac{1}{2}$ in. ($\pm$13 mm) was determined to be reasonable for the type of requirement under which the decks were built.

•CONCRETE bridge deck durability continues to be a major concern of those responsible for maintenance of the nation's highways. Much research has been directed toward determining both the extent and the causes of concrete bridge deck deterioration (1). In June 1972, the New York State Department of Transportation completed a study of bridge deck conditions in New York (2) and concluded, as have others, that surface spalling is the most troublesome problem and that attention should be directed predominantly toward overcoming that defect.

Surface spalling usually manifests itself as a "depression caused by a separation and removal of the surface concrete" (1). The deck is weakened locally, reinforcement is usually exposed, and riding quality is impaired. Repair is difficult and expensive, and there is no assurance that a repaired deck may not subsequently spall at other locations. Surface spalling is almost always associated with and preceded by corrosion of reinforcement. Factors that have been identified as major contributors to these conditions include (a) penetration of chlorides into relatively permeable concrete, (b) inadequate concrete cover over top reinforcing bars, and (c) cracks over the top bars (3).

Of these three major factors contributing to surface spalling, that most easily dealt with is depth of clear cover over top reinforcing steel. Several investigators have found a high degree of association between the occurrence of spalling and "insufficient" cover, including Stark (4), Carrier and Cady (5), and Newlon et al. (6). Although a number of factors interact to determine whether a spall develops at a particular location, experience has shown that very few develop when the depth of cover is $1\frac{1}{2}$ to 2 in. (38 to 51 mm) or more, even in very old decks (4). In response to such evidence New York, along with much of the nation, in 1968 increased its requirement for clear cover over top reinforcement to 2 in. (51 mm).

Since 1965, New York has built concrete decks consisting of an 8-in. (203-mm) monolithically placed structural deck and wearing surface without a protective membrane. Because in such a design a combination of low concrete permeability and adequate cover is relied on to inhibit spalling, a survey was initiated in September 1973 to determine the degree of compliance with the 2-in. (51-mm) cover requirement and to provide background data for subsequent studies of the performance of these decks. Since then, results of a similar survey in New Jersey have been published (7).

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The Federal Highway Administration also has initiated a project to determine among other things variability in concrete cover and to demonstrate design and construction practices that contribute to wide variability or deficiencies in cover depth (FHWA Region 15 Demonstration Project 33, "Bridge Deck Evaluation Techniques").

The purpose of this paper is to present a brief summary of the results of the depth-of-cover survey in New York, to offer some information concerning the conduct of such surveys that may be useful to others, and to share some tentative conclusions that can be drawn regarding possible causes of noncompliance with the New York requirement.

PROCEDURES

Between 1968, when the design requirement for 2 in. (51 mm) of cover was adopted, and the end of 1972, 93 bridges consisting of 174 spans were completed. Fifty of these spans were selected at random for inclusion in the survey and are categorized in Table 1. Depth of cover was measured with one of 2 hand-held "pachometers" (James Electronics Inc. Model C4949) at 200 randomly chosen locations on each of the 50 spans selected for study. Each measurement was made on the transverse (topmost) bar nearest to the chosen point and midway between the two adjacent longitudinal bars.

The instruments were found to be sensitive to the spacing of both longitudinal and transverse bars as well as to bar diameter, and calibration curves were developed for each; a sample set is shown in Figure 1. Using these curves, measurements of cover depth were found to be within 0.05 in. (1.27 mm) of the true depth 100 percent of the time when the instruments were used within their calibrated range of 1.25 to 3.00 in. (31.75 to 76.2 mm). When depth of cover was less than 1.25 in., a suitable shim was used to bring the reading within the calibrated range of the instrument. When the depth of cover exceeded 3.00 in., it was assigned a value of 3.08 in. (This practice introduced errors of unknown magnitude in subsequent calculations of means and ranges. However, on only 11 spans—Nos. 1, 2, 3, 19, 22, 24, 25, 28, 41, 47, and 49—did it occur in an amount greater than 2.5 percent of the measurements.) If a measurement site occurred within a wheel path, a correction was applied based on an estimate of rutting, determined by gauging offsets from a 4-ft (1.22-m) steel straightedge placed perpendicular to the direction of traffic.

RESULTS AND DISCUSSION

The survey resulted in 9,977 measurements of cover depth on the 50 spans; results are summarized in Table 2 and Figures 2 and 3.

Compliance

The requirement governing placement of these spans simply directs that all steel reinforcement be located at a minimum depth of 2 in. (51 mm) below the finished surface. No tolerances or maximum depth of concrete cover were specified. Further, there had been no requirement that the top mat be physically tied to the bottom mat or bar supports. Before the survey it was decided that "substantial" compliance would be defined as no more than 2.5 percent of all measured values (on all surveyed structures) and no more than 5.0 percent on any single span being less than 2 in. (51 mm). These somewhat arbitrary criteria were developed in consultation with the Department's chief bridge engineer as reasonable statements of design expectations.

Figure 3 and Table 2 show that substantial compliance in these terms was not obtained; 22.7 percent of all measured values fell below 2 in. (51 mm) and 30 of the 50 spans had more than 5.0 percent of values less than 2 in. On the other hand, the fact that total compliance was obtained on 7 spans (Nos. 5, 22, 24, 28, 39, 41, 49) demonstrates that this is possible.

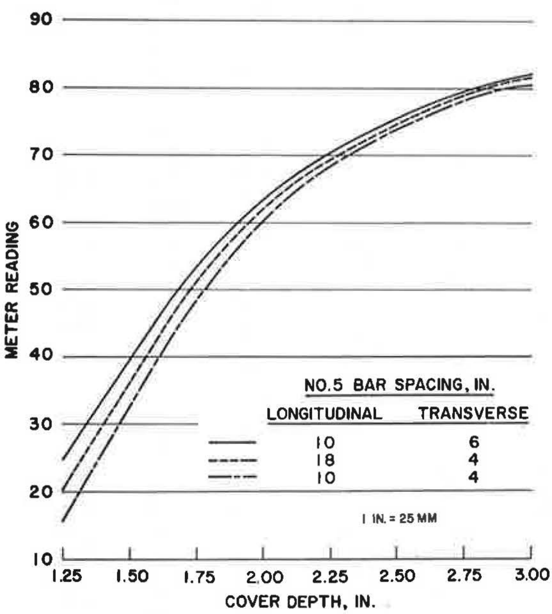
A recent design requirement now specifies securing the top mat of steel to the

Table 1. Characteristics of sampled spans.

	Type of Structure				
Span Length, ft	Simple Beam or Girder	Continuous Beam or Girder	Continuous Box Girder	Simple Prestressed Concrete Box Beam	Totals
0-40	1	0	0	0	1
41-80	11	0	0	0	11
81-120	13	2	0	4	19
121-160	14	0	2	0	16
161-200	1	0	2	0	3
Totals	40	2	4	4	50

Note: 1 ft = 0.30 m.

Figure 1. Sample calibration curves for first pachometer.



bottom mat, and specific instructions have been issued to field staff personnel to give particular attention to obtaining the required cover. The current requirement for depth of cover is $3\frac{1}{4} \pm \frac{1}{4}$ in. (83 ± 6 mm).

Sampling for Depth of Cover

One may wish to sample a bridge deck for depth of cover over steel reinforcement for a number of reasons:

1. To determine cover at a particular location because of its relevance to other features on the same deck (e.g., spalling);
2. To determine compliance with a specification as a basis for acceptance or payment; or
3. To determine the distribution of cover depth as a basis for correlation with present or future condition.

In the second and third cases, it is probable that estimates would be attempted of

Table 2. Summary of survey results.

Span	Total Measurements	Percent of Sites or Measurements Having Value Less Than Shown					Cover Depth, in.[a]		
		2.00 in.	1.75 in.	1.50 in.	1.25 in.	1.00 in.	Mean	Range	Median
1	200	13.0	3.0	0.0	0.0	0.0	2.34	1.50	2.25
2	200	25.5	7.0	0.0	0.0	0.0	2.27	1.56	2.31
3	200	0.5	0.0	0.0	0.0	0.0	2.64	1.12	2.56
4	200	2.0	0.0	0.0	0.0	0.0	2.36	1.19	2.31
5	200	0.0	0.0	0.0	0.0	0.0	2.52	0.94	2.50
6	200	6.5	1.5	0.0	0.0	0.0	2.41	1.44	2.44
7	200	36.0	18.0	8.0	2.5	1.0	2.13	2.19	2.19
8	202	48.5	33.7	24.8	11.9	1.0	1.96	2.19	2.00
9	200	11.5	1.0	0.0	0.0	0.0	2.21	1.38	2.19
10	203	41.9	27.1	15.3	7.9	1.5	2.00	2.06	2.06
11	200	59.5	41.5	24.0	11.0	2.0	1.86	2.19	1.81
12	198	42.4	25.2	10.1	1.5	0.0	1.99	1.69	2.06
13	202	34.6	4.0	0.0	0.0	0.0	2.03	1.19	2.06
14	200	4.0	0.0	0.0	0.0	0.0	2.34	1.12	2.31
15	202	3.5	1.0	0.0	0.0	0.0	2.36	1.44	2.38
16	200	69.5	32.5	3.0	0.0	0.0	1.85	1.25	1.81
17	200	26.5	3.5	2.0	0.5	0.0	2.12	1.75	2.12
18	200	9.5	1.5	0.0	0.0	0.0	2.34	1.50	2.38
19	200	16.0	2.0	0.5	0.0	0.0	2.39	1.62	2.31
20	200	11.5	3.5	0.0	0.0	0.0	2.33	1.56	2.31
21	200	38.0	6.5	0.0	0.0	0.0	2.07	1.50	2.06
22	200	0.0	0.0	0.0	0.0	0.0	2.66	0.94	2.56
23	202	14.8	2.5	0.0	0.0	0.0	2.23	1.44	2.19
24	200	0.0	0.0	0.0	0.0	0.0	2.86	0.81	2.94
25	200	5.5	0.5	0.0	0.0	0.0	2.48	1.38	2.50
26	200	11.0	0.5	0.0	0.0	0.0	2.33	1.38	2.38
27	200	7.0	1.0	0.0	0.0	0.0	2.36	1.31	2.38
28	200	0.0	0.0	0.0	0.0	0.0	2.73	1.00	2.75
29	200	67.5	44.0	23.0	14.0	9.5	1.72	2.25	1.75
30	200	46.5	21.0	4.0	0.5	0.0	1.97	1.50	2.00
31	200	58.5	25.0	2.0	0.0	0.0	1.93	1.56	1.94
32	164	11.0	1.8	0.0	0.0	0.0	2.33	1.38	2.44
33	200	86.0	64.5	33.5	6.0	0.0	1.62	1.44	1.56
34	200	78.0	55.5	25.5	7.5	0.5	1.69	1.69	1.69
35	200	1.0	0.0	0.0	0.0	0.0	2.46	1.00	2.44
36	200	1.0	0.0	0.0	0.0	0.0	2.38	1.12	2.38
37	200	30.0	2.5	0.0	0.0	0.0	2.18	1.50	2.12
38	200	30.0	6.0	0.0	0.0	0.0	2.04	1.00	2.06
39	199	0.0	0.0	0.0	0.0	0.0	2.52	0.88	2.50
40	200	1.0	0.0	0.0	0.0	0.0	2.44	1.12	2.38
41	200	0.0	0.0	0.0	0.0	0.0	2.72	0.94	2.75
42	200	4.0	0.0	0.0	0.0	0.0	2.39	1.19	2.38
43	200	85.0	69.5	50.0	28.0	7.0	1.50	1.81	1.44
44	204	1.0	0.0	0.0	0.0	0.0	2.48	1.12	2.44
45	202	19.8	3.0	0.0	0.0	0.0	2.18	1.56	2.19
46	199	2.0	0.5	0.0	0.0	0.0	2.44	1.38	2.44
47	200	1.5	0.0	0.0	0.0	0.0	2.46	1.31	2.50
48	200	6.0	0.0	0.0	0.0	0.0	2.36	1.31	2.38
49	200	0.0	0.0	0.0	0.0	0.0	2.65	0.88	2.69
50	200	66.5	23.0	2.5	0.0	0.0	1.89	1.06	1.88
Average	200	22.74				0.45	2.25	1.39	2.25

Note: 1 in. = 25.4 mm.

[a]Measured cover depths over 3.0 in. were assigned a value of 3.08 in. and included in the computations.

Figure 2. Distribution of cover depths for all spans combined.

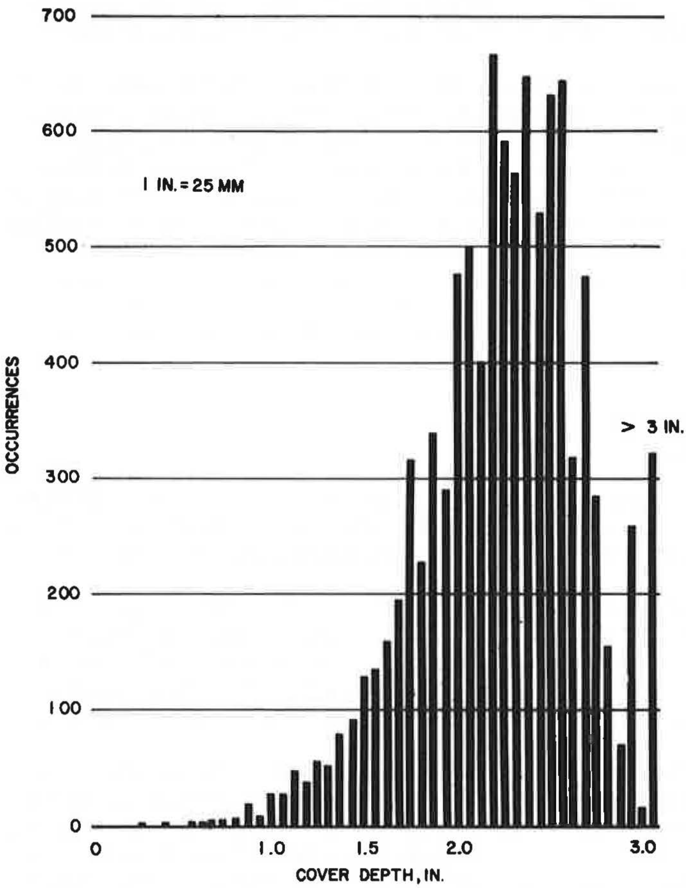
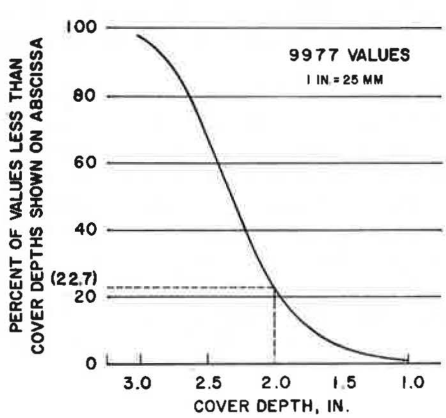


Figure 3. Cumulative distribution of concrete cover depths.



either the proportion of defects (i.e., insufficient cover) or the parameters of cover depth distribution (i.e., mean and standard deviation). In either case, it is important to know the nature of the distribution of cover depth or, more specifically, whether it can be assumed to be normal.

One reason for taking 200 random measurements on each span was to have a sound basis for judging normality of the distributions (8). On 36 of the 50 spans examined, the distributions tested as not normal at the 0.05 level of significance using the chi-square method. This observation suggests that any investigator attempting to characterize the depth of concrete cover (obtained under requirements of the type in effect for these decks) on an entire span or deck, in order to draw statistical inferences from a sampling of that deck, should not assume that cover depth is normally distributed. It further suggests that sampling plans based on attributes rather than variables are more appropriate for depth-of-cover surveys. The characteristics and applications of sampling plans using attributes and variables have been discussed thoroughly by DiCocco (9).

Construction Placement Tolerances

From only a casual inspection of the data from each of the 50 spans (Table 2), it was apparent that the range of values for depth of cover is substantially different from span to span. The smallest encountered was 0.81 in. (21 mm) and the largest 2.25 in. (57 mm).

An attempt was made to judge what a reasonable placement tolerance might be for the type of requirement under which these spans were placed. In many industries, such information is obtained from what is called a "process capability study", in which the process is closely controlled so as to reduce product variability to that inherent in the process. Quality levels and tolerances are then set on the basis of results from the process capability study.

The closest possible analogy to such a study was to examine the variability of the 20 spans (Table 3) that met the satisfactory compliance criterion of having no more than 5 percent of cover depths measuring less than 2 in. (51 mm). These spans were among those with the smallest ranges, and in fact good concurrence was generally found between compliance and cover depth range. In other words, spans having a high proportion of steel with the acceptable minimum cover tended also to have less variation in steel placement depth, as shown in Figure 4. These 20 spans thus were considered to represent "good" construction in terms of both compliance and variability in cover depth.

Table 3 gives the 100th and the inner 99th and 95th percentile ranges for measurements on these 20 spans. Although the distribution of values on any particular span may not have been normal, it tended to be symmetrical, and a particular inference was drawn from the inner 95th percentile column of Table 3. When "good" construction practices are observed under this type of specification, a range of 1.00 in. should occur no more than 5 percent of the time. This suggests that a 95 percent tolerance of $\pm\frac{1}{2}$ in. may be reasonable. Further, this appears to be easily attainable since it was found in 63 percent of the 50 spans observed, even when no tolerance was specified (Figure 5). This is consistent with expectations of the American Concrete Institute in the "Recommended Practice for Concrete Highway Bridge Deck Construction" (10).

Causes of Noncompliance

At the outset of the survey, it was decided that, if the informal criteria chosen to judge compliance were not met, an effort would be made to find possible causes. To that end, a large quantity of design, construction, and materials information was collected concerning the 50 measured spans. A preliminary analysis of selected portions of that information seems to confirm the opinion, often heard from experienced construction engineers, that attainment of proper cover depth depends primarily on the care and

Table 3. Scatter in cover depth for 20 spans.

Span	Inner Range, in.[a] 100th Percentile	99th Percentile	95th Percentile
3[b]	1.12	1.00	0.94
4	1.19	1.00	0.75
5	0.94	0.88	0.69
6	1.44	1.31	0.94
14	1.12	1.12	0.88
15	1.44	1.31	0.88
22[b]	0.94	0.88	0.81
24[b]	0.81	0.75	0.63
27	1.31	1.25	0.81
28[b]	1.00	0.94	0.88
35	1.00	1.00	0.94
36	1.12	1.12	1.00
39	0.88	0.81	0.56
40	1.12	1.12	0.88
41[b]	0.94	0.75	0.75
42	1.19	1.12	1.00
44	1.12	1.12	0.75
46	1.38	1.31	0.88
47	1.31	1.19	1.06
49[b]	0.88	0.88	0.81
Mean	1.11	1.04	0.84

[a]Range containing 100, 99, or 95 percent of measured cover depth.
[b]Estimates of ranges for these spans are low by an undeterminable amount because a substantial number of measurements exceeded the 3.0-in. upper limit of the useful range of the equipment (see discussion under the heading "Procedures").

Figure 4. Specification compliance and range in cover depths for 50 spans.

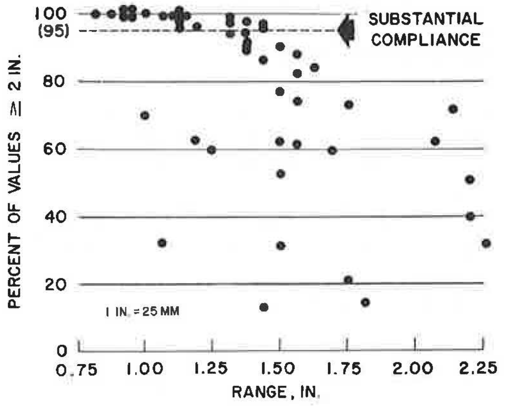


Figure 5. Cover depth range within individual spans.

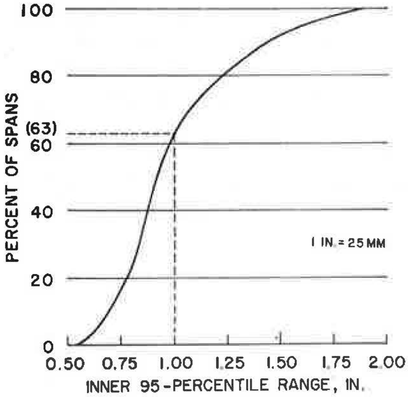
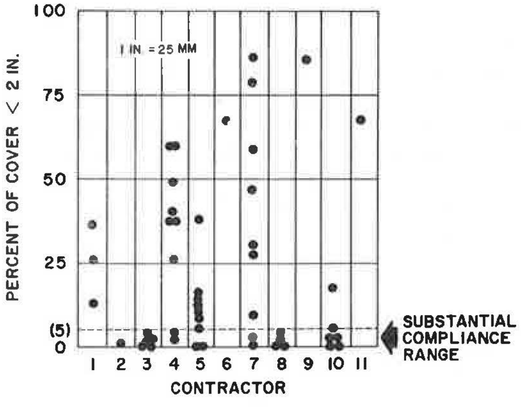


Figure 6. Contractor performance.



attention given that feature during construction.

The best evidence of this is shown in Figure 6. The 50 surveyed spans were built by 11 contractors on 14 different projects. The proportion of steel in each span with less than 2 in. (51 mm) of cover is plotted in Figure 6 by contractor. It is fairly obvious that contractors have had varying degrees of success in meeting the New York requirement. Contractors 3, 8, and 10 repeatedly demonstrated "good" compliance. Contractors 4, 5, and 7, on the other hand, demonstrated "variable" compliance. The point is that contractors appear to differ significantly in their ability to comply consistently with the requirement.

If the percentage of cover less than 2 in. (51 mm) is the measure of contractor performance, the hypothesis of no difference in performance among contractors can be tested by a one-way variance analysis (11). By using data from those spans associated with contractors who contributed 3 or more spans to the 50-span sample, the following analysis of variance table was constructed:

Source of Variation	Sum of Squares	Degrees of Freedom	Variance Estimate	F
Between contractor	9,779	6	1,630	4.5
Within contractor	14,278	39	366	
Total	24,057	45		

Since the computed variance ratio (4.5) exceeds the critical variance ratio at the 5 percent level (2.2), it is concluded that performance differed significantly among contractors.

The only other analysis made to date deals superficially with the relationship of "high" steel having a depth of cover less than 2 in. (51 mm) and its location within and among spans. Each of the 50 spans was divided into a 3-by-3 grid of 9 approximately equal areas. The frequency of occurrence of high steel in each grid cell was tested for random occurrence by the method of chi-square at the 0.05 significance level (8). The same analysis was performed on the pooled data for all 50 spans. The results indicate that the occurrence of high steel on a particular span tends to be concentrated in small areas, but the locations of these areas vary from span to span in an unpredictable manner. This analysis was performed separately on simple and continuous spans with the same results.

CONCLUSIONS AND INTERPRETATIONS

New York's experience over the 5-year period from 1968 to 1972, during which 2 in. (51 mm) of clear concrete cover was required over topmost steel reinforcement, with no requirement for cover tolerance or mat tie-down, has been as follows:

1. The degree of compliance with the requirement has been rather poor. Of all measurement locations sampled, 23 percent had less than the minimum 2-in. (51-mm) cover. Of the individual spans sampled, 60 percent had less than the required cover at more than 5.0 percent of their measurement locations. New requirements for mat tie-down are expected to improve this situation.
2. Cover depths were generally found to be other than normally distributed; thus, for determining compliance, attribute sampling plans appear to be more appropriate than those based on variables.
3. A tolerance of $\pm\frac{1}{2}$ in. (13 mm) should be attainable 95 percent of the time where "good" construction practices are observed.
4. Bridge deck spans for which a high degree of compliance was attained also tended

to have relatively uniform depth of cover.

5. A high degree of association was found between extremes of compliance (i.e., "good" and "variable") and specific but different groups of contractors. This evidence was taken to support the position that the attainment of required cover is highly related to construction practices.

6. The occurrence of "high" steel tends to be localized on a particular span but varies in location from span to span in an unpredictable manner.

ACKNOWLEDGMENTS

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REFERENCES

1. Concrete Bridge Deck Durability. Synthesis of Highway Practice 4, National Cooperative Highway Research Program, 1970.
2. W. P. Chamberlin, D. E. Amsler, and J. J. Jaqeway. A Condition Survey of Monolithic Bridge Decks in New York State. Special Report 11, Engineering Research and Development Bureau, New York State Department of Transportation, Aug. 1972.
3. Durability of Concrete Bridge Decks—A Cooperative Study: Final Report. Publication EB067.01E, Portland Cement Association, 1970.
4. D. Stark. Studies of the Relationships Among Crack Patterns, Cover Over Reinforcing Steel, and Development of Surface Spalls in Bridge Decks. HRB Spec. Rept. 116, 1971, pp. 13-21.
5. R. E. Carrier and P. D. Cady. Deterioration of 249 Bridge Decks. Highway Research Record 423, 1973, pp. 46-57.
6. H. H. Newlon, Jr., J. Davis, and M. North. Bridge Deck Performance in Virginia. Highway Research Record 423, 1973, pp. 58-72.
7. R. M. Weed. Recommended Depth of Cover for Bridge Deck Steel. Transportation Research Record 500, 1974, pp. 32-35.
8. W. G. Cochran. The χ^2 Test of Goodness of Fit. Annals of Mathematical Statistics, Vol. 23, No. 3, Sept. 1952, pp. 315-345.
9. J. B. DiCocco. Quality Assurance for Portland Cement Concrete. Research Report 10, Engineering Research and Development Bureau, New York State Department of Transportation, Sept. 1973. (Published and distributed by FHWA as Report FHWA-RD-73-77.)
10. ACI Committee 345. Proposed ACI Standard: Recommended Practice for Concrete Highway Bridge Deck Construction. Title 70-41, Journal of the American Concrete Institute, Proc. Vol. 70, June 1973, pp. 381-415.
11. W. C. Guenther. Analysis of Variance. Prentice-Hall, Inc., Englewood Cliffs, N. J., 1964, pp. 32-43.

THE USE OF TIME-LAPSE PHOTOGRAPHY AND COMPUTER SIMULATION FOR LOADER-TRUCK PRODUCTION STUDIES

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This paper illustrates how a combination of 2 relatively new analysis techniques for the construction industry—computer simulation and time-lapse photography—may be used to provide a construction estimator and planner with additional tools for determining the optimum method of earthmoving with loaders and trucks. The paper discusses the method of data collection using time-lapse photography and the production results obtained. It also discusses the development of the SIMSCRIPT computer simulation program based on the results of these field observations and indicates how a program of this type may be validated. The authors conclude that the results obtained should be helpful to contractors in their decision-making responsibilities.

•ONE of the most commonly used production systems for the earth-moving phase of a highway construction project is the loader-truck operation. This operation may be viewed as the link-node system of interdependent linked operations shown schematically in Figure 1.

Link 1 represents the loader operating in the cut area, and node 1 represents the interface activity of the loader dumping into the trucks. In a similar fashion, link 3 represents the compaction equipment at the fill and node 2 represents the interface operation between the trucks and the compaction equipment.

Two basic approaches may be taken when an estimator makes an economic comparison between alternative methods of construction for earth-moving operations of this type. The most common one (i.e., the one suggested by equipment manufacturers) views the operation as being deterministic, where single-value time durations are assigned to each component of the production cycle. A number of researchers (1, 2, 3) have felt that this approach results in inaccurate estimates, and they have suggested that a stochastic approach should be used (i.e., where an attempt is made to model the natural random variations of time for each component of the cycle).

For relatively simple construction operations, the mathematical modeling approach explained by Griffis (4) may be used for a stochastic analysis. For more complicated operations (particularly when alternative methods of construction are to be evaluated), a more efficient stochastic approach involves the use of computer simulation. O'Neil (5), for example, has developed an open-mine loader-truck simulation in FORTRAN, and Gaarslev (1) has developed one in GPSS. No one as yet has attempted to develop such a simulation in the more flexible simulation language known as SIMSCRIPT.

All of these methods of computer simulation have one thing in common, however, since they all require some knowledge of the probability distributions of the various time components of the production cycle. As noted by Parker and Oglesby (6), one of the techniques that may be used for a time-study analysis of construction operations is time-lapse photography. It may be used by a contractor to determine accurate field

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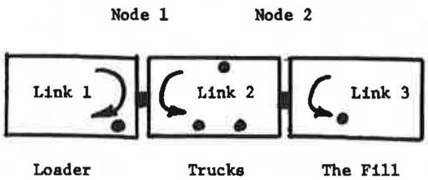
*Mr. Lee was a graduate student at Pennsylvania State University when this research was performed.

Figure 1. A 3-link system, with 2 dependent nodes.



productivity and field cost information.

The principal objective of this paper is to illustrate how these 2 relatively new analysis techniques for the construction industry, computer simulation and time-lapse photography, may be used in combination to provide a contractor's cost estimator with additional tools to determine an optimum method of production. This is done by relating some of the steps taken by the writers in the research on several Pennsylvania highway construction projects. The paper is divided into the following parts:

1. A discussion of the method of data collection on loader-truck operations using time-lapse photography;
2. A discussion of the results of the time-lapse data analysis; and
3. A discussion of the SIMSCRIPT computer program that was developed based on the field observations.

COLLECTION OF FIELD DATA

The loader-truck operation on 3 highway projects being built for the Pennsylvania Department of Transportation by 3 different Pennsylvania contractors was observed to define the important elements of the operation and to collect the cycle time data required. All 3 projects employed large loaders to charge sizable off-highway trucks with blasted rock. The objective of this paper, as noted earlier, is not only to indicate the type of data that may be obtained by time-lapse photography but also to show the type of computer simulation program that would allow an estimator to consider all of the typical situations in the description of the projects when he is evaluating alternative plans for a particular project.

Description of the Projects

On Project A, 4 Euclid 70-ton (63.5-tonne) bottom-dump trucks were being loaded with a well-blasted mixture of sandstone and limestone by a Hough 400 loader with a 10-yd^3 (7.65-m^3) bucket. A bulldozer assisted the loader in working the face of the cut, which was 200 ft (61 m) wide. The haul road, which was 4,750 ft (1448 m) long, had downhill grades that varied up to a maximum of 8 percent and a rolling resistance that varied from 2.5 to 5.5 percent. The fill area was 250 ft (76 m) wide by 600 ft (183 m) long. The data were collected while the first lift was being placed in this fill. The original ground was of a swampy nature, so it was necessary for the bottom-dump trucks to turn and dump at the same time. This caused the trucks to spend considerably more time at the fill than was required to dump the trucks. There was one bulldozer at the fill that, in addition to maintaining the grade, assisted in extricating the trucks that became stuck.

On Project B, 3 Euclid 50-ton (45.4-tonne) rear-dump trucks were being loaded by a Michigan 475 loader with a 12-yd^3 (9-m^3) bucket. On the first visit (called Project B1) the material being loaded was earth loam, and on the second visit (called Project B2) the material was a very hard, not well-blasted, grey limestone. The cut was approximately 300 ft (91 m) wide, allowing ample maneuvering space for the trucks. A bulldozer was not used to assist the loader. The haul road was approximately 1,600 ft (488 m) long and was almost entirely downhill, with grades up to 24 percent and a rolling resistance of about 3.0 percent. The fill covered a large area. There was 1 bulldozer and 1 compactor working in the fill area. The trucks backed into position and dumped without the direction of a foreman or laborer on the ground. The return trip began before the box of the truck had reseated itself.

On Project C, 3 observations (called Projects C1, C2, and C3) were made over a 3-month period. The well-blasted sandstone was loaded by a Michigan 475 loader with a 12-yd^3 (9-m^3) bucket into the following equipment: on Project C1, 2 WABCO and 2 Euclid 50-ton (45.4-tonne) rear-dump trucks; on Project C2, 3 WABCO and 3 Euclid 50-ton (45.4-tonne) rear-dump trucks; and on Project C3, 4 Euclid 50-ton (45.4-tonne) rear-dump trucks. The cut area was approximately 150 ft (46 m) wide and 250 ft (76 m) long, and a bulldozer was pushing material to the loader at all times. On Project C1 the haul road was 2,600 ft (792 m) long and had grades that varied from 15 percent up to 14 percent down and rolling resistances from 2.5 percent to 5.0 percent. On Projects C2 and C3, the haul road was 6,300 ft (1920 m) long and had grades that varied from 3 percent up to 20 percent down and rolling resistance of about 3 percent. The fill on Projects C1 and C3 was nearly identical to the fill on Project B. The trucks backed into position, discharged the material, and pulled away a short distance. The trucks stopped at this point until the box was lowered to its "rest" position and then resumed their return trip. On Project C2, the fill operation was unique, at least with respect to the other operations observed in this study. The material was being used as backfill for a concrete box culvert 500 ft (152 m) long. There was considerable congestion in this area. The fill was located near the project office. There was traffic caused by the supervisors, mechanics, inspectors, etc., traveling through the area. The trucks dumped their loads near the culvert. Small front-end loaders were then used to transport the backfill material to the excavation; 2 compactors were also used.

Data Collection

A stopwatch is the most commonly used means of collecting component time data on construction operations. A stopwatch is an excellent means of recording sequential events involving 1 or 2 men, a man and a machine, or 2 machines. There are, however, several problems that arise in stopwatch studies:

1. There is a limit to the number of events that a single observer can record;
2. The observer must decide instantly when one cycle or event stops and another begins; and
3. The only permanent record of the observation is the notes kept by the observer.

Time-lapse photography was used in this research in an attempt to overcome these shortcomings. The basic idea behind the time-lapse method is to allow some time to elapse between the exposure of 2 consecutive frames of film. The effect is to compress the time required to observe an event. A time-lapse movie has no limitation on the number of events that may be recorded as long as they are within the field of view of the camera. The film provides a permanent record of the operation, and there is no immediate field decision required regarding the beginning or the end of events.

The time interval between frames for this study was 2 seconds. This interval, rather than a 4-second interval, was chosen because the error inherent in the cycle time calculations is reduced as this time interval is reduced. On many occasions when taking time-lapse movies, the beginning or end of a cycle will occur between frames. By decreasing the time interval between exposures, the number of these occurrences is decreased.

Two super-8-mm movie cameras manufactured by Time-lapse, Inc., of Palo Alto, California, were used so that both the loading and the dumping of the trucks could be photographed at the same time. It was felt that, if 2 cameras could be synchronized, the haul and return travel times of the trucks could be recorded by comparing the 2 films. The 2 operators of the cameras were supplied with 2-way radios so that they could be in constant communication. It was necessary for the operators to maintain communication to ensure that the cameras were started and stopped at the same time.

Colored magnetized plastic signs were fastened to the trucks. The signs were 18 in. (457 mm) square and were placed on each side of the truck so that each truck could be identified in both the loading and dumping films.

The films were analyzed by using a projector also manufactured by Time-lapse, Inc. This projector is capable of a variable speed of viewing, and it has a built-in frame counter. This made it possible to count the number of frames between events (for example, between the beginning and end of a load cycle).

Data Collection Problems

As noted, the original intent was to obtain haul and return times as well as loading and dumping cycle times directly from the synchronized films. Although the latter objective was achieved, unforeseen problems prevented the accomplishment of the former objective. These problems should be explained because other researchers may also want to apply the synchronized camera approach to construction operations.

The sales literature from Time-lapse, Inc., stated that there is a ±2 percent maximum error in the time interval. The analysis of the processed films indicated that, although this 2 percent error did not adversely affect each film, it had a profound effect on the travel times, which required a comparison of 2 films. The error is cumulative because it depends on the duration of the component time being observed. In the case of a load cycle of 40 seconds (i.e., 20 frames if the timing interval is 2 seconds per frame), if the camera has a positive error of 2 percent, the error per cycle is 0.8 second or 0.4 frame. This means that the actual cycle time is 40.8 seconds, not 40 seconds. This error is less than the 2-second interval between frames, and therefore it was deemed insignificant.

For the travel time calculations, however, it was necessary to maintain a continuous frame count. The frame number for an event (truck arrival at the fill or truck departure at the cut) is noted for each film. The travel time was calculated by subtracting the frame number of the departure on one film from the frame number of the arrival on the other film for both the haul and return segments of the truck cycle. An event occurring at a frame count of 3,000 frames (i.e., 6,000 seconds from the start of filming) with a positive 2 percent error has an actual error of 120 seconds. If the second camera has a negative 2 percent error, then the total resultant error in the travel time components will be 4 minutes. This distortion would have resulted in an actual travel time of approximately 3 minutes being recorded as a negative quantity.

This situation was discovered after all the films had been processed and were being studied. It was felt at first that a calibration curve for each camera could be established that would allow a correction factor to be applied in order to establish the respective haul and return times. This proved to be impossible, however, because it appears that the error exhibited by a time-lapse movie camera is dependent on the charge of the battery at the time the pictures are taken. This in turn is dependent on the time interval between the end of the charging period and the beginning of the filming period, an extremely variable length of time. The conclusion drawn from this experience was that, because of the budget limitations of the research, the only reliable method of collecting truck travel times with the equipment available was by the traditional stopwatch method.

On the basis of this information, a second visit was made to one of the projects. For this visit, stopwatches were used to record the travel times of the trucks. The 2 men observing this operation stationed themselves at the cut and fill respectively, each equipped with a time-lapse camera, a 2-way radio, and a stopwatch. The cameras were synchronized. Using the radios, one observer recorded the haul time of the trucks and the other recorded the return times of the trucks. The period of observation was approximately 2 hours, during which 27 completed travel cycles were recorded.

TIME-LAPSE DATA RESULTS

This section of the paper discusses some of the results obtained when the collected time-lapse data were analyzed. The results are of interest not only because they are an example of the type of production data that an estimator may need but also because they provide basic data about the loader-truck operation needed for the computer simulation program (as will be seen in the next section).

The loader-truck system, as is the case with many other materials-handling operations, is a cyclic operation. One method of viewing this is based on an analysis of the complete cycle of the truck. That is, the time required for the truck to arrive at the cut, be loaded, travel to the fill, dump, and return to the cut can be taken as the complete cycle time of the operation. This overall cycle time of the truck can be represented by the following equation:

$$C_t = Q_{t1} + L_t + HT + Q_{tf} + D_t + RT$$

where

C_t is the complete cycle time of the operation;
Q_{t1} is the queue time of a truck at the loader;
L_t is the time required to load a truck, composed of a number of loader service times (l_t);
HT is the haul travel time of a truck from the cut to the fill;
Q_{tf} is the queue time of a truck at the fill;
D_t is the time required for a truck to dump its load; and
RT is the return travel time of a truck from the fill to the cut.

The analysis of the data resulting from the time-lapse films can determine the parameters (mean, variance, etc.) as well as the shape of the frequency distributions of each of these elements of the cycle. A discussion of some of the results obtained from the projects follows.

Time to Load Truck

The time to load a truck is actually composed of the number of individual loader service times l_t (where l_t is defined as the time required to place 1 bucketful of material into a truck) needed to fill the truck. The data collected for the loading operation were actually collected in terms of the individual loader service times because they were easier to define on the films. The basic components of the loader service times as determined from the films were

1. Forward to cut face;
2. Load bucket;
3. Back up;
4. Forward to truck;
5. Dump bucket into truck; and
6. Back up.

It was noted during the film analysis that generally 3 or 4 of these load service times were required to completely load each truck. The frequency histogram for the loader service time on Project A is shown in Figure 2. An attempt was made to determine a theoretical model that would "fit" the frequency distribution of loader service times for each project. The hypothesis that either a normal, exponential, gamma, 2-parameter Wiebull, or 2-parameter log-normal distribution could be used was tested and rejected. This did not create a problem, however, because it was decided that the relative cumulative frequency distribution resulting from the data would be used directly in the computer simulation program.

The mean and variance for the loader service time for all of the projects are given in Table 1. It is interesting to note that, although the type and size of loader used and

Figure 2. Histogram of loader service time (l_t) for Project A.

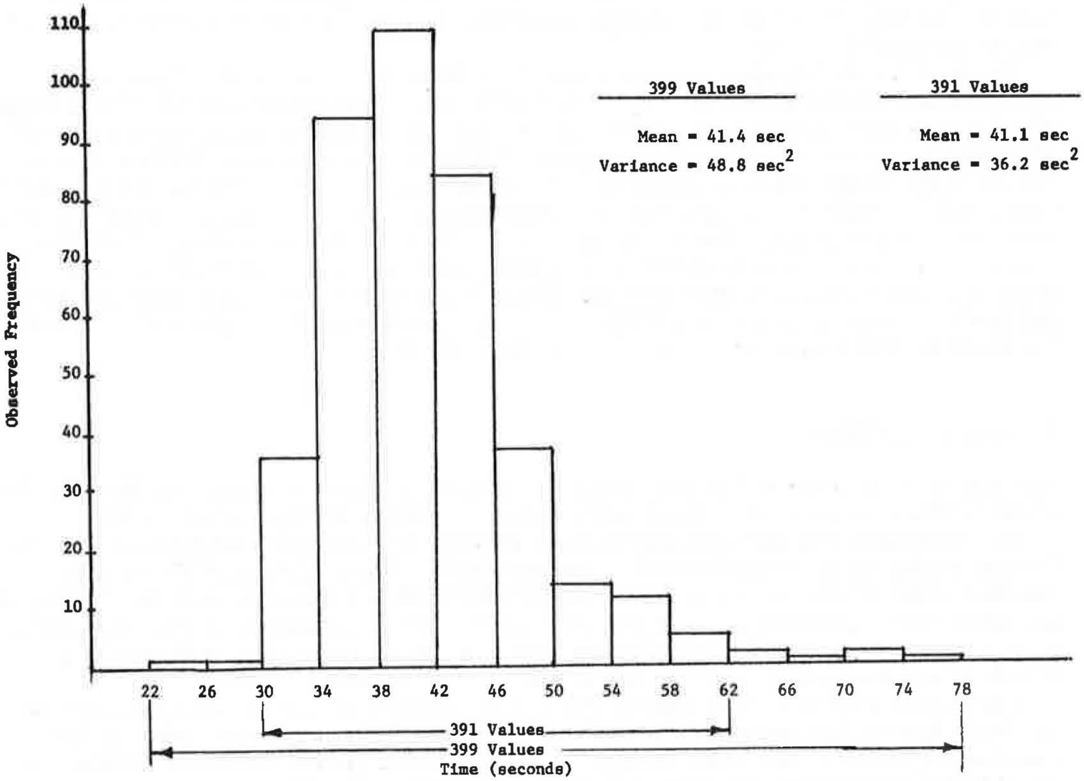


Table 1. Summary of load and dump cycle results for all projects.

	Load Service Time l_t			Dump Cycle Time D_t		
Project	Mean (seconds)	Variance (seconds²)	No. of Observations	Mean (seconds)	Variance (seconds²)	No. of Observations
A	41.1	36.2	391	42.6	129.0	71 (trucks only)
				220.3	1,397.0	71 (bulldozer)
B1	38.3	27.0	165	59.9	108.2	61
B2	42.9	44.9	135	62.7	132.2	43
C1	38.9	32.5	119	79.9	118.8	28 (WABCO)
				55.3	36.0	28 (Euclid)
C2	39.6	50.4	130	137.6	3,203.5	113
C3	42.1	265.7	119	73.7	57.7	31

the type of rock loaded varied from project to project, the average loader service time was generally close to 40 seconds. The order of magnitude of the variance (a measure of dispersion) of the frequency distribution for all projects except Project C3 was essentially the same.

Pruess (7) has indicated that truck spot time (maneuver time of the truck prior to being loaded) should be included because it accounts for approximately 15 to 20 percent of the time to load a truck. According to Pruess, this should average between 20 and 40 seconds. In the course of the collection of data for this research, the loadings of almost 1,100 trucks were observed on 3 different projects. At no time was a 40-second truck spot time observed. In most cases the truck spot time did not exceed 75 percent of the time required for 1 loader service time. As a result of this observation, the truck spot time was included in the first loader service time for each truck. The case where the truck spot time exceeded the initial loader service time and hence required the loader to wait occurred so rarely that it was considered to be sufficiently accurate to consider "truck spot time" as a part of the load cycle.

Time to Dump Truck

The definition of the time to dump a truck is slightly more conditional than the definition of the loader service time. There were 2 distinct cases observed on the films.

The first case was the situation where it was not necessary for the truck to wait for the completion of the dump time of a previous truck. The beginning of the dump time was therefore identified as the point when the truck turned from the haul path and began the maneuver time at the fill. In the case of the rear-dump trucks, it was necessary to back into the position indicated by the foreman. In the case of the bottom-dump truck, it was necessary for the truck to pull into the required position.

The second case occurred only on Project A. On this project it was necessary for the truck to wait for the completion of the dump time of the previous truck. In this case the beginning of the dump service time was defined as that point in time when the truck began moving into position to dump. In addition to the dump time of the truck, the definition of the service time of the bulldozer in the fill was also required on Project A. This is the time from the beginning of the truck dump time until the material deposited by that truck was spread on the grade. It was necessary for a second truck to wait until the bulldozer had finished spreading the material deposited by the first truck. There appeared to be 2 reasons for this truck queue time: First, clearing the area gave the trucks more room to maneuver, and second, on most of the cycles the trucks required the assistance of the bulldozer to extricate themselves from the deposited material.

There were also 2 distinct cases that occurred at the end of the dump time. When rear-dump trucks were used to transport the material (on Projects B, C1, and C2), the end of the dump time was defined as the time when the box resumed its "rest" or travel position. On Project A, where bottom-dump trucks were used, the end of the dump service time was defined as the time when the truck completed its turn to reenter the return road.

The frequency histograms for the dump time on Project A for both the trucks and the bulldozer are shown in Figures 3 and 4 respectively. As noted for all the other projects, only a frequency histogram for trucks was determined. The statistical analysis of the dump time for the projects indicated (as had been the case for loader service times) that the standard mathematical models did not "fit" the data. As a result, a relative cumulative frequency polygon was again calculated for each dump time on each project and was used directly in the computer simulation program.

Table 1 gives the mean and variance for the dump times for all of the projects. The first thing to be noted about these results is that the variance of the data is very large in almost all cases. This indicates that, although the loader service time was fairly well-controlled (i.e., the variances were smaller), the same statement cannot be made for the dump times. This indicates the need to study seriously the dumping operation on a loader-truck operation because there appears to be a real opportunity to reduce

Figure 3. Histogram of dump time of trucks at the fill (D_t) for Project A.

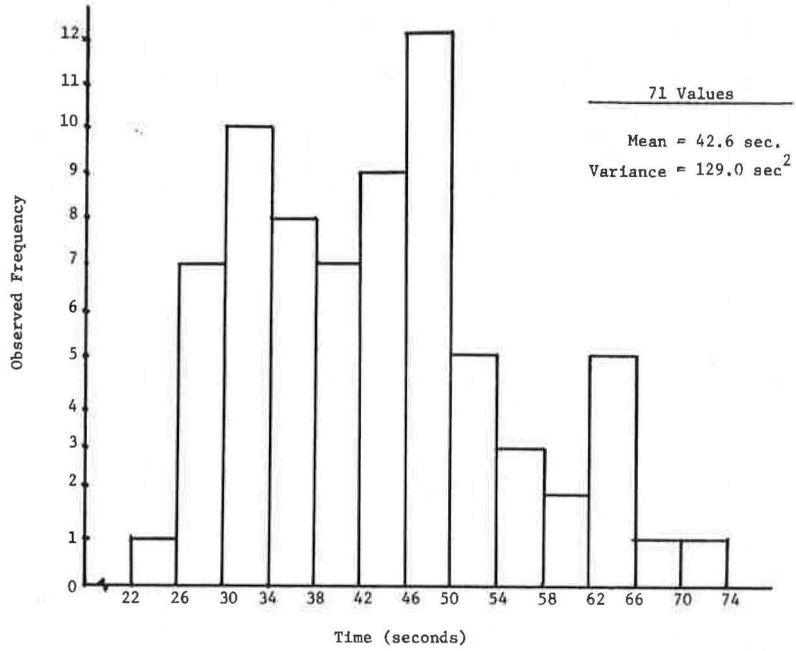
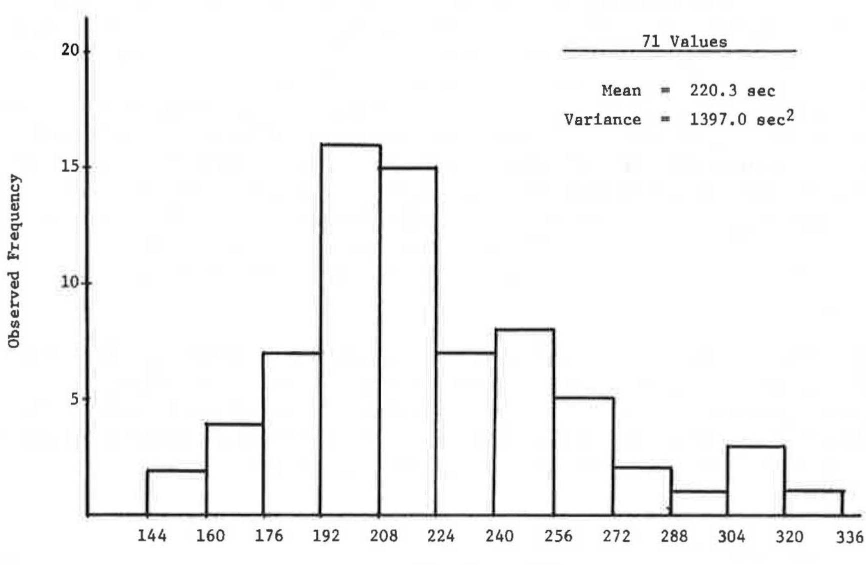


Figure 4. Histogram of dump time (governed by dozer at fill) for Project A.



the truck cycle time by modifying the practices at that location.

It is interesting to note that on Project A, which had a bulldozer affecting the operation, the dump time is much higher than on other projects. It is also interesting to note that on Project C1, when the dump times for the WABCO and Euclid trucks are studied separately, it appears that the former truck requires more time to dump and has a wider dispersion of dump times.

The final point to be made concerns the dump time for Project C2, which involved the backfilling of the box culvert. The greater length of time required and the relative confusion of the operation are clearly indicated by the fact that the average dump time is 137.6 seconds and the variance is 3,203.5 seconds^2.

Other Cycle Time Components

Other items of the truck cycle time were analyzed in the full research report (8) that are not discussed in detail in this paper. An attempt was made to identify the following items from the films (or related stopwatch readings):

1. Truck delays, which were defined as events that caused a truck to spend a greater amount of time on the haul or return road than the minimum observed haul or return time. In most cases a truck delay occurred out of the view of the camera. As a result, the delay times were calculated by subtracting the minimum observed haul or return time from each of the observed haul and return times.
2. Loader-caused delays, which might also be called loader breakdowns. In many cases these delays were a result of mechanical failures of the loader. In other cases, the delays were caused by outside interference—for example, a short interruption due to the drilling and shooting operation. The case of a loader cleaning up the work area was also classified as a loader-caused delay if a truck was waiting to be serviced during the cleanup work. This cleanup work usually involved removing loose rock fragments from, and maintaining a level surface in, the maneuvering area. The time between loader-caused delays was also accumulated from the time-lapse films because this information was needed in the computer simulation program.
3. Equipment wait times, which were of two types, the time the trucks spent waiting to be serviced at either the cut or the fill (truck queue time) and the time the loader spent waiting for trucks to arrive (loader idle time). The loader idle time included the time the loader spent cleaning up the cut floor if there were no trucks waiting for service. It should be noted that the only occurrence of truck queue time at the fill was on Project A. This was caused by the type of trucks being used and the nature of the dumping operation. There were no truck queue times at the fill observed on any of the other projects.

The time-lapse data results discussed in this section should indicate that the level of information that an estimator has about the loader-truck operation by using this method is clearly superior to the estimating rules of thumb he is probably using. The next section of this paper shows the use for these time component results in a computer simulation program that attempts to describe the loader-truck operation.

THE COMPUTER SIMULATION

The experiences gained by the field observations of the loader-truck operation permitted the boundaries of a typical loader-truck operation and the elements that interact within it to be defined. At this point in a production study, one of a number of computer simulation languages can be used to prepare a stochastic model of the system. This section of the paper describes the type of simulation model that may be developed in SIMSCRIPT II.5 in order to define the operation. Additional details are given elsewhere (8).

Background

The SIMSCRIPT II.5 language used in the simulation program was developed initially at the Rand Corporation in the early 1960s to advance the "simulation art" as well as to facilitate the writing of Air Force Logistics Simulators. The language requires that the world to be simulated be structured in terms of the concepts mentioned by Markowitz (9) in the following description:

> The SIMSCRIPT programming system is especially designed to facilitate the writing of simulation programs. Digital simulations generally consist of a numerical description of "status" which is modified at various points in simulated time called "events." SIMSCRIPT simulations consist primarily of a collection of "event routines" written by the user describing how different kinds of events in a particular simulated world affect current status and cause future events. Status is defined in terms of various "entities," "attributes," and "sets" as specified by the user.

Computer Program Use of Time-Lapse Data

The computer program may be explained by discussing the event interactions that model the loader-truck operation. It should be noted that to use the program on a particular project certain information must be obtained from the time-lapse films taken for a similar equipment fleet used on a previous project. The data required for the loader-truck situation are given in Table 2.

It should be noted that information about truck travel times from time-lapse films of previous projects is not needed because the program has been designed so that a subprogram in FORTRAN calculates these times in a deterministic fashion. It uses information about the rolling resistance, road profile data, etc., which must be provided for the project being studied. These travel times are then modified by applying stochastic truck delay times to the values determined in the subprogram.

Event Interactions

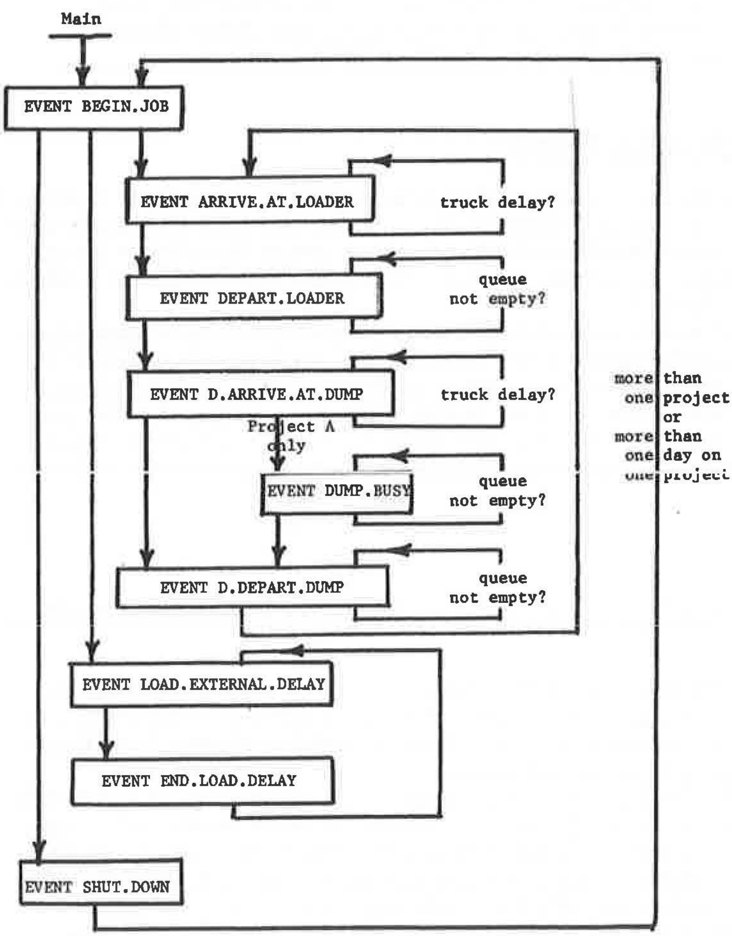
Figure 5 shows the relationship between the 9 events that define the operation in the simulation program:

1. The event BEGIN.JOB initializes the various entities, attributes, and arrays that are required to execute the simulation based on data put into the program by the user. A loader breakdown (based on a frequency distribution determined from time-lapse data) may also be scheduled from this event.
2. The event ARRIVE.AT.LOADER makes a check to see if there was a truck delay (again generated from a cumulative frequency distribution of truck delays determined from the time-lapse films). If there is a truck delay, that truck is rescheduled to arrive at the loader. If the loader is busy, the truck is filed in the set QUEUE. If the loader is not busy, the routine R.L.SERVE schedules a DEPART.LOADER and calls the routine QUANTITY, which computes the quantity of material to be transported. [Note: The routine R.L.SERVE is used to calculate the amount of time required to load a truck, which is dependent on the service time of the loader and the number of cycles required to fill a truck. The load service time is randomly chosen from a cumulative frequency distribution of load service times determined from time-lapse film data. The routine QUANTITY computes the amount of material to be carried by a given truck on a given trip. This is done by choosing a random number from a normal distribution with a mean equal to the capacity of the loader in loose cubic yards and a standard deviation of 15 percent of the mean as suggested by Gates (10). The number of times this random number is generated depends on the number of loader cycles required to fill the truck. The quantity of material transported is accumulated as the sum of these random variables for each truck.]

Table 2. Summary of the time-dependent components used in the simulation.

Component	Type of Distribution	Determination of Parameters
Load service time	Cumulative frequency polygon	Time-lapse films
Truck dump time	Cumulative frequency polygon	Time-lapse films
Bulldozer dump service time	Cumulative frequency polygon	Time-lapse films
Truck delay time	Cumulative frequency polygon	Time-lapse films
Loader delay time	Exponential	Time-lapse films
Time between loader delays	Exponential	Time-lapse films
Truck travel time		Calculated in simulation

Figure 5. Event diagram of the loader-truck model.



3. The event DEPART.LOADER checks for an empty QUEUE. If the QUEUE is not empty, the first truck in the set QUEUE is removed from the set. To simulate the loading of this truck, another DEPART.LOADER is scheduled, and the quantity of material to be transported is computed. The routine R.FILE is called to file the departing truck in the HAUL.SET. The routine H.TIME.HAUL in turn calls the routine TRAVEL.TIME to compute the arrival time of the truck at the fill. A D.ARRIVE.AT.DUMP is then scheduled. [Note: The routine TRAVEL.TIME is used to calculate the haul or return times of the trucks. This calculation is based on the rolling resistance, grade, and length of the haul (or return) road and the weight (which as stated is a random variable), rimpull, and retarder characteristics of the trucks. The haul or return road may be divided into any number of sections. Different haul and return routes may be specified.]

4. Event D.ARRIVE.AT.DUMP checks for a truck delay occurring along the haul road. If a truck delay does occur, that truck is rescheduled to arrive at the fill. If the dump site is busy, the event D.ARRIVE.AT.DUMP files the truck into the set D.QUEUE. If the dump site is not busy, the routine R.D.SERVE is called and a D.DEPART.DUMP is scheduled (to simulate a truck unloading at the dump site). If the set D.QUEUE is not empty, the event D.DEPART.DUMP calls the routine R.D.SERVE to schedule another departure from the dump site, for the first truck in D.QUEUE. The routine R.TIME.RETURN is then called, which in turn calls the routine TRAVEL.TIME to calculate the time required for the truck to return to the loader. The truck is then filed into the set RETURN.SET and an ARRIVE.AT.LOADER is then scheduled. [Note: The routine R.D.SERVE calculates the service times of the trucks at the fill from a cumulative frequency polygon determined from time-lapse data. If a DUMP.BUSY is to be scheduled, the service time of the bulldozer is computed from a cumulative frequency polygon also determined from time-lapse data.]

5. If the event DUMP.BUSY is used, the status of the dump site is set to not busy. If the set D.QUEUE is not empty, R.D.SERVE is called, and a D.DEPART.DUMP and another DUMP.BUSY are scheduled.

6. When event LOAD.EXTERNAL.DELAY occurs, the status of the loader is set to busy, and an END.LOAD.DELAY is scheduled. The length of the loader delay is a random number drawn from an exponential distribution. The mean of this distribution is supplied to the simulation as a real number in the variable MLDT, which is based on time-lapse data. If a truck is being served, R.FILE is called and a D.ARRIVE.AT.DUMP is scheduled.

7. The event END.LOAD.DELAY schedules the next LOAD.EXTERNAL.DELAY and removes the first truck from the loader queue. The time between loader delays is a random number drawn from an exponential distribution. The mean of this distribution is supplied to the simulation as a real number in the variable MLDI, and is based on time-lapse data. Routine R.L.SERVE is then called, which in turn calls routine QUANTITY and schedules a DEPART.LOADER.

8. The event SHUT.DOWN cancels any pending DEPART.LOADER events. The queue times of any trucks currently waiting to be serviced are accumulated. The status of the loader is set equal to 2, which indicates that, when all the trucks have returned to the loader, the routine CLEAN.UP is to be called. The routine CLEAN.UP calls the routine DAILY.REPORT and all the currently scheduled events are canceled. If another project is to be simulated, all the entities and their attributes are destroyed; if the same project is to be simulated for another day, the entities are saved. If the required number of days has been simulated or if the required quantity of material has been moved, the routine END.SIMULATION is called. If necessary, a BEGIN.JOB is scheduled for the start of the next day. The routine DAILY.REPORT prints out the accumulated daily statistics for the project being simulated.

Validation of Simulation Program

A very important step in the development of any simulation program is the validation stage, which attempts to determine if the predictions made by the program correctly model reality. The procedures used to validate the program are only briefly discussed here because of space limitations. The interested reader is referred to the full report (8) for a more detailed explanation.

There are a number of parameters determined by the simulation program that may be statistically compared in the validation phase to the results determined from the time-lapse films. The first of these is the travel time of the trucks. The travel time results of the program can be compared with stopwatch readings taken at the job site while the filming was taking place. (The results from the films could be used directly if the problem of synchronization mentioned earlier could be corrected.) If the two results compare favorably, the program may be considered partially validated. Other parameters that may be treated in a similar fashion are the following:

1. The predicted versus the actual hourly production rates.
2. The frequency distribution of the predicted versus the actual truck queue times at the loader.
3. The frequency distribution of the predicted versus the actual truck queue times at the fill.
4. The frequency distribution of the predicted versus the actual loader wait times.

If it appears that these program parameters do not satisfactorily model reality, then basic changes to the assumptions in the program must be made. Once a program is successfully validated, however, the authors feel that an estimator may use the combination of time-lapse photography and the computer simulation program as valuable aids in his day-to-day activities.

SUMMARY AND CONCLUSIONS

This paper has described a procedure that combines the techniques of time-lapse photography and computer simulation to define the loader-truck operation in order to provide a construction estimator and planner with a better basis for decision-making. The method of collecting data by using time-lapse photography is discussed, and the results obtained on 3 Pennsylvania highway construction projects are presented. A SIMSCRIPT computer program that models the loader-truck operations observed is discussed.

The actual SIMSCRIPT computer program is only partially documented in the report of the project (8). A lack of research funds prevented the development of a "user's manual" for the program as well as a more extensive validation of the program. Such a validation study would typically be required in order to properly "debug" the program. It is hoped that future research support can be acquired that will allow this study to be continued. It is the authors' opinion that a continuation of this study would eventually encourage the use of the combination of time-lapse photography and computer simulation in actual practice.

REFERENCES

1. A. Gaarslev. Stochastic Models to Estimate Production of Materials Handling Systems in the Construction Industry. Technical Report No. 111, Department of Civil Engineering, Stanford University, Aug. 1969.
2. J. Douglas. Prediction of Shovel-Truck Production: A Reconciliation of Computer and Conventional Estimates. Technical Report No. 39, Department of Civil Engineering, Stanford University, Aug. 1963.
3. P. Teicholz. An Analysis of Two-Link Material Handling Systems With One Carrier in One of the Links. Technical Report No. 29, Department of Civil Engineering, Stanford University, Aug. 1963.
4. F. A. Griffis. Optimizing Haul Fleet Size Using Queuing Theory. Journal of the Construction Division, ASCE, Jan. 1968.

5. T. J. O'Neil. Computer Simulation of Materials Handling in Open Pit Mining. Thesis in Mining Engineering, Pennsylvania State University, 1966.
6. H. W. Parker and C. H. Oglesby. Methods Improvement for Construction Managers. McGraw-Hill Book Co., New York, 1972.
7. D. Pruess. Production and Cost Estimating for Earthwork Construction. Presented at a Continuing Education Seminar, Pennsylvania State University, March 1, 1973.
8. J. Willenbrock and T. Lee. Analysis of Earthmoving Operations Using Time-Lapse Photography and Computer Simulation. Construction Management Research Series Report No. 6, Department of Civil Engineering, Pennsylvania State University, Aug. 1974.
9. H. Markowitz. Simulating With SIMSCRIPT. Management Science, Vol. 12, No. 10, June 1966.
10. M. Gates. Bidding Contingencies and Probabilities. Journal of the Construction Division, ASCE, Nov. 1971.

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