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Texturing New Concrete Pavements

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Several texturing experiments on heavily traveled portland cement concrete pavements in Virginia are described. Included in the experiments were textures imparted by using a heavy burlap drag, metal tines (transverse and longitudinal striations), sprinkled aggregate, mortar removal, and imprinting. All textures were imparted to concrete in the plastic state. Some of the problems encountered in achieving the desired textures are discussed. Evaluations of the effectiveness and the acceptability of the textures included noise, roughness, and skid-resistance studies. These studies resulted in the rejection for future use of several textures for one or more reasons. A consideration of all factors gave strong indications that transversely tined grooves spaced 19 mm (0.75 in) apart (center-to-center) would be preferable on tangent roadway sections whereas longitudinally tined grooves spaced 19 mm apart in combination with transverse grooves spaced 76 mm (3 in) apart would be preferable on curves.

Until the increase in traffic volumes and speed limits that accompanied the building of the Interstate highway system, Virginia had experienced no difficulty with skid resistance on portland cement concrete (PCC) pavements. For the most part, this type of pavement was built in the eastern portion of the state where polish-resistant siliceous aggregates and sands abound. The stopping distance number (SDN) at a test speed of 64 km/h (40 mph), i.e., 40 (SDN_{40}), which the state has attempted to provide on its primary system, was easily maintained for the life of the pavement surface. In fact, there is no recollection of problems with wet-pavement accidents on PCC pavements nor of SDN_{40} values as low as 45 at pre-Interstate traffic volumes and speed limits. It should be remembered that prior to the building of the Interstate system, the maximum daily traffic volume on a dual-divided PCC pavement in the state was about 25 000 vehicles, and the speed limit was 96 km/h (60 mph). With the coming of the Interstate system, the figures for average daily traffic (ADT) were increased to about 90 000 vehicles/d and 113 km/h (70 mph); it is thus understandable that some relatively low SDN_{40} values of around 40 were found and that there was a relatively high percentage of wet-pavement accidents on some PCC Interstate highways (1). The relatively low skid numbers and pavement surfaces that had become rather smooth combined with bald tires and thick films of water to create a potential for hydroplaning.

In an attempt to remedy the problems that accompany increased traffic volumes and speeds, a project was undertaken to devise means of providing durable surface finishes for new concrete pavements that would provide both good skid resistance and enough texture to prevent the buildup of thick water films. Because highly polish-resistant materials were already being used in concrete pavements in Virginia, the additional skid resistance needed as well as the surface drainage to prevent hydroplaning obviously had to come from a harsh and lasting surface texture.

Little information on harshly textured concrete surfaces was found in the literature, but it was learned that other states were recognizing similar problems. California had learned about the grooving of airport runways and had done some experimental grooving at sites where there was a high rate of wet-pavement accidents. On the basis of the apparent success of the California effort, Virginia had considered grooving the pavement at several such sites.

Ohio had been experimenting with finishing methods that imparted a harsh texture to new concrete surfaces. Arrangements were made for a visit with the materials engineers in Ohio to inspect and discuss with them the test section they had installed. It was found that the Ohio Department of Transportation had asked the contractors to explore methods and equipment for providing a deep texture on the surface of new concrete pavements. The contractors had responded by making three or four passes using four thicknesses of burlap; one pass with rug backing; roping or mops attached to the burlap drag; longitudinal grooving with a coarse, plastic bristled broom; and, in some cases where the local aggregate had a low silica content, "seeding" with skid-resistant aggregate on the plastic concrete.

Although it would have been desirable to do so, in 1969 it was not possible to try all of the Ohio experimental finishes in Virginia because the one scheduled concrete paving project in the state for that year had already been awarded. Thus, only experimental burlap textures could be scheduled immediately. The other finishes had to be delayed so that descriptions of the desired surface textures could be included in advertisements. This paper discusses experimental texturing activities in Virginia, beginning with the experiments with burlap.

TEXTURING EXPERIMENTS

Use of Burlap

Texturing experiments using burlap were conducted in 1969 and 1970 on some 48 km (30 miles) of I-64 around Charlottesville, Virginia. The roadway is a continuously reinforced concrete pavement (CRCP) 7.3 m (24 ft) wide by 203 mm (8 in) thick.

Because the paving contracts were let at approximately the same time that the need for harsher textures was realized, plans to provide special texturing on the pavement had to be made hurriedly. Furthermore, because the contracts had been bid under 1966 specifications that prohibited striations more than 1.6 mm (0.062 in) deep (2), any action that would result in a harsher texture on the pavement would clearly have to be negotiated with the two paving contractors. The contractors agreed to make a reasonable effort to texture the pavements as desired. It was decided that longitudinal striations would be used because transverse striations might create undue tire-pavement noise. A sample texture block was prepared for the guidance of the contractors.

Procedures and Materials

Paving was begun in late 1969. Project personnel had the sample texture block for comparison purposes, and state personnel were on hand to observe the operations. The concrete met the state's specifications for class A3 paving concrete (2).

In the paving operations, a slip-form paver placed and screeded the full 7.3-m (24-ft) pavement width in one pass and a tube float applied the initial finish. The float unit was equipped with a hydraulic mechanism that carried the burlap drag used to apply the final finish. Following the float was a curing unit that applied either polyethylene sheeting or a liquid membrane. All units in the paving train were remotely controlled transversely by a guideline placed on the edge of the roadway.

Several attempts were necessary before a texture of the desired harshness was achieved by using the burlap drag. Success was achieved with one to four passes depending on the consistency of the concrete, the rate of surface drying, and the number of layers of burlap used.

Figure 1. Obliteration of burlapped texture by polyethylene sheeting.

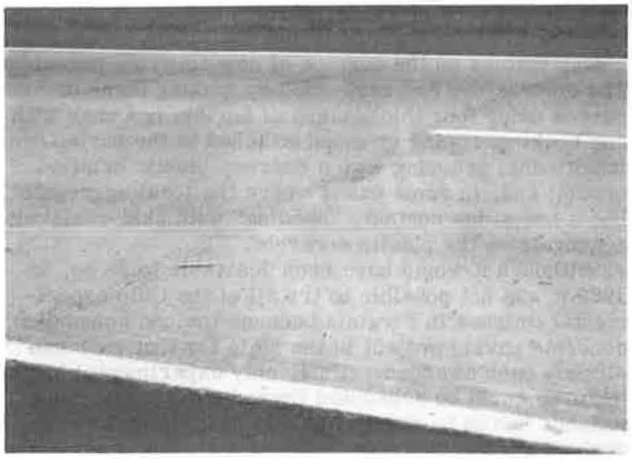
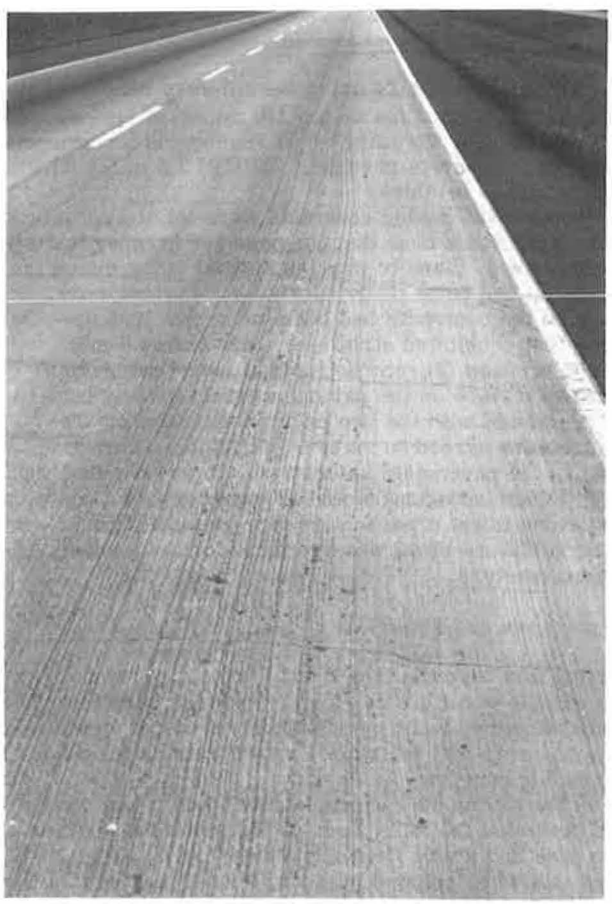


Figure 2. Texture achieved by using heavy burlap.



Project personnel were required to exercise a good deal of judgment in determining the number of passes to apply under given circumstances. On long sections of pavement, drags consisting of two layers, two or three layers, and four layers of burlap were tried. A four-layer drag with the necessary number of passes was finally determined to be the most effective procedure.

Other factors that affected the texturing procedure were the length of the trailing burlap cords and the condition of the burlap. Trailing cords 100 to 150 mm (4 to 6 in) in length were helpful, and burlap that had accumulated some mortar through use was found to be effective. Again, the judgment of project personnel was relied on for the proper number of passes to be applied.

The major technical problems encountered with heavy burlap texturing were a failure to achieve the desired texture because of late application of the burlap drag and, under certain circumstances, obliteration of the texture by the polyethylene sheeting used for curing purposes (Figure 1). The first of these problems was easily eliminated by paying close attention to the time of texturing to ensure that the burlap drag was applied well before the pavement surface had achieved any significant degree of set. In fact, the concrete used in the slip-form paving operation was very uniform and necessarily of such a consistency that texturing was possible immediately after floating. The second problem was solved by using a liquid-membrane curing compound in place of the polyethylene sheeting. The curing compound was applied approximately at the time the sheen disappeared from the concrete surface so that the texture was not affected by the curing operation. Subsequently, polyethylene sheeting was used only in special cases, such as when it became necessary to protect the surface from heavy rains or during extremely cold weather. In the event of cold weather, the sheeting was not applied until there was no danger of damage to the texture.

Results

Figure 2 is a photograph of the most desirable texture achieved by using the special burlap drag. For comparison purposes, Figure 3 shows a typical texture caused by the use of light burlap, which could be found on many Virginia pavements before this study was begun. The new texture is much more evident to the naked eye. As Figure 2 shows, the striations are randomly spaced according to the weave of the burlap. Both contractors were of the opinion that the heavy-burlap texturing could be achieved in normal paving operations; it was thus decided to use a similar texture for the remaining pavement in the contracts for I-64 in the Charlottesville area.

The burlap-textured pavement was opened to traffic in September 1970. Road roughness tests conducted in the fall of 1970 showed that two of the three pavement projects resulted in the best riding concrete pavements ever tested in the state. The results of these tests are given below and include Bureau of Public Roads (BPR) roughness values for each of the three textured projects (1 km = 0.62 mile):

Project Number	Length (km)	Average BPR Roughness (units/km)
1	16.4	40
2	14.8	46
3	13.7	57

The BPR roughness of 25 projects constructed between 1965 and 1969 ranges from 54 to 88 units/km; the average roughness of these projects is 61 units/km.

The roughness value for even the roughest of the textured projects is toward the low side of average roughness values for pavements built during the previous 5-year period. A more detailed evaluation of roughness, skid resistance, and noise is given later in this paper.

Some initial public observations of poor riding quality were felt to be a psychological reaction attributable to the appearance of the pavement texture rather than to actual roughness. There were also a few early adverse public reactions concerning the occasional waviness of the striations, which was caused by side sway in the hydraulic arms carrying the burlap and by the flexibility of the burlap material.

Although the burlapped texture seemed very satisfactory at the time the pavement was opened to traffic, after it had been in service about 8 months under an ADT of 8000 vehicles/d, there was a significant degree of wear in the wheel paths that was easily discernible from a moving automobile and that seemed to be reasonably uniform throughout the pavement sections. It was noted that the loss of texture was inversely related to the initial harshness of texture. Thus, it was concluded that pavements having a heavier initial texture would retain their texture longer. This tentative conclusion seems to have been borne out by the pavement's performance over the 5 years since the earlier observations and by laboratory studies reported by Ozyildirim (3). Figure 4 shows the wear in the wheel paths after 6 years under traffic.

Figure 3. Light-burlapped texture used before texturing experiments.

Figure 4. Heavy-burlapped texture after 6 years under traffic.

The relatively rapid wear of the texture caused concern, and the consensus was that the limited area between the asperities in the texture tended to make the asperities weak and subject to damage from the abrasive action of vehicle tires, including a small percentage of studded tires. An indicated reduction in the rate of wear with time was believed to result when the sharp points in the texture abraded, leaving the broader and stronger remains of the asperities. This finding was also supported by Ozyildirim (3). In addition, it was felt that the burlap finish did not provide sufficient channels for water drainage. These observations and tentative conclusions led to experiments with lands-and-grooves types of textures.

Use of Metal Tines

At about the time the I-64 pavement at Charlottesville was opened to traffic, contract documents were prepared for two projects totaling 35.2 km (21.9 miles) of rural I-64 east of Richmond. This roadway, which is the main route between western and central Virginia and the coastal area, was also designed as a divided 7.3-m (24-ft) wide by 203-mm (8-in) thick CRCP.

So that more texture could be obtained in the pavement surface than the standard specifications demanded or the heavy burlap texture provided on the Charlottesville projects, the following special provision was made a part of the contract documents:

> As soon as construction operations permit, and before the water sheen has disappeared, the surface of the pavement shall be dragged longitudinally (in the direction of the concrete placement) with a coarse bristled broom or series of such brooms. The drag shall be passed over the fresh concrete one or more times as required to produce a surface texture having characteristics equivalent to the texture which has been produced on sample blocks available for inspection. The ridges and grooves of the texture shall be reasonably straight and parallel with respect to the centerline of the pavement.

Procedures

The sample blocks were prepared in the laboratory by using a standard push broom to produce the desired pattern. However, because of the early wear of the experimental finishes on I-64 at Charlottesville, the durability of a broomed concrete surface was questioned and consideration was given to using grooved surfaces. Pavements that had been grooved at accident-prone locations had resulted in improved safety (4), and the grooved surfaces had proved to be quite durable. For these reasons, it was decided that the surfaces for the two projects being awarded in the Richmond area would more closely approximate the grooved rather than the broomed finishing.

The contractor was approached about the feasibility of changing from the planned broomed finish to longitudinal striations similar to grooves. The contractor thus began the project by using a wire comb consisting of metal tines 3.2 mm (0.125 in) wide and spaced 3.2 mm apart, center-to-center (throughout this paper, groove- and tine-spacing measurements indicate center-to-center spacing). The tines were about 100 mm (4 in) long. This arrangement did not cover a satisfactory land area and tended to displace an excessive amount of mortar. After only a short distance had been constructed, the contractor was asked to change to 3.2-mm-wide tines spaced 9.5 mm (0.375 in) apart. These tines were about 180 mm (7 in) long and set at about a

30° angle to the pavement. The tines were secured in wooden heads similar to the common rectangular push-broom head. They were dragged through the fresh concrete with approximately 25 mm (1 in) of the tine parallel to the pavement surface. According to Vann, a resident engineer at the time of construction, as the tine width wore to about 1.6 mm (0.062 in), it became necessary to clip the ends to maintain the original width.

From the construction standpoint, the second arrangement was much more satisfactory, but it was still felt that insufficient land area was being covered. After limited operation, the contractor was requested to bend up every other tine; this provided groove spacings of approximately 19 mm (0.75 in). This pattern was satisfactory and was used on the remainder of the project. Once the desired pattern was established, no difficulties were encountered in the texturing operation.

The operation was the same as that described for the project on I-64 at Charlottesville; that is, the slip-form paver was followed by a magnesium float, a burlap drag, the texturing apparatus, and a liquid-membrane seal. Of course, more curing liquid is required for the tined surface than for the burlapped surface. The concrete met the same specifications as those described for the burlapped project. Even if the consistency of the concrete mixture is well controlled, the timing of the texturing operation is crucial. The desired depth was 3.2 mm (0.125 in), and the operator of the texturing machine quickly learned the proper time to start that operation.

Results

When the state of Virginia first started grooving hardened pavements, complaints were received from motorcyclists and operators of compact automobiles to the effect that the grooving tended to override their steering. A review of the findings of a California study on motorcycle reactions to grooved pavement (5) dispelled much of this concern but, as a concession to drivers, GROOVED PAVEMENT AHEAD signs were placed before the grooved sections.

Because the Richmond projects comprised a much longer stretch of grooved pavement than had been placed before, a public relations effort was undertaken to publicize what was being done, emphasizing the safety aspects of the new type of finish. The results have been very satisfactory. A minimal number of complaints have been received and, as noted previously, they have been largely psychologically based.

The project was opened to traffic in early December 1972. After 4 years under an ADT of 14 000 vehicles/d, the texture shows little sign of wear.

A possible disadvantage of the finish is the tendency of deicing chemicals to remain longer in the grooves than they would on pavement with a smooth surface or some transverse texture and thus to cause the concrete to deteriorate at a faster than normal rate. There are no quantitative data to support such a conjecture and, in fact, Virginia concrete technologists feel that, if the concrete has a low water-cement ratio and the proper air entrainment and is properly cured, deterioration from salt action should cause little concern. On the other hand, the retention of the chemicals on the pavement may provide an ice-free pavement for an extended period.

One factor that could affect the durability of the texture is the use of studded tires; this area of Virginia, however, has such a low incidence of studded tires that the effect is difficult to assess. Obviously, where high percentages of studs are used, this texture or any other would not remain very long.

As a result of the experience with the two projects, it was decided to continue finishing concrete pavements with tined grooves until other patterns could be studied. The following special provision was immediately put into effect and then included in the 1974 Virginia Road and Bridge Specifications (9):

FINAL FINISH (TEXTURE): The contractor is advised that the surface of the pavement shall have more pronounced ridges and grooves than can be obtained by the normal methods of texturing with burlap or stiff bristle brooms. Prior to the beginning of paving operations, the Contractor shall prepare and submit for approval a sample texture block having a minimum size of 12" x 12" (305 mm x 305 mm), utilizing the texturing device he plans to use on the project. A surface texture having characteristics equivalent to the texture on the approved sample block shall be produced on the concrete pavement. The ridges and grooves of the texture shall be reasonably straight and parallel to the centerline of the pavement.

However, it was also decided that other texturing schemes should be tried, and planning was begun for test sites to be included in the next PCC pavement contract to be awarded.

Test Sites on International Terminal Boulevard

The advertising schedule for the next PCC pavement contract allowed for the planning of comprehensive experiments. As many texturing schemes as showed promise for providing skid resistance and removing water from the tire-pavement contact area were to be included, and consideration would be given to tire-pavement noise, the practicality of applying the finishes, and costs.

The textures finally decided on were as follows (1 mm = 0.039 in):

Site	Texture
1	Longitudinal striations spaced 19 mm apart
2	Transverse striations spaced 76 mm apart
3	Exposed aggregate
4	Sprinkled aggregate (large)
5	Sprinkled aggregate (small)
6	Dimpled (imprinted)
7	Transverse striations spaced 19 mm apart
8	Combination of longitudinal and transverse striations spaced 19 and 76 mm apart respectively
9	Transverse striations spaced 38 mm apart
10	Combination of longitudinal and transverse striations spaced 19 and 38 mm apart respectively

The experimental sections are located in the extreme southeastern part of the state where there are relatively few freeze-thaw cycles and the pavements are subjected to small quantities of deicing chemicals during the winter months. Practically no studded tires are used in the area.

The project involved the construction of dual divided lanes 7.3 or 7.9 m (24 or 26 ft) wide on International Terminal Boulevard between I-564 and the International Terminal in Norfolk. Estimated ADT in 1969 was 11 160 vehicles/d, of which 12 percent were tractor-trailers and buses. Projected 1992 ADT is 21 400 vehicles/d (12 percent tractor-trailers and buses). The design speed is 72 km/h (45 mph).

Procedures

The project was advertised in June 1972, and the contract was awarded in August. Because the contract was awarded to the same firm that had constructed the 35.2 km (21.9 miles) of rural Interstate east of Richmond, the equipment used was the same as that used to impart the longitudinal striations. This equipment was modified to impart the transverse and dimpled textures as well. A wire comb similar to that used on the pavement in Richmond, with properly spaced tines, was used for the longitudinal and transverse striations, and a steel drum approximately 305 mm (12 in) in diameter and 1.82 m (6 ft) long, with properly spaced chloroprene blocks epoxied to it, was rolled transversely to produce the dimpled texture.

The exposed-aggregate surface was produced by spraying the finished concrete with approximately 31.5 dm^3/m^2 (7 gal/yd^2) of a commercial retarder and water mixed at a ratio of 5:3 by volume. The retarding mixture was allowed to stand overnight, and the mortar was washed from the surface on the day following placement.

The sprinkled surface was obtained by hand distributing the aggregate over the surface of the plastic concrete at an approximate rate of 2.7 kg/m^2 (5 lb/yd^2). These sections were 45.7 m (150 ft) long; the sprinkle material used was 19-mm (0.75-in) aggregate on one section and 13-mm (0.5-in) aggregate on the other. The aggregate was precoated by using a cement-and-water paste and was rolled into the finished surface of the plastic concrete by a roller 1.8 m (6 ft) long, 205 mm (12 in) in diameter, and approximately 113 kg (250 lb) in weight.

Results

Figures 5 through 14 show the textures obtained on the experimental test sections. The texture shown in Figure 5 was the standard texture at the time the International Terminal Boulevard test sections were installed. Note also that the sprinkled textures shown in Figures 8 and 9 are the harshest ones produced.

The contrast during rainfall between a section with both transverse and longitudinal striations and one with only a longitudinally tined texture is of special interest. Figure 15, a photograph taken during a steady rain, shows a texture longitudinally and transversely grooved with 19-mm (0.75-in) spacing between grooves (in the foreground) and the conventional, longitudinally tined texture produced by 19-mm groove spacing (in the background). Note that the transverse texture is providing excellent drainage; compared to the pavement in the background, it appears to be dry.

NOISE AND ROUGHNESS STUDIES

In an effort to determine potential noise- or roughness-related problems with the various types of texture, noise and roughness studies were conducted on each of them. These studies were felt to be essential to the final selection of a texture type for routine paving operations.

Noise Studies

Measurements

Noise measurements for both the roadside and the interior of an automobile were conducted on the experimentally textured sections and on several asphalt concrete surfaces that were used for comparative purposes. The details of these studies, in which a change in noise level (dBA) of 2.5 units was considered significant, have been given in two reports by Noble (6, 7).

The roadside tests were normalized at 77 km/h (48 mph). The sound-sensing device was located 7.6 m (25 ft) from the center of the traveled lane. Although both ribbed and snow tires were used, the automobile-interior noise measurements were conducted using ribbed tires only, at a speed of 89 km/h (55 mph), the microphone being positioned at approximately the ear level of front-seat passengers. In both series of tests, tire pressures were maintained at 200 to 210 kPa (28 to 30 lbf/in^2).

The results of both series of tests, a summary of which is given in Table 1, are generally inconclusive. Although there are some significant differences in the noise levels generated, they are usually related to types of tires or to pavement wear. The noise levels for exposed-aggregate, sprinkled, and dimpled textures tend to be somewhat greater than those for either the tined textures or the bituminous concrete pavements, which generate noises of similar intensities. None of the tined textures was significantly different in noise level except the worn, longitudinally tined texture with 19-mm (0.75-in) groove spacing on I-64, which rode slightly quieter.

These inconclusive findings led to the conducting of frequency analyses, from which it was concluded, in part, that some of the transversely tined textures and the dimpled texture generated noise of a relatively pure tone in a frequency range (1000 Hz and greater) in which the noise is more noticeable than noise of equal intensity but of lower frequency (6, 7). The implication of the finding was that some textures may generate noises that, though they are of relatively low intensity, are objectionable to the human ear. For this reason, a subjective evaluation was conducted.

Subjective Evaluation

The subjective evaluation was made for only the International Terminal Boulevard texturing experiment. It consisted of roadside and automobile-interior evaluation by five persons involved in the planning of the experiments and the noise-measurement expert of the Virginia Highway and Transportation Research Council.

The roadside evaluation consisted of 5-min evaluation periods at each site while normal traffic traversed the site. During these periods the evaluation team stood approximately 7.6 m (25 ft) from the pavement edge. Each member of the team was asked to consider, for each pavement texture, the intensity and the pitch of noise, the degree of annoyance, the background noise, and the relations between pavement-tire noise and engine and exhaust noise. The evaluation inside the automobile was conducted by having the evaluation team ride over the variously textured sections at approximately a 72-km/h (45-mph) speed limit in a medium-size and a full-size automobile.

The evaluation team reached a strong consensus on the following observations:

1. Roadside observation—(a) For trucks and buses, engine and exhaust noise completely masked tire-pavement noise, and no effect of pavement texture was discernible; (b) for automobiles, although most of the experimental textures were slightly louder than the longitudinally tined texture with 19-mm (0.75-in) groove spacing (the standard at the time), only the sprinkled texture was considered objectionable.
2. Observation inside the automobile—The sprinkled and the dimpled textures and the 38-mm (1.5-in) groove spacing of the transversely tined texture produced a discernibly more intense level of noise, but only the latter was considered objectionable.

Roughness Studies

Roughness tests were conducted in August 1976 on all the experimentally textured pavement sections. These tests were run at a speed of 64 km/h (40 mph) with a Mays roadmeter, and a summary of the results is given in Table 1.

The tests were conducted in all lanes included in the texturing experiment. Thus, the results given for the I-64 projects represent average roughness for four lanes, and those given for International Terminal Boulevard represent average roughness for two lanes.

It appears from the results that the worn, longitudinally tined textures on I-64 were somewhat smoother than the new textures on International Terminal Boulevard. With the exception of the sprinkled-aggregate section on International Terminal Boulevard, all the roughnesses measured were in the range considered acceptable for Virginia. The 108-units/km roughness of the sprinkled aggregate is significantly rougher than

Figure 5. Longitudinally tined texture with 19-mm (0.75-in) groove spacing.

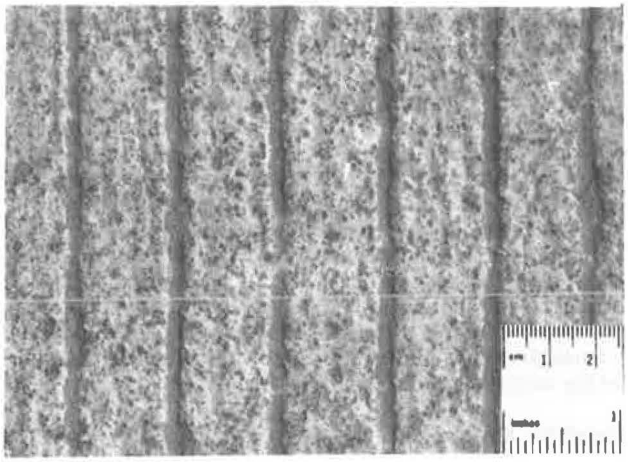


Figure 6. Transversely tined texture with 76-mm (3-in) groove spacing.

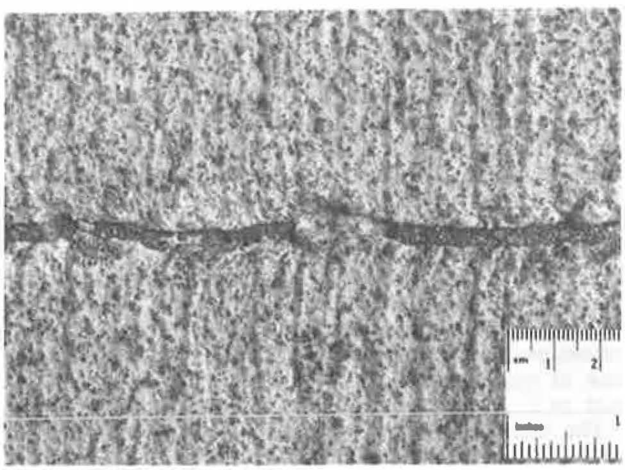


Figure 7. Exposed-aggregate texture.

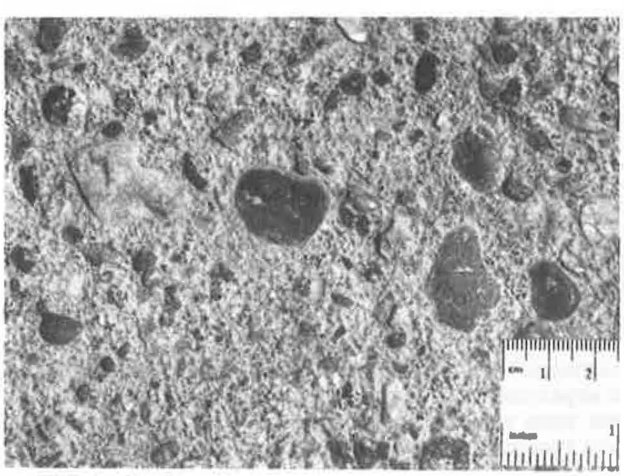


Figure 8. Sprinkled-aggregate texture with maximum aggregate size of 19 mm (0.75 in).

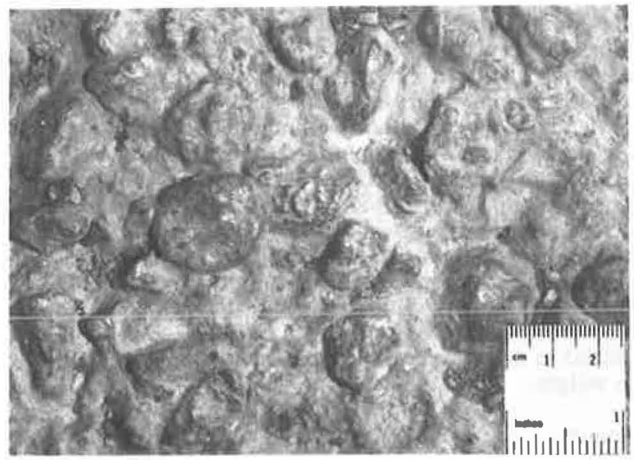


Figure 9. Sprinkled-aggregate texture with maximum aggregate size of 13 mm (0.5 in).

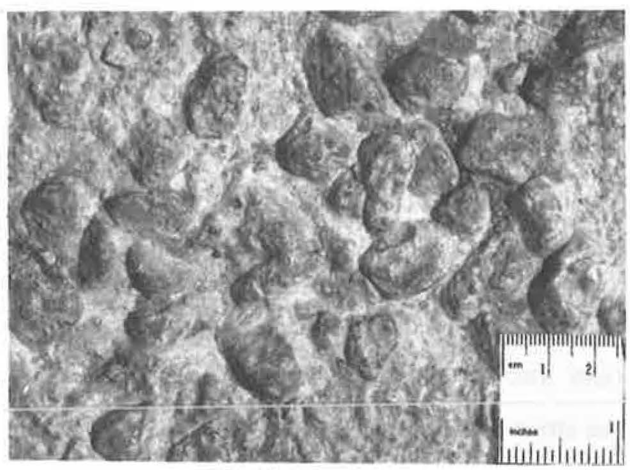
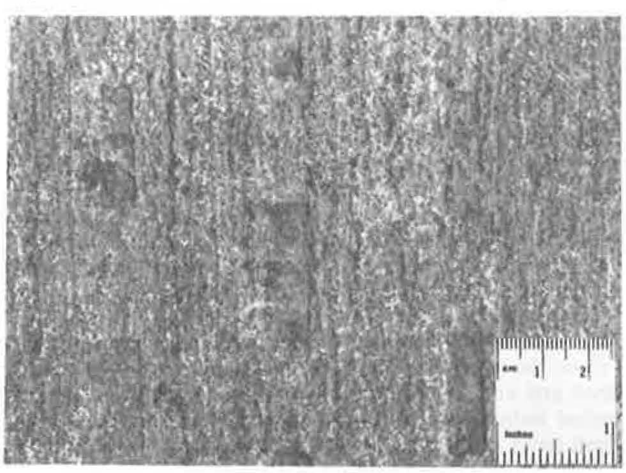


Figure 10. Dimpled texture.



that for any other test section and probably reflects the difficulty in spreading the aggregate uniformly on the pavement surface and the undulations created by attempting to roll the aggregate into the surface. None of the tined textures had roughness values outside the normally expected range, and the variations occurring were very likely related to problems inherent in constructing the short sections represented by the tests. These findings are in agreement with those from a similar study conducted in Louisiana (8).

As a result of the tests and the above discussion, it is concluded that roughness considerations would not dictate the type of texture desired except that they would eliminate sprinkle-type applications.

SKID-RESISTANCE STUDIES

In the design of pavement surfaces that will provide enough tire-pavement friction to ensure vehicle stability, there are two major areas of concern. The first is fabricating the surface with polish-resistant materials so that sufficient friction can be maintained between the tire and the pavement surface when thin films of water are present. As mentioned earlier, this consideration has not presented a problem in building PCC pavements in Virginia because highly polish-resistant, siliceous aggregates abound in the eastern portion of the state where such pavements are popular. The second concern is providing sufficient means for water drainage so that only thin films of water will be encountered.

Potential skid resistance can be measured by using the ASTM E 274-70 test method but, because the tire used in this test method has good tread and thus provides channels for the passage of water, the test method is inadequate for evaluating the capacity of the pavement surface to provide drainage. To circumvent this problem, the skid tests performed used two types of tires—treaded for the evaluation of skid resistance and bald or untreaded for the evaluation of surface drainage potential.

It should be remembered that in the ASTM E 274-70 test method only a thin film of water is applied and therefore at low test speeds there is little need for escape passages for the water. However, as test speeds increase, the escape channels are needed more and more. Even the thin films of water used in the ASTM test will result in rapid deterioration of skid numbers when bald tires are used unless the pavement surface provides a good means by which water can escape.

Skid tests were conducted on several occasions, at multiple speeds, and with both treaded and untreaded tires on all experimental test sections. Only once, however, were the tests performed on all of the sections

Figure 11. Transversely tined texture with 19-mm (0.75-in) groove spacing.

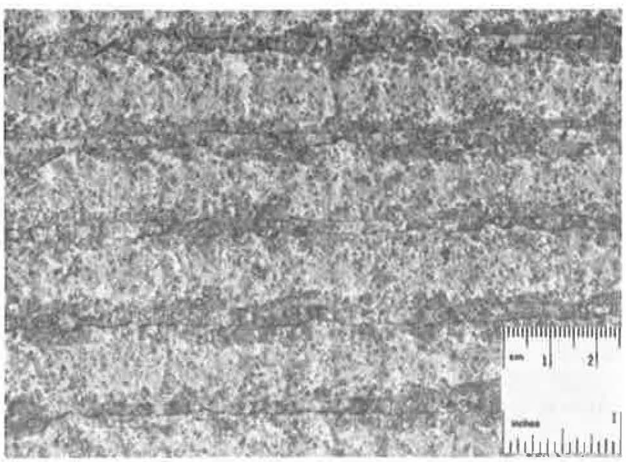


Figure 12. Texture with longitudinally tined and transversely tined grooves spaced 19 and 76 mm (0.75 and 3 in) apart respectively.

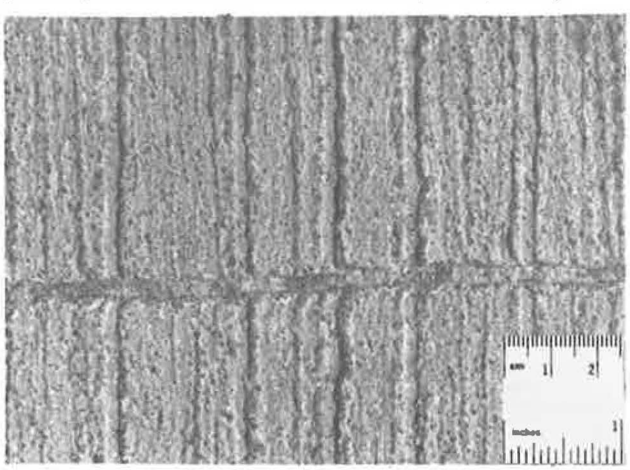


Figure 13. Transversely tined texture with 38-mm (1.5-in) groove spacing.

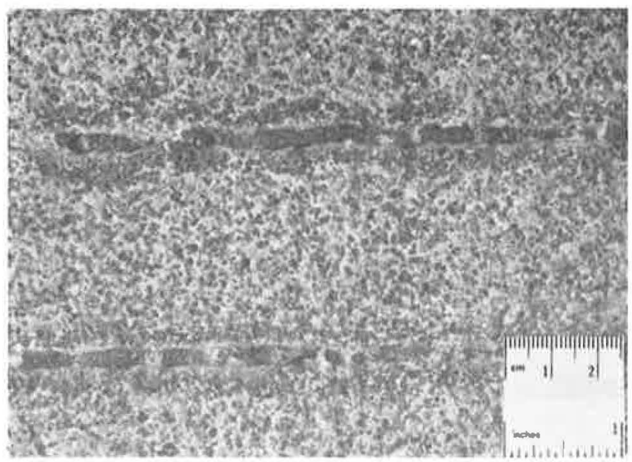
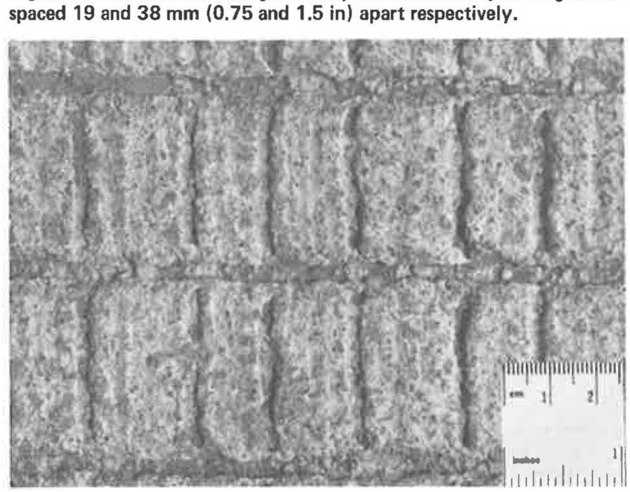


Figure 14. Texture with longitudinally and transversely tined grooves spaced 19 and 38 mm (0.75 and 1.5 in) apart respectively.



during the same week. Because skid numbers are known to vary with varying weather conditions, only the tests performed between July 29 and August 4, 1976, are discussed here for reasons of clarity and simplicity. The reader is assured that no data have been omitted that would change the findings.

Skid data are given in Table 2. Note that the skid numbers for the treaded-tire tests are excellent for all surfaces, regardless of the texture; in fact, the highest number shown is that for the passing lane of the burlapped surface. These findings should be expected when it is remembered that the tires provide ample escape for the thin films of water applied in the ASTM test. On the other hand, the bald-tire test results for the burlapped and the dimpled surfaces are quite low. This indicates not only that if bald tires are used little tire-pavement friction will occur in a moderate rain but also that during a heavy rain a well-worn tire might lack friction because the tread might be insufficient to provide escape for the water.

Each of the other surfaces provided good to very good skid resistance. When the test results for bald tires are carefully examined, the surfaces are rated in the following order (1 mm = 0.039 in):

Figure 15. Contrast during rainfall between longitudinally and transversely tined texture (foreground) and conventional, longitudinally tined texture (background).

Rating	Surface
1	Longitudinally tined with 19-mm groove spacing plus transversely tined with 38-mm groove spacing
2	Longitudinally tined with 19-mm groove spacing
3	Longitudinally tined with 19-mm groove spacing
4	Transversely tined with 38-mm groove spacing
5	Sprinkled aggregate
6	Washed mortar
7	Transversely tined with 76-mm groove spacing
8	Longitudinally tined with 19-mm groove spacing
9	Dimpled
10	Burlapped

Although at least the first 8 of these surfaces appear to be acceptable with respect to skid resistance, other factors—some of which have already been mentioned—need to be considered. The authors feel that, though the washed-mortar surface provides good skid resistance, it will wear rather rapidly and approach the same condition as the burlapped surface in a few years. It also involves an expensive finishing process.

The sprinkled-aggregate surfaces also add considerable expense to the finishing operation and, as discussed earlier, produce more noise than some of the other surfaces. Because the state of Virginia has a sufficient supply of polish-resistant aggregate, sprinkling does not seem necessary; but in states where polish-resistance aggregates are scarce, this means of providing skid resistance might prove desirable.

As mentioned earlier, the transversely tined surface with 38-mm (1.5-in) groove spacing produces an undesirable noise level inside the traveling vehicle. Because other surfaces provide as good or better skid resistance, this surface should not be considered. This finding would, of course, also eliminate the surface that is longitudinally tined with 19-mm (0.75-in) groove spacing and transversely tined with 38-mm groove spacing.

The limited data available for the four remaining surfaces appear to indicate that the transversely tined surface with 19-mm groove spacing and the surface that is longitudinally tined with 19-mm groove spacing and transversely tined with 76-mm groove spacing provide better skid resistance. Both of these surfaces provide channels for lateral water runoff; the second one also has been credited with providing lateral stability on curves. Of course, the transversely tined surface with 19-mm groove spacing provides the most channels for lateral runoff of water. The authors there-

Table 1. Summary of noise (sound pressure) levels and roughness tests.

			Sound Pressure (dBA)			
			Roadside			Mays Roadmeter Roughness (units/km)
Pavement Type	Location	Texture	Rib Tire	Snow Tire	Interior of Automobile (rib tire)	
Bituminous concrete	Various	13-mm maximum size	78.8	80.9	61.8	—
Bituminous concrete	Various	19-mm maximum size	79.2	81.8	—	—
Surface treatment	Various	Protruding aggregate	84.3	84.7	61.6	—
PCC	I-64	Harsh burlap (worn)	—	—	61.6	47
PCC	I-64	19-mm longitudinally tined (worn)	—	—	59.8	44
PCC	International Boulevard	19-mm longitudinally tined	82.5	81.1	61.0	64
PCC	International Boulevard	76-mm transversely tined	79.6	86.3	61.0	61
PCC	International Boulevard	19-mm transversely tined	—	82.3	60.8	69
PCC	International Boulevard	19-mm longitudinally tined plus 76-mm transversely tined	79.6	84.4	62.3	60
PCC	International Boulevard	38-mm transversely tined	—	—	61.9	56
PCC	International Boulevard	19-mm longitudinally tined and 38-mm transversely tined	80.7	83.7	62.2	49
PCC	International Boulevard	Exposed aggregate	85.8	84.5	60.0	71
PCC	International Boulevard	Sprinkle[a]	85.8	84.5	62.6	108
PCC	International Boulevard	Dimpled	86.9	85.1	60.3	53

Note: 1 km = 0.62 mile; 1 mm = 0.039 in.

[a]The two sprinkle-mix sections were treated as one section in both roadside and automobile-interior tests.

Table 2. Skid number as a function of texture, vehicle speed, and tire tread.

Location	Texture	Skid Number 32 km/h Treaded Tire	32 km/h Bald Tire	48 km/h Treaded Tire	48 km/h Bald Tire	64 km/h Treaded Tire	64 km/h Bald Tire	80 km/h Treaded Tire	80 km/h Bald Tire	96 km/h Treaded Tire	96 km/h Bald Tire
I-64											
Traffic lane	Burlapped	62	42	58	29	51	20	46	19	42	16
Passing lane	Burlapped	71	50	69	39	61	35	56	28	54	23
	19-mm longitudinally tined	62	48	62	46	56	36	54	34	52	30
International Boulevard	19-mm longitudinally tined	58	48	54	41	50	33	46	32		
	76-mm transversely tined	63	52	52	42	49	37	47	34		
	Washed mortar	56	51	52	47	48	33	46	37		
	Sprinkled aggregate (small)	57	49	51	42	46	39	43	39		
	Sprinkled aggregate (large)	53	47	49	41	43	37	42	37		
	Dimpled	62	45	55	33	51	26	46	23		
	19-mm transversely tined	66	58	60	51	54	47	52	39		
	19-mm longitudinally tined and 76-mm transversely tined	63	58	60	51	57	43	53	40		
	38-mm transversely tined	62	56	60	50	55	41	50	37		
	19-mm longitudinally tined and 38-mm transversely tined	62	59	59	50	56	47	52	43		

Note: 1 km = 0.62 mile; 1 mm = 0.039 in.

fore feel it would be desirable to use that surface on tangents and use the other surface—i.e., the longitudinally tined with 19-mm groove spacing and transversely tined with 76-mm groove spacing—on curves. The operation required to provide either of these two finishes should add little or no cost to the placement of the pavement.

CONCLUSIONS

The most important requirement in providing a skid-resistant pavement is that the surface be fabricated with polish-resistant aggregates. The best texturing techniques known can be applied, but if the aggregates are polish susceptible the pavement will still become slippery. With this thought in mind, the following conclusions are presented.

1. In view of current traffic volumes, a burlap drag alone does not provide the initial harshness desired on a PCC riding surface.
2. The burlap-drag finish applied to I-64 at Charlottesville was much harsher than previous burlap finishes applied to surfaces in Virginia, but the wear with age was substantial.
3. Because the tined surface on I-64 east of Richmond has been subjected to an ADT of 14 000 vehicles/d for 4 years and has shown little wear, it is felt that, in the absence of studded-tire traffic, tined surfaces will provide an adequately harsh surface for many years under most traffic conditions.
4. The surfaces in this study provide good to very good skid resistance. When the bald-tire test results are examined carefully, the tined surface with a combination of longitudinal and transverse striations spaced 19 mm and 38 mm (0.75 and 1.5 in) apart respectively rates best, followed by the transversely tined surface with striations 19 mm apart. In addition, all the surfaces except those with a dimpled or a burlapped finish are certainly good from a skid-resistance standpoint.
5. Although the washed-mortar surface provides good skid resistance, it is believed that it will wear rapidly and approach the same condition as the burlapped surface in a few years. Further, this finishing process involves considerable added expense.
6. The sprinkled-aggregate surface also adds considerable expense to the finishing operation and produces more noise than some of the other surfaces.
7. In noise tests conducted by Noble (6, 7), noise level for exposed-aggregate, sprinkled, and dimpled textures tended to be somewhat greater than those for any of the tined surfaces or bituminous concrete surfaces found in Virginia that generate noises of similar intensities.
8. In a subjective noise evaluation, it was concluded from roadside observations that (a) for trucks and buses, engine and exhaust noise completely masked tire-pavement noise and (b) noise levels for most of the experimental textures were slightly greater than those for the longitudinally tined surface with 19-mm groove spacing, which was then the standard texture, but only the sprinkled texture was considered objectionable. Observations made from inside the automobile led to the conclusion that the sprinkled and the dimpled surfaces and the surface transversely tined with grooves spaced 38 mm (1.5 in) apart produced a discernibly intense noise, but only the noise from the transversely tined texture was considered objectionable.
9. As a result of roughness tests conducted at 64 km/h (40 mph) with a Mays roadmeter, it was concluded that roughness considerations would not dictate the type of texture desired except in the case of sprinkle-type applications, which in this study were significantly rougher than any of the other test surfaces.
10. All things considered, the two most desirable surfaces are the surface with transversely tined striations 19 mm apart and the surface with longitudinally tined striations 19 mm apart and transverse striations 76 mm (3 in) apart. Both of these surfaces provide channels for lateral runoff of water; the latter one also has been said to provide lateral stability on curves. Of course, the surface with transverse striations spaced 19 mm apart provides the most channels for lateral water runoff. We thus feel that it would be desirable to use that surface on tangents and the other of the two on curves.

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Two-Course Construction of an Internally Sealed Concrete Bridge Deck

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A unique form of two-course construction—placement of a 51-mm (2-in) wax-bead concrete overlay on a fresh, conventional-concrete lower course after a 1-h wait period—was used in construction of a bridge deck near Seattle, Washington. Construction and evaluation of test slabs before placement of the deck showed that the direct tensile bond developed between the two courses was as great as the direct tensile strength of the wax-bead concrete. Other test-slab construction and evaluation work showed that the more conventional approach of placing a 51-mm overlay on a 1-d-old hardened lower course (after a water-and-sand blast and grout placement) also resulted in adequate bond. Cores evaluated after construction and heating of the deck confirmed the adequacy of the bond and the complete sealing of the top 13 to 25 mm (0.5 to 1.0 in) of the concrete overlay.

This report describes two-course construction techniques used in the construction of an experimental project undertaken by the Washington State Department of Highways using internally sealed concrete (1). The project required the use of wax beads in the top 51 mm (2 in) of a concrete bridge deck to develop an internally sealed concrete for the protection of the reinforcing steel of the deck. The structure itself—a three-span continuous structure with spans of 22.6, 24, and 22.6 m (74, 80, and 74 ft)—is a railroad overcrossing on I-90 in the western foothills of the Cascade Mountains approximately 48 km (30 miles) east of Seattle. The northern spans of the twin prestressed concrete girder bridges, which received the internally sealed concrete bridge-deck treatment, carry the westbound lanes of traffic. The structure has a 180-mm (7-in) thick, cast-in-place, reinforced concrete roadway slab supported by precast, prestressed concrete girders. The 180-mm-thick concrete deck is comprised of 127 mm (5 in) of Washington State standard structural class AX concrete, and the top 51 mm (2 in) is internally sealed with wax. The structure is 69.5 m (228 ft) in length and has a roadway width of 15.8 m (52 ft) between the concrete curbs. The roadway is on a curve with a radius of 914 m (3000 ft), and the piers are constructed at a skew angle of approximately 47° to the centerline of the roadway. The deck reinforcement consists of No. 6 reinforcing bars spaced 180 mm center to center, both top and bottom. The specified thickness of the concrete cover over the top transverse reinforcing steel is 51 mm.

MATERIALS

The concrete used in this project met Washington State specifications for class AX concrete, which is designed for a compressive strength of 27.6 MPa (4000 lbf/in^2). The sand has a fineness modulus of about 3.2; the grading for Washington State No. 5 coarse aggregate is as follows:

	Percent Passing by Weight	
Sieve	Minimum	Maximum
25 mm (1 in)	100	–
19 mm (¾ in)	80	100
10 mm (⅜ in)	10	40
4.8 mm (No. 4)	0	4

All slab concrete is non-air-entrained because the internal sealing will provide the necessary freeze-thaw protection. This avoids an unnecessary sacrifice of strength that would occur if the concrete contained both wax beads and entrained air.

The typical concrete mixes used for the test slabs and the structure are as follows (1 kg = 2.205 lb and 1 L = 0.264 gal):

Item	Standard Class AX Concrete	Class AX Concrete With Wax Beads
Cement, kg	276.6	276.6
Fine aggregate, kg	642.2	496.1
Coarse aggregate, kg	815.4	815.4
Wax beads, kg	–	53.5
Total water, L	113.6	113.6

The maximum amount of water allowed for class AX concrete by Washington State standard specifications is 173 L/m^3 (35 gal/yd^3). The concrete was centrally mixed and delivered.

TEST SLABS

Four test slabs were required to evaluate the various options available for two-course construction and concrete deck heating. These test slabs were each 2.4 m (8 ft) wide by 3.7 m (12 ft) long and had the same deck thickness and reinforcing steel that are called for on the railroad overcrossing structure. Figures 1 and 2 show the test-slab forms and the reinforcing steel.

In September 1975, the test slabs were cast. In all cases, the bottom 12.7 cm (5 in) of concrete was placed first and was vibrated with internal vibrators. The second course of concrete was then placed in the following manner (1 mm = 0.039 in and 1 MPa = 145 lbf/in^2):

Slab	Procedure
1	Bottom 129 mm standard class AX concrete placed; top 51 mm wax-bead concrete placed 1 h later by concrete bucket
2	Same as test slab 1 to construction joint (top 51 mm) Bottom 129 mm standard class AX concrete placed; top 51 mm wax-bead concrete placed next day; surface cleaned by 8.3-MPa waterjet; grout placed
3	Bottom 129 mm standard class AX concrete placed; top 51 mm wax-bead concrete placed next day; entire surface cleaned by 8.3-MPa water-and-sand jet; grout placed
4	Bottom 129 mm wax-bead concrete placed; top 51 mm wax-bead concrete placed same day by concrete pump about 30 min after first pour

The ambient temperature during placement was approximately 18°C (65°F). There was no displacement of the underlying standard concrete during the placement of the top course of wax-bead concrete in slabs 1, 2, or 4 (Figure 3). The finishing of the top course was done

Figure 1. Placement of reinforcement for the four test slabs (test slab 1 in foreground).

Figure 3. Second (wax-bead) concrete course being deposited at test slab 1 from concrete bucket with no displacement of underlying concrete.

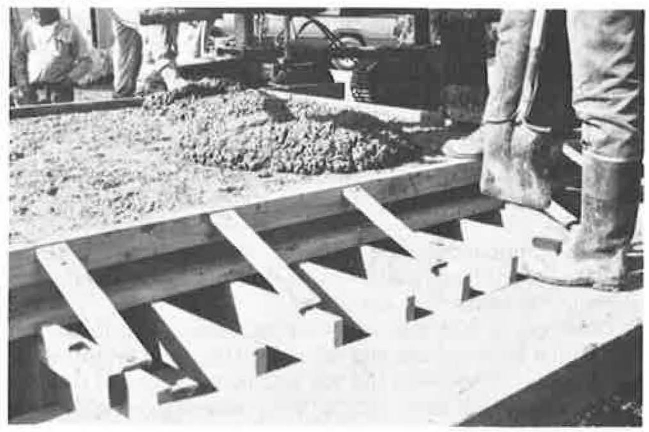


Figure 2. Slab reinforcement (deepened sections represent prestressed, precast concrete girders).

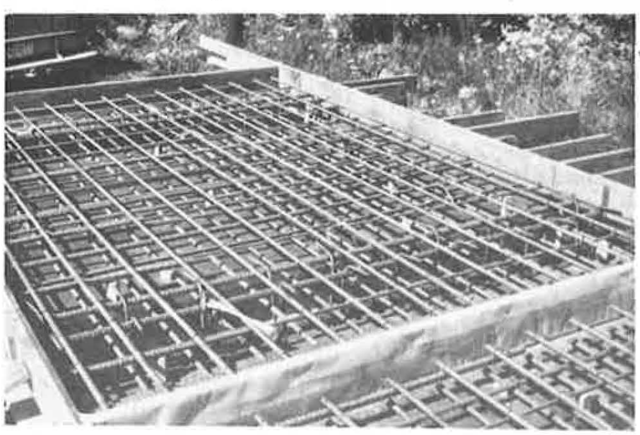
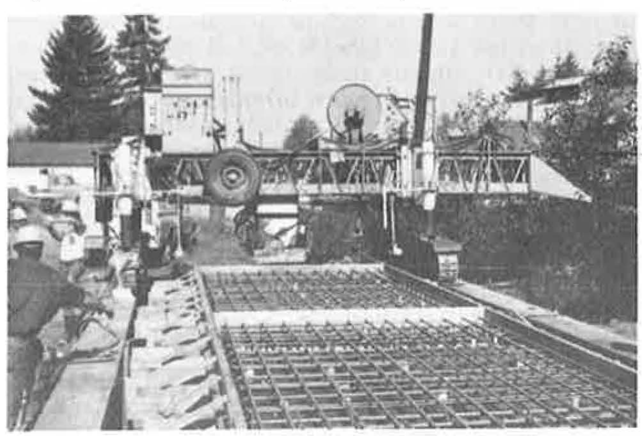


Figure 4. Gomaco 350 finishing machine in position at test slab 1.



by using a Gomaco 350 finishing machine with a vibrating pan for compaction (Figure 4). The finishing machine has twin rollers 123 cm (48 in) long and 20.5 cm (8 in) in diameter, in front of which is a strike-off auger and the vibrating pan (Figures 5 and 6). The effectiveness of the vibrating pan appeared to be reduced by positioning the strike-off auger following it. This was corrected on the bridge deck by placing the vibrating pan between the rollers and the strike-off auger. No attempt was made to smooth or give any special treatment to the surface of the standard concrete (lower course). As shown in Figure 7, a rough surface was achieved through the normal activities of the work crew.

The surface preparation for the second day's placement (half each of slab 2 and slab 3) called for the use of an 8.3-MPa (1200-lbf/in^2) waterjet with a flow of 76 L/min (20 gal/min) and the capability of introducing sand into the jet. The 1.4-m (4.5-ft) unfinished length of test slab 2 was cleaned by using the waterjet without sand. Test slab 3 was cleaned by introducing sand into the waterjet stream. A sand deposit left on the surface of the concrete had to be broomed off before placement of the overlay. The jet also left puddles of water in the depressions of the surface, and these were removed by blotting with burlap. For a larger area, water removal would be accomplished more efficiently by using compressed air. The entire area to be overlaid was coated with a cement-sand slurry (in a 1:1 mixture) that was brushed into the surface (Figure 8).

The overlays on a portion of slab 2 and slab 3 and both lifts of slab 4 were then placed by using a Thompson 102-mm (4-in) piston-type concrete pump (Figure 9). The initial operation involved the placement of the lower course for slab 4 (to the top of the reinforcing steel) and consolidation of the concrete by use of internal vibrators. After the first lift of test slab 4 had been placed, the second lifts for test slabs 2 and 3 were placed. The second lift of test slab 4 was placed last, approximately 30 min after the first lift was placed.

The slabs were covered overnight with plastic sheeting. The next day the plastic sheeting was removed, and the slabs were covered with burlap and kept wet for 10 d. Curing compounds were not used because it was believed that they would inhibit the escape of moisture during the subsequent heating of the slabs.

Test cores 102 mm in diameter were taken from the test slabs both before and after heating to determine the bond strength between the top and bottom lifts of concrete. A design bond strength for two-course construction and the best test method for evaluating bond strength have not been universally established. Furr and Ingram (2) have estimated the horizontal shear at the interface of a 180-mm (7-in) uncracked slab and a 51-mm (2-in) overlay under an AASHTO H20 truck to be 440 kPa (64 lbf/in^2). Work done in England (3) indicated that a bond strength as low as 280 kPa (40 lbf/in^2) may be adequate for an overlay. In this study, it was decided that a direct tensile bond test (Figure 10) would be used and that success of a two-course construction procedure would be indicated by tensile breaks in the concrete rather than at the bond line. In addition, it was believed that the tensile break strength should be in excess of 689 kPa (100 lbf/in^2).

The results of the core studies are given in Table 1. All three two-course procedures (overlay on fresh lower course after 1-h delay, overlay on 1-d-old lower course after water blast and grout, and overlay on 1-d-old lower course after water-and-sand blast and grout) were studied. A bond break did not occur in any of the specimens; rather, all breaks occurred in the wax-bead concrete above the bond line. The tensile break strength for cores from the unheated slabs varied from 0.69 to 1.69 MPa (101 to 246 lbf/in^2). All three two-course construction procedures appeared to be acceptable in that bond breaks did not occur. However, the magnitude of the tensile break values indicated that the best procedures were

1. Placing the 51-mm (2-in) wax-bead concrete on the fresh lower course concrete after a 30-min to 1-h delay and
2. Placing a 51-mm wax-bead concrete overlay on a hardened conventional concrete after water-and-sand blasting and following this by placement of a portland cement grout.

The test slabs were heated by using very hot air, electric blankets, and gas-fired infrared and electric infrared techniques. Because the test-slab concrete was saturated by rains, the high-intensity heating systems caused many spalls approximately 19 mm (0.75 in) deep. The slower heating of test slab 4 by use of electric blankets did not cause spalling. Thus, after heating, bond testing could be performed on this slab only. Slab 4 had been constructed by placing a 51-mm overlay on the fresh lower course concrete after a 30- to 45-min delay. The heating process involved placing electric blankets covered with fiberglass insulation on the slab surface. The blankets, shown in Figure 11, received 2 kW/m^2 (190 W/ft^2) of electrical power from a diesel generator, and 5 h of heating were required to achieve a temperature of 85°C (185°F) at the 51-mm depth. The maximum temperature recorded on thermometers placed on the concrete surface under the blankets was 167°C (332°F); the outside air temperature during heating was approximately 4°C (40°F).

The results of bond testing after heating indicate that the bond was not adversely affected by electric-blanket heating. As in the tests conducted before heating, no bond breaks occurred; instead, tensile breaks occurred in the wax-bead concrete.

BRIDGE DECK

The bridge deck was placed on November 11, 1975. The weather was overcast; a light drizzle fell during the late morning and afternoon. The rain was not heavy enough, however, to have an adverse effect on the concrete-placing operation. The temperature was approximately 10°C (60°F).

The results from the test-slab work indicated that the simplest method was to place both concrete courses the same day. The contractor elected to place all concrete by pumping. To avoid confusion and the possibility of a mix-up in the type of concrete being placed, two concrete pumps were used—one for the standard and the other for the wax-bead concrete (Figures 12 and 13). A Bidwell finishing machine was used for the bridge deck, and the vibrating pan was placed between the roller and the strike-off auger (Figure 14). The placement operation for the standard class AX concrete was the same as that used for the test slabs. The class AX concrete was placed in the forms, consolidated by internal vibrators, and struck off to the top of the upper reinforcing steel mat with hand tools. No other finishing was done on the class AX concrete.

The standard class AX concrete was delivered to the project in agitator trucks of 7-m^3 (9-yd^3) capacity; the wax-bead concrete was delivered in 4.9-m^3 (6.4-yd^3) batches in mixer trucks. The odd batch size for the wax-bead concrete was chosen to simplify addition of the wax beads at the bridge site (Figure 15). (After the wax beads were added, the concrete was remixed for 85 drum revolutions.) The first three concrete truck loads ordered were the standard class AX concrete to give the standard concrete placement approximately a 9.1-m (30-ft) lead on the second course of wax-bead concrete. Delivery was scheduled so that two 7-m^3 loads of standard class AX concrete were delivered to the project for every 4.9-m^3 load of wax-bead concrete. The 9.1-m lead the standard concrete had in placement represented, in time, approximately 1 h. During the latter portion of

Figure 5. Gomaco 350 finishing machine in operation with strike-off auger positioned between vibrating pad and rollers.

Figure 6. Rear view of Gomaco 350 machine in operation.

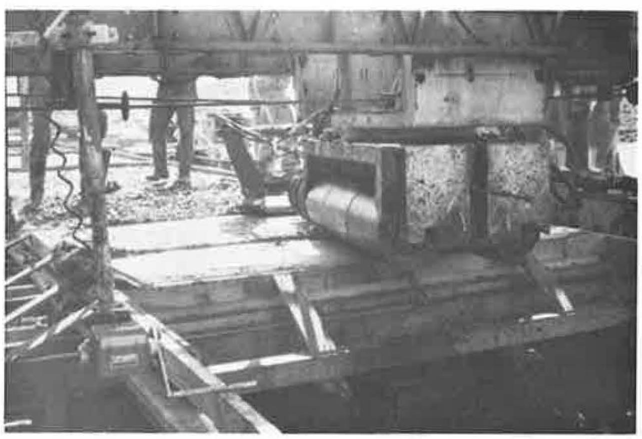


Figure 7. Texture of surface of first concrete course for test slab 3.

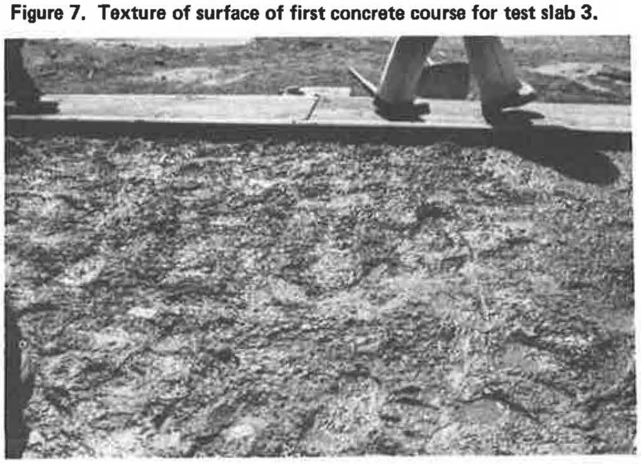


Figure 8. Grout slurry being broomed into concrete surface as a bonding aid.

Figure 9. Thompson 10.2-cm (4-in) piston-type concrete pump placing concrete for test slabs.

Figure 10. Apparatus for testing direct tensile bond.

Table 1. Results of bond testing.

Slab Number	Two-Course Procedure	Direct Tensile Bond (MPa)	Location of Break
1	Wax-bead overlay placed on fresh, conventional concrete after 1-h delay	1.34	1.3 cm above bond line
2	Wax-bead overlay placed on hardened lower course after water blast and portland cement grout	0.69	Slightly above bond line
3	Wax-bead overlay placed on hardened lower course after water-and-sand blast and portland cement grout	1.69	1.3 cm above bond line
4	Wax-bead overlay on fresh, wax-bead lower course after 30- to 45-min delay	1.52	2.5 cm above bond line

Notes: 1 MPa = 145 lbf/in²; 1 cm = 0.39 in.
Only test slab 4 could be tested after heating. Results after electric-blanket heating were the same

Figure 11. Electric blankets on test slab 4.

Figure 12. Bridge deck pour in operation: concrete pump in foreground placing standard class AX concrete and pump in background placing wax-bead concrete overlay.

Figure 13. Lower course of conventional concrete being raked off at level of top mat of reinforcing steel after placement and consolidation by internal vibration.

Figure 14. Bidwell finishing machine in operation with vibrating pad located between rollers and strike-off auger.

the deck pour, the lead distance was reduced to approximately 4.5 m (15 ft), but there was no evidence of displacement of the standard concrete during the placement of the wax-bead concrete. The deck pour, which was completed in approximately 8 h, required 149 m^3 (195 yd^3) of standard class AX concrete and 61 m^3 (80 yd^3) of wax-bead concrete. The placement of the deck in two courses required an operator for one extra concrete pump and two additional concrete finishers.

The two-course technique of bridge-deck construction was checked by taking cores 51 mm (2 in) in diameter from the bridge deck after it had been heated by an electric-blanket heating system. Cores of this size were used rather than larger cores to minimize the chance of cutting the reinforcing steel. It was realized that the 51-mm cores would not provide as accurate an estimate of actual bond strength [because of the use of 25-mm (1-in) maximum-size aggregate], but the purpose was to determine whether bond breaks occurred rather than to determine the tensile breakstrength of the concrete.

No bond breaks occurred when direct tensile bond tests were performed on four 51-mm-diameter bridge-deck cores; instead, the breaks occurred 25 to 38 mm (1 to 1.5 in) above the bond line in the wax-bead concrete. Thus, the results indicate that the bond between the two layers was stronger than the wax-bead concrete. The direct tensile break strengths varied from 0.76 to 0.97 MPa (110 to 140 lbf/in²) and averaged 0.83 MPa (121 lbf/in²). Compressive strength tests on a companion 51-mm core yielded a strength of 35.4 MPa (5130 lbf/in²), and the tensile splitting strength of another 51-mm deck core was 4.6 MPa (665 lbf/in²).

Thus, the data indicate that the two-course bridge-deck construction technique of placing a wax-bead over-

Figure 15. Wax beads being added by concrete bucket to already mixed concrete at jobsite.

lay on a fresh concrete lower course within 1 h of lower course placement was successful. In addition, tests on other cores indicated that the top 13 to 25 mm (0.5 to 1 in) of the deck was completely sealed by the wax-bead process.

CONCLUSIONS

The results of this project indicate that a good bond can be developed between two concrete courses (internally sealed and conventional concretes) when both courses are placed in the same operation. The integrity of this bond was not reduced by the temperatures it experienced during the heat treating of the wax-bead concrete, temperatures ranging from 85° to 93°C (185° to 200°F).

When the second concrete course was placed on the test slabs 1 d after the first course was placed, both surface preparation procedures—8.3-MPa (1200-lbf/in^2) water blast and grout and 8.3-MPa water-and-sand blast and grout—proved to be effective. However, the water-and-sand blast and grout procedure yielded higher direct tensile strength of the bond.

Recommended areas of future research on two-course construction are as follows:

1. Further study should be done on the effectiveness of various methods of consolidating the second course of concrete.
2. More research should be performed on the water-blast procedure (minus sand) because it was a simpler and thus less costly procedure than the water-and-sand blast cleaning.

ACKNOWLEDGMENTS

The test-slab and bridge-deck construction described here was accomplished as a joint effort between the Washington State Department of Highways and the Federal Highway Administration. State highway department personnel who contributed significantly to the effort were Jack Launceford, project engineer; Mike Myette, assitant project engineer; and Tom Marshall, assistant materials engineer. The bridge contractor was Lockyear and Sons, and the heating subcontractor was Frank Murry of Lahr Corporation. The electric blankets used to heat the deck were manufactured by Johanson Electrotherm.

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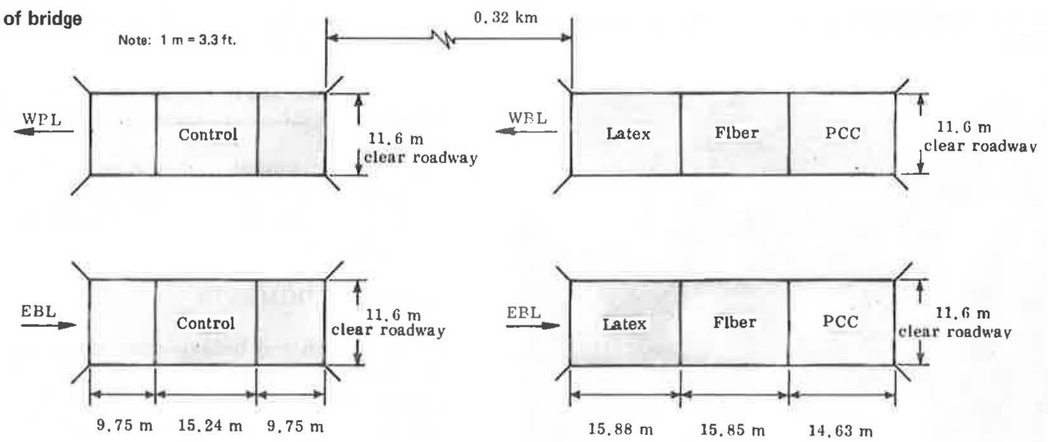
Two-Course Bonded Concrete Bridge-Deck Construction in Virginia

Samuel S. Tyson, Virginia Highway and Transportation Research Council

Six bridge decks were constructed in Virginia by using the two-course bonded technique. The wearing course layers consisted of a high-quality portland cement concrete, a wire-fiber-reinforced concrete, and a latex-polymer-modified concrete. Analyses of construction activities, labor requirements, and cost are used to demonstrate the viability of two-course construction when additional protection of the upper reinforcing steel is warranted. It is shown that normal cover depths and an adequate degree of consolidation were attained in the two-course decks. The significantly better strength of the overlay concretes, as compared to that of concrete placed on two conventional decks used as controls in the study, underscores the primary purpose of the two-course technique, which is to promote the placement of high-quality protective concretes in the upper cover zone of bridge decks. A basis for assessing the future performance of the decks was provided by evaluating them at the age of 1 year, before they were opened to traffic and before application of deicing salts.

In June 1974, the two-course bonded technique for concrete bridge-deck construction was used in the construction of six bridge decks on the Va-7 bypass over the Norfolk and Western Railway at Berryville, Virginia. A research study was initiated to investigate the construction, the condition, and the 5-year performance of these decks and two control decks constructed nearby on bridges over Va-615. The project plan for the six

Figure 1. Plan view of bridge structures.



two-course decks as well as that for the two control decks is shown in Figure 1. The control decks were constructed under the same contract by using a conventional, single-lift technique.

Three concretes were selected for use in the wearing-course layers: a high-quality portland cement concrete (PCC) with a low water-cement ratio, a concrete reinforced with wire fiber, and a latex-polymer-modified concrete. Each concrete was selected to improve the resistance of the concrete above the reinforcement to the intrusion of corrosive substances, particularly chlorides. Low water-cement ratios reduce the permeability of the cement paste; the most significant improvement is realized by not exceeding a ratio of 0.4 (1). Fiber reinforcement resists the formation of microcracks that permit the accelerated intrusion of deleterious substances. The polymer modifiers resist such cracking and also occupy a portion of the internal pore structure of the concrete.

CONSTRUCTION

The eight study decks had a minimum design thickness of 216 mm (8.5 in) and rested on simply supported steel girders with a center-to-center spacing of 2.5 m (8.5 ft). The decks were approximately 15.2 m (50 ft) in length with a clear roadway width of 11.6 m (38 ft).

The two control spans were constructed by the single-lift technique by using a longitudinal oscillating screed that is commonly used in Virginia for bridge-deck construction. This screed rested on supports beyond the ends of the deck and did not impart loads to it. Internal vibrators were used for consolidation.

A transverse screed was used for the two-course installations. The essential elements of this screed included a surface vibrating unit for consolidation of the concrete, an auger that moved excess concrete forward, and a rotating drum that oscillated transversely over the bridge and finished the deck surface.

The sequence of construction for the two-course decks was planned so that 2 d would elapse between placement of the base layer and that of the overlay concretes on each deck. This delay period was selected because earlier placement of the overlays would risk (a) creating voids around the upper reinforcing steel and (b) causing delaminations from the outset of construction because of the tendency for the bleeding rate of the base-layer concrete to exceed the bleeding rates of the denser concrete overlays. Delays exceeding 2 d would have been acceptable structurally, but the 2-d delay was used because at that time light sandblasting was enough to remove from the surfaces of the base layers any laitance resulting from bleeding.

The high-quality PCC and fiber-reinforced concretes were delivered to the jobsite in conventional, ready-mix trucks. The latex-modified concrete was produced at the jobsite by two mobile mixer trucks. Each concrete overlay was deposited on a hardened base layer from a 0.6-m^3 (0.75-yd^3) capacity crane bucket.

Within 1 h before the placement of the concrete overlays, water was broomed onto the base layers. The intent was to maintain a moist concrete surface without free water. Before the first bucket of PCC or fiber concrete was deposited, a cement slurry with a water-cement ratio of 0.40 was broomed over the deck. No slurry was used with the latex-modified concrete; however, a portion of this concrete was broomed onto the deck surface before overlay placement.

Activities

The activities associated with the construction of the two control spans and the six experimental spans were observed and documented for the purpose of making a comparison of single-lift and two-course construction techniques. Descriptions of the placement procedures were obtained by compiling location charts for each truckload of concrete as the decks were constructed. Typical location charts for concrete placed in the single-lift control decks and in the base layers and the overlays of the two-course decks are shown in Figures 2 and 3 respectively. Figure 2 shows a chart for one of the control decks, and Figure 3 shows charts for the base course and the overlay of a two-course, fiber-reinforced deck. Records were also made of the time sequences for batching, delivering, depositing, screeding, texturing, and curing each truckload of concrete in the study decks. A complete description of the construction activities for each deck has been reported (2).

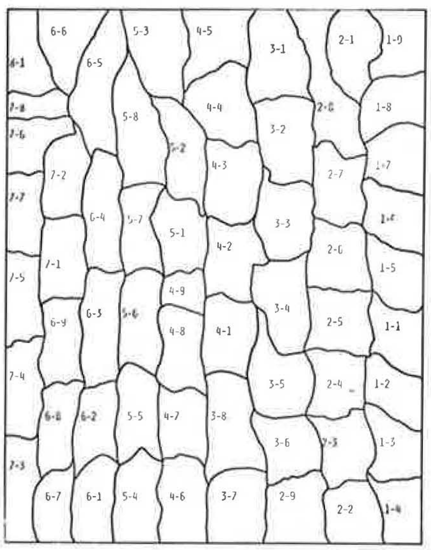
The time records for installation of all truckloads of concrete were grouped and averaged for the single-lift decks and for each layer of the two-course decks. Figure 4 shows these time periods in sequence and shows that the installation activities in the three groups progressed in a similar, orderly way.

The latex-modified concrete, which was continuously batched at the site, required zero delivery time. The wire-fiber-reinforced concrete required that considerable attention be paid to the addition of the fiber during batching. Figure 5 shows the influence of such variables by contrasting the times required to complete all activities from batching through screeding. The mean times for completion were significantly different, primarily because of the differences cited in the batching operations. The figure also shows the "datum time" for computing the time interval required for completion of the screeding activity—shown as the time of the initial deposit from each truckload of concrete. A comparison among the several installations indicated that, on the average, the duration of these critical placement operations, which required exposure and manipulation of the concrete on the decks, was approximately the same for both construction techniques and for all types of concrete in the project.

Labor

A comparison of the relative labor requirements for the two construction techniques was made by using the activities records to compute the average total times for single-lift, base-layer, and overlay installations: 5.5, 4.25, and 3 h respectively. The average total time required to install concrete in the three types of two-course decks was therefore 7.25 h, 33 percent greater than the time required to install conventional, single-lift decks. The total number of worker hours was also 33 percent greater for the two-course decks because the same crew was used to construct all of the decks.

Figure 2. Typical location chart for concrete placed in single-lift decks.



Cost

The costs of the overlay concretes in place were approximately \$18, \$29, and \$38/m^2 (\$15, \$24, and \$32/yd^2) for the high-quality PCC, the latex-modified concrete, and the wire-fiber-reinforced concrete respectively. This range of values is probably representative of the costs that can reasonably be anticipated for these and other overlay concretes in the immediate future. An increase of about 5 percent in the total cost of the bridge superstructure is therefore indicated when the two-course technique is specified instead of the conventional single-lift construction technique.

STRUCTURAL CONDITION

Depth of cover and degree of consolidation were measured in the control decks and in the two-course decks to provide on-site indications of the condition of the structures. In addition, tests to determine the strength and durability characteristics of the base-layer, overlay, and control concretes were conducted by using hardened concrete specimens from the project.

Depth of Cover

The depth of concrete cover over the topmost reinforcing steel of the study decks was determined by making direct probes in the fresh concrete during construction. The results are shown in Figure 6. The total depth of cover provided by the two-course technique was found to be equivalent to that provided by conventional, single-lift construction for identical deck designs. The average depth dimensions of the six two-course decks in the study are shown in the typical transverse section in Figure 7.

Consolidation

A nuclear gauge was used to determine the degree of consolidation for each concrete. The gauge was used in the backscatter configuration rather than the direct-transmission configuration because of the shallow depths of the overlays.

In the field, the rodded unit weight of each type of

Figure 3. Typical location chart for concrete placed in two-course decks.

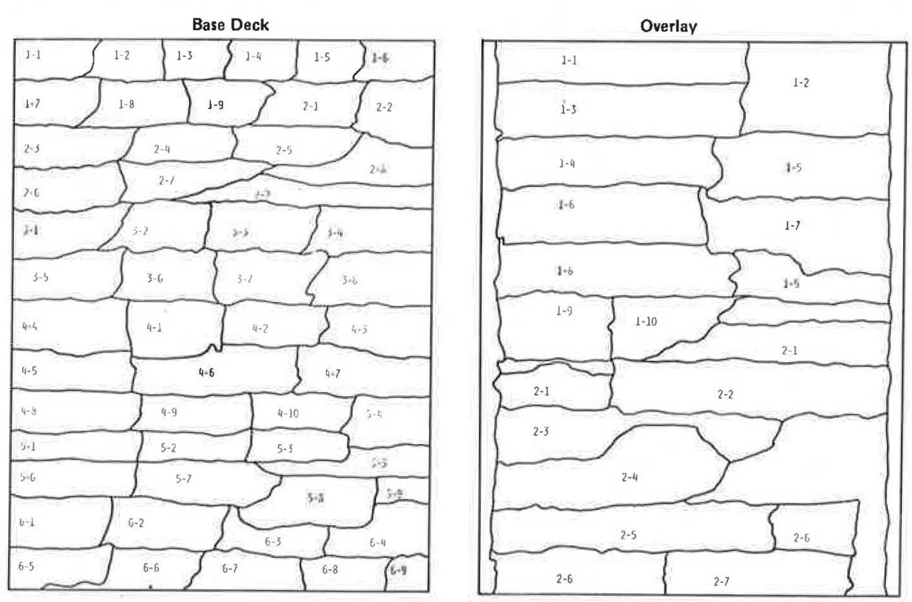


Figure 4. Sequences and average durations for concrete-installation activities from delivery through screeding.

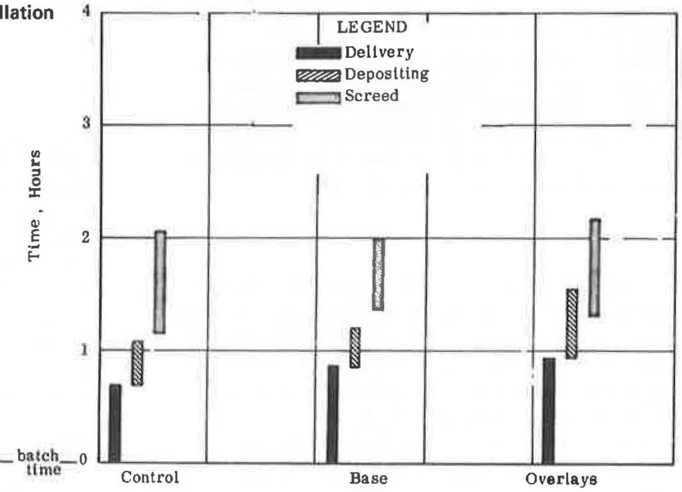
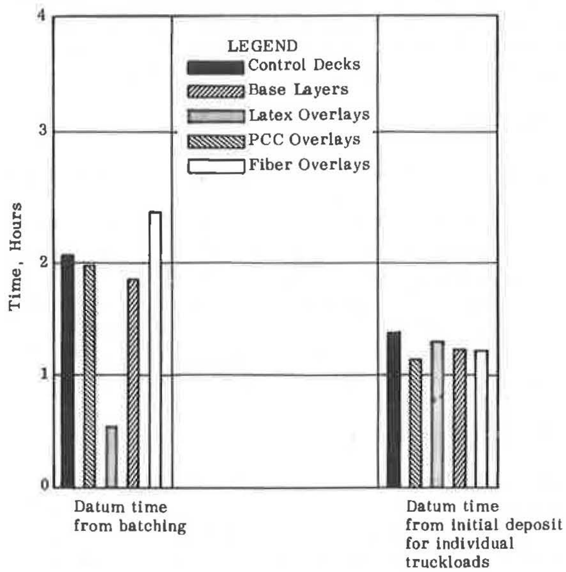


Figure 5. Average times to completion of screeding activity.



fresh concrete was determined, and nuclear density readings were obtained on the surface of the decks immediately after the screeding operation. A summary of these data is given below (1 kg/m^3 = 0.062 lb/ft^3):

Concrete	Nuclear Density (kg/m^3)	Nuclear Density as Percentage of Rodded Unit Weight
Control	2227	97.8
Base layer	2341	99.2
Latex-modified	2344	100.4
PCC	2417	103.0
Wire fiber	2429	104.9

The data seem to indicate good control of consolidation in the study decks.

A core was extracted from each of the two-course decks, and microscopic examinations of vertical sections verified good consolidation in the overlays and revealed the excellent condition of the bond interface between the overlays and the base layers.

Strength and Durability

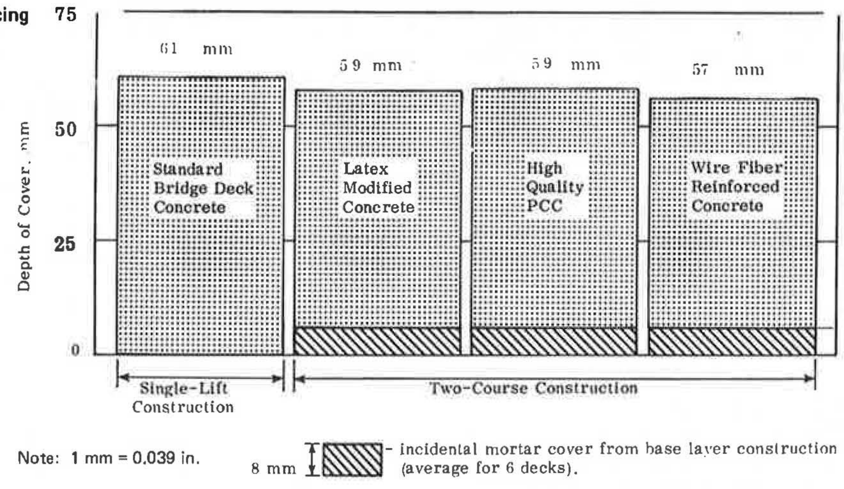
Air contents, slumps, and temperatures were within normal ranges for each concrete, but the average strength of the normal superstructure concrete used in the single-lift control decks was lower than the average strength for the base-layer concrete of the two-course decks, which satisfied the design minimum of 2.76 MPa (4000 lbf/in^2) at 28 d. The only difference in the proportions of the base-layer concrete was the substitution of a locally available, polishing fine aggregate, which had a lower void content than the nonpolishing fine aggregate used for skid resistance in the control decks. The lower average strength of concrete in the control decks was attributed to the higher water demand of the fine aggregate. A nonpolishing fine aggregate with a lower void content than normally required was specified for the overlay concretes, and the resulting average compressive strengths exceeded the design minimum because of lower water-cement ratios. Attaining specified strengths is not always a problem in Virginia. However, a very definite advantage of two-course construction is apparent in the situation just described: The imported, nonpolishing fine aggregate needed to be specified only for the overlay portion of the decks, where skid resistance had to be provided, and the more economical, locally available fine aggregate with poor wearing characteristics could be used in the larger base layers of the decks.

The freeze-thaw resistance of field samples from each concrete was determined with guidance from ASTM C 666 (procedure A), which was modified by adding 2 percent sodium chloride by weight to the water surrounding the specimens. Percentage of weight loss and durability factor are given below for each type of concrete after 300 cycles of freezing and thawing. The surface rating system used is as follows: 0 = no scaling, 1 = very slight scaling [3-mm (0.12-in) maximum depth and no coarse aggregate visible], 2 = slight to moderate scaling, 3 = moderate scaling (coarse aggregate visible), and 4 = severe scaling. The allowable weight loss is 7 percent, and the minimum durability factor is 60.

Concrete	Number of Specimens	Surface Rating	Weight Loss (%)	Durability Factor
Control	8	1.9	2.0	96
Base layer	24	2.2	1.8	96
Latex-modified	8	2.3	2.8	88
PCC	8	3.0	1.5	100
Wire fiber	8	1.9	0.6	99

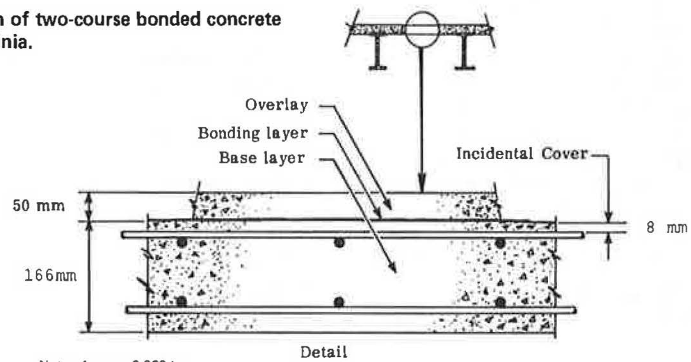
Each of the concretes performed well in this test; this finding ensures desirable freezing and thawing characteristics of the concretes delivered for placement in the decks.

Figure 6. Depth of cover over topmost reinforcing steel for eight study decks.



Note: 1 mm = 0.039 in.

Figure 7. Typical transverse section of two-course bonded concrete bridge decks as constructed in Virginia.



Note: 1 mm = 0.039 in.

PERFORMANCE EVALUATION

The study decks were constructed 12 months before they were opened to traffic, and they were 18 months old before deicing operations were required. An analysis of the condition of the concrete decks at the age of 12 months was made both to assess their quality at that time and to establish a data base for future performance evaluations (3).

Chloride Contents

An important aspect of future determinations of bridge-deck performance will be the degree to which chlorides from deicing chemicals are able to infiltrate the wearing-course concretes. The corrosion of reinforcing steel embedded in concrete depends on several factors including the presence of moisture and oxygen and the pH of the concrete; however, suspected active corrosion has been reported when the chloride ion (Cl^-) content of a concrete reaches a threshold value of approximately 0.77 kg/m^3 (1.3 lb/yd^3) (4). Investigations in Virginia and elsewhere of the specific ion electrode titration method of determining chloride content have shown it to be reproducible and satisfactory (5), and it was used to analyze the concretes included in this study.

Chloride analyses were made of hardened concrete from cylinders cast at the project and from samples drilled from the decks. The results of the analysis of samples for each type of concrete are given below (1 kg/m^3 = 1.68 lb/yd^3):

	Chloride Content (kg/m^3)	
Concrete	Cylinder Samples	Deck Samples
Control	0.48	0.46
Base layer	0.59	—
Latex-modified	0.35	0.42
PCC	0.37	0.44
Wire fiber	0.42	0.53

Clearly, the two sampling techniques caused no significant difference in the test results.

The chloride contents of the uncontaminated concretes were nearly equal in magnitude to what was considered to be the chloride-content corrosion threshold. An investigation was therefore conducted to determine the source of these chlorides (6). Samples of aggregates from several quarries in Virginia were tested and were found to contain different amounts of Cl^-. The results are given below.

Aggregate	Chloride Ion Content as Percentage of Aggregate Weight
Limestone A	0.028
Granite	0.019
Gravel A	0.014
Gravel B	0.012
Limestone B	0.012
Diabase	0.001

The chloride contents of the aggregates used on the project were thus found to be the source of approximately 85 percent of the. chlorides identified in the concretes. These chloride ions are bound within the aggregates, and only minor amounts of the ions (3.75 percent) were found to be leached from the project aggregates when they were soaked in distilled water and the pH was ajusted to 12 to simulate the alkaline environment of concrete. It is therefore assumed that the chlorides in the project concretes will not contribute to the corrosion process. However, because these chloride ions are measured by the titration method used in the analysis, they should be accounted for as baseline chloride contents in the test results. The theoretical corrosion threshold for the study decks will accordingly be 1.19 kg/m^3 (2 lb/yd^3) for the chloride content measured at the level of the upper reinforcement.

Sonic Pulse Velocities

The travel times for sonic pulses transmitted vertically from the bottom to the top of the study decks were measured and recorded at 44 locations on each deck. Before the testing of the field structures, pulse velocities were measured vertically through single-lift and two-course deck models that had the same thickness and reinforcement as the structures. Tests of the deck models revealed no significant difference in pulse velocities among the three types of two-course slabs nor between the two-course and single-lift slabs. In addition, the location of the reinforcing steel relative to the path of the wave had no noticeable influence on pulse transmission time.

In the results given below of the sonic testing on the bridge decks, the 88 measurements from each pair of decks are presented as average pulse velocities (1 m/s = 3.28 ft/s):

Concrete	Average Pulse Velocity (m/s)
Control	3090
Latex-modified	3860
PCC	3970
Wire fiber	3790

On the basis of published ratings for normal ranges of sonic pulse velocities (7), the relative quality of concrete in the two-course decks appears to be better than that of concrete in the single-lift control decks. The following table gives the rating scale used:

Pulse Velocity (m/s)	Rating
	Excellent
4500	
	Good
3700	
	Questionable
3000	
	Poor
2100	
	Very poor

Future evaluations of the study decks will consider any significant decreases in pulse velocities for individual locations or in the average pulse velocity for each deck because such changes could indicate deterioration of the concrete caused by corrosion-induced spalling or other mechanisms.

Skid Resistance

In Virginia, the safety of highway traffic has for many years been safeguarded by constructing and maintaining pavement and bridge-deck surfaces that provide adequate skid resistance. The minimum value for stopping distance numbers has been determined to be 40 on the basis of a skid-test method that uses an automobile on wet pavement. In recent years the skid trailer has come into use as a safer and more convenient method of evaluating skid resistance. Skid numbers resulting from this test have been correlated with those from the method that used the automobile, and the equivalent numbers are reported as predicted stopping distance numbers (PSDNs).

PSDNs for the travel and passing lanes of the study decks were derived from an average of five measurements in each lane by using the skid trailer in accordance with the procedures of ASTM E 274. In the following table, the average PSDN for all lanes of each study deck is seen to be greater than 60, which is excellent:

Concrete	Average PSDNs
Control	62
Latex-modified	66
PCC	66
Wire fiber	65

The skid resistance of the decks will be evaluated again at appropriate times.

Other Procedures

In addition to the evaluation procedures already discussed, visual surveys and soundings and measurements of electrical potentials were made, and these data indicated no problems in the study decks. All of these procedures will be used to evaluate the decks in the third and fifth years after construction.

CONCLUSIONS

This investigation of two-course bridge-deck construction in Virginia led to the following conclusions:

1. An analysis of construction activities and related labor and cost requirements has shown the two-course construction technique to be a viable alternative to conventional construction when additional protection of the upper reinforcing steel is warranted.
2. Attainment of normal cover depths and an adequate degree of consolidation were verified for the two-course decks.
3. The superior strength of the overlay concretes as compared to that of the conventional concrete on this project underscores the primary purpose of the two-course technique, which is to promote the placement of high-quality protective concretes in the upper cover zone of bridge decks.
4. The true benefits of two-course construction must be determined by future deck-performance evaluation procedures such as visual surveys, soundings, determinations of chloride content, measurements of electrical potentials, sonic evaluations, and skid tests.

ACKNOWLEDGMENTS

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Bonded Concrete Bridge Pavements in Switzerland

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Bonded cement concrete pavements have been successfully constructed on more than 150 long, medium, and short bridges in Switzerland during the past 15 years. Traffic in Switzerland never rolls directly over the bridge structure (the deck) but always over a bridge pavement (overlay) instead. The concrete used in bridge construction is generally not proof against combined frost-thaw-salt action. The abundant use of deicing chemicals in winter increases the risks of damage to structural concrete and steel reinforcement. In addition, today's road traffic (a) subjects carriageways to horizontal shearing forces, (b) demands high standard riding quality of pavements, and (c) results in pavement wear caused by abrasion (pronounced wear in the case of studded tires). The criteria required of bonded concrete bridge pavements in Switzerland are presented. The results of on-site measurements taken on 13 test bridges prove that (a) crack-free bonded pavements perceptibly increase the bending stiffness of bridges and (b) use of bonded concrete pavement allows economies in longitudinal steel reinforcement of 7 percent or more depending on the bending stiffness of the bridge and up to 40 percent in transverse reinforcement if the pavement is fully bonded to the bridge deck. Rules to be followed in construction of such bridge-deck pavements are also given.

The subject of bridge pavement construction in Switzerland is conditioned by three factors: (a) the extraordinary relief of the earth's crust in Switzerland, (b) developments in bridge construction, and (c) significant changes that have occurred in the past 15 years in the realm of keeping roads serviceable (i.e., snow- and ice-free) in winter.

TOPOGRAPHY

The Swiss road construction engineer has to face an exceptionally dynamic relief of the earth's crust. This results in the large number of bridges found in very small geographic areas of the country. Bridge requirements per route kilometer in Switzerland are as follows:

1. In mountainous regions, 0.55 to 1.55 bridges/route·km (2 to 10 percent of route length);
2. In hilly regions, 0.30 to 0.60 bridge/route·km (5 to 28 percent of route length); and
3. In the plains, which are crossed by streams, canals, and lakes, 0.5 to 0.75 bridge/route·km (0.5 to 13 percent of route length).

These characteristics make Switzerland a classic bridge-building country.

DEVELOPMENTS IN BRIDGE CONSTRUCTION

Prestressed concrete, which came into use in Swiss bridge construction around 1960, made it possible to achieve practically crack-free structures. The prestressed concrete system, however, requires high-grade steel reinforcement, which is more vulnerable to the action of corrosive agents.

ROAD SERVICEABILITY IN WINTER

Since about 1965, deicing salts (mainly of the sodium chloride group) have been used to keep Swiss roads snow- and ice-free in winter. No sufficient frost-resistant concrete has yet been achieved in Swiss road construction. Inspections of concrete roads in Switzerland have shown that

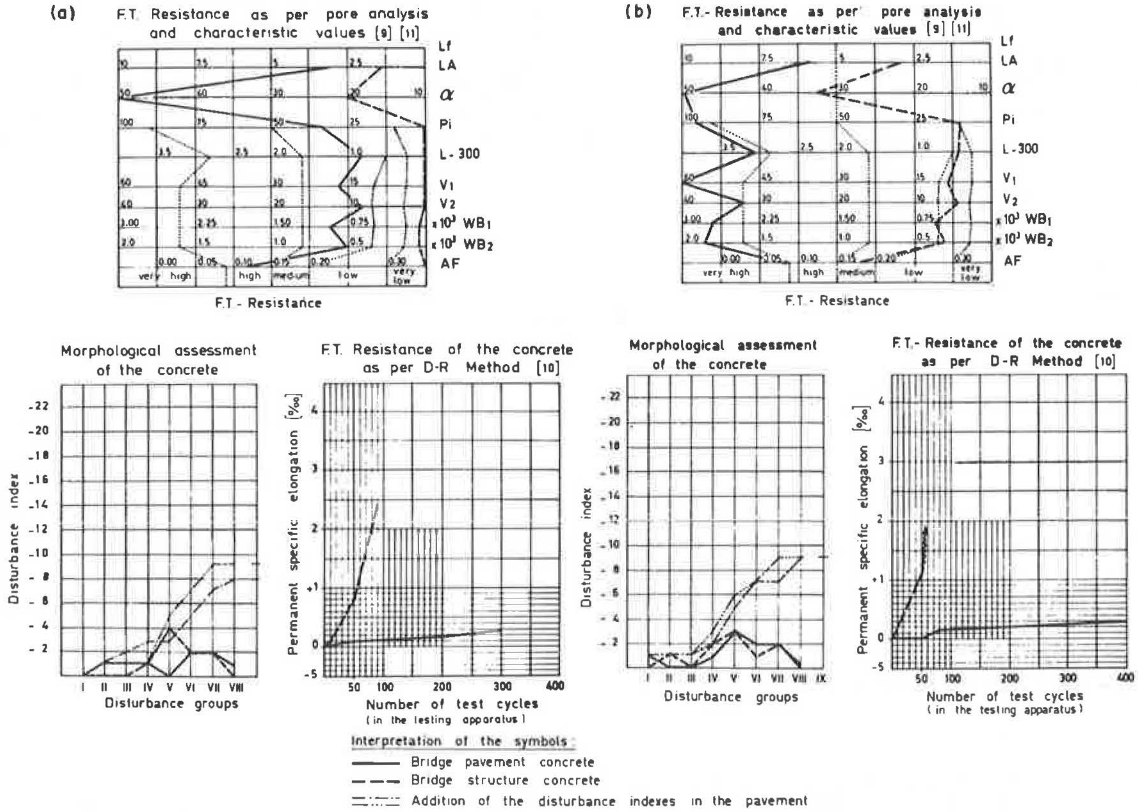
1. No frost damage was found on such roads before deicing salts were frequently used (1960) and
2. Random scaling damages started to appear on these pavements around 1965 as a result of the extensive use of deicing salts.

In relation to concrete durability, therefore, frost-thaw-salt impact is stronger than frost impact. As a result, empirically proved air entrainers have to be added to all cement concretes used in Swiss pavement construction, and compliance is checked by means of stringent quality controls (11).

BRIDGE-DECK-OVERLAY CONSTRUCTION CONCEPTS IN SWITZERLAND

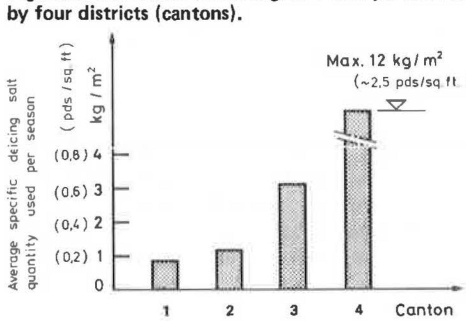
Bridges are not operated unpaved in Switzerland. This seems to be a significant difference between Swiss con-

Figure 1. Pore distribution, morphology, and frost-thaw-salt resistance of concrete for two bridges.



I	Pore agglomerations
II	Compaction pores
III	Pores due to concentration of water
IV	Capillarity of the hardened cement paste
V	Cracks in the cement paste
VI	Cracks in the aggregates
VII	Bond between aggregates and cement paste
VIII	Hydration
IX	Overall disturbance index (the disturbances augment with the number of disturbance indexes and when the disturbance index ⩾ -10, the Dobrolubov-Romer frost-thaw-salt test is advisable).

Figure 2. Quantities of deicing salts used per season by four districts (cantons).



struction concepts and those applied, at least until today, in the United States. The Swiss concept—that bridge traffic should roll over proper pavements—is based on several considerations.

The road-pavement performance demanded by modern traffic requires that particular attention be focused on the horizontal forces caused by vehicle braking and acceleration. To counteract these forces, resistance is required against (a) pavement surface deformation, which mainly occurs in pavements made of thermoplastic materials; and (b) skidding. Other resistant properties required in a pavement are

1. Riding quality (on which heavy demands are made),
2. Imperviousness to climatic (frost-thaw) influences,
3. Resistance to abrasion, and
4. Resistance to corrosion caused by deicing agents.

In the past, the concrete used for bridge construction (including the bridge deck) was non-air-entrained, and its resistance to frost-thaw-salt effects was generally poor. Figure 1 shows pore distribution, morphology, and frost-thaw-salt resistance for two bridges. Figure 2 shows in detail the quantities of deicing salts used.

Because the surface on which the traffic rolls is the only part of the bridge directly exposed to these forces, in Switzerland and throughout Western Europe the bridge structure is protected by means of a specially designed element: the pavement. The major bridge paving systems used are shown in Figure 3. An examination of these systems, which differ in principle from one another, reveals the following facts:

1. The construction of an overlay (bridge-deck pavement) adds weight (permanent load) to the structure.

2. In the stress analysis by the bridge design engineer, all unbonded overlays mean dead weight. Although such systems create a certain load distribution according to their stiffness, they do not contribute to load bearing.

3. In the case of a shear-resistant bond (the cement concrete pavement system shown in Figure 4), the pavement not only adds additional weight to the bridge structure but also considerably increases its transverse and longitudinal stiffness.

Therefore, in any comparative economic study of bridge construction systems, it is necessary to prove to what extent a pavement type contributes not only to the durability of the bridge structure but also to its load-bearing capacity. In this way, possible savings in steel reinforcement can be determined.

Bonded Paving Systems

The two bonded paving systems shown in Figure 4 require placement of the pavement at different stages of construction. The system referred to as monolithic requires placement of the pavement while the deck is still plastic. Although this approach is sound from an engineering viewpoint, it is hampered by (a) the impossibility of slipforming and (b) the difficulty and the cost of conventional placement (by means of a paver moving on rails), which is thus feasible only on short bridges. In the two-layer system (Figure 4), pavement is placed after hardening of the deck concrete. Because of the feasibility and the moderate placement costs of this system, more than 150 bridges of all sizes [none shorter than 300 m (986 ft)] have been paved according to this method in Switzerland during the last 15 years.

Stress

The characteristics of the transverse and longitudinal stresses for the two static bridge-deck conditions to be analyzed—unpaved and paved—are shown in Figures 5 and 6 (1). In the figures, $\sigma(g + V\infty)$ = stresses caused by dead weight and prestressing; $\sigma(g + V\infty + p)$ = stresses caused by dead weight, prestressing, and traffic load; and d_B = concrete pavement thickness. The paved and unpaved conditions differ from one another in the levels of S_1 and S_2 (safety factors) of the neutral axes in the cross section, shown in Figure 6. Figure 7 shows moment of inertia with and without concrete pavement in the Felsegg Bridge over the Thur River (12).

As a consequence of the shear-resistant bond between the pavement and the bridge deck, deformation of the bridge deck causes stresses in the pavement (Figures 5 and 6), a factor that must be considered in designing an overlay.

CRITERIA FOR TWO-LAYER PAVING SYSTEMS

The suitability of the two-layer paving system shown in Figure 4 is determined by whether (a) there is sufficient permanent bond between the pavement and the bridge deck and (b) the pavement offers sufficient protection to the structural reinforcement against the corrosive action of deicing agents.

Fatigue Tests

Fatigue tests have been conducted at the Swiss Federal Materials Testing Institute (EMPA) (2, 3) for the purpose of determining

1. The efficiency of the bond between the bridge deck and the pavement under repeated loadings and
2. The ultimate strength of the bond.

These tests complement on-site load tests conducted on the St. Margrethen twin bridge (4). The aim of the EMPA tests was to clarify the behavior of the combined system—the pavement and the bridge deck on which it has been placed—under any number of traffic load cycles.

Models

Figure 8 shows the two mock bridge-deck slabs that, with their respective pavements, were constructed in the laboratory. Each was as thick as a normal, paved bridge deck. The pavement of slab 1 was designed without considering any load-bearing participation. It was only meant to function as a pavement and, in the structural design, its weight was considered as a permanent dead load. Slab 2, on the other hand, was designed as a genuinely combined system (bridge deck with bonded cement concrete pavement). The concrete pavement was considered not only a dead weight but also an agent cooperating with the bridge deck to bear traffic loadings. For this purpose, steel reinforcement was designed for the pavement with additional reinforcement over the midcolumn to resist the negative bending moment.

Procedure

In selecting the disposition and the size of the test loads, care was taken to approximate as closely as possible the conditions of the in situ bridge structures. A strip load of 19 Mg (21 tons) was applied to each slab at a cyclical frequency of 4 Hz (equivalent to approximately 250 cycles/min). Stronger loading demands than would occur in reality were intentionally made on the test slabs for greater certainty in the results.

Results

Under the stresses caused by 2 million load cycles, the bond remained unaffected. One of the two test slabs was also submitted to another 2 million loading cycles (the total load of which was 125 percent of the original load) and then to a further 2 million cycles (the total load of which amounted to 150 percent of the original load). Even after the last test, the bond did not appear to be weakened.

Tests to determine the ultimate strength of both slabs yielded the following factors of safety (S): $S_1 = 5.2$ (slab 1) and $S_2 = 4.0$ (slab 2). The safety factor can be defined as follows:

$$S = (P_u + P_p)/(N + P_p) \quad (1)$$

where

P_u = ultimate load,
P_p = equivalent load expressing the influence the permanent load of each whole slab exerts on the respective N location, and
N = traffic load [19 Mg (21 tons)].

Shearing Stresses in the Bonding Zone

In a load-bearing construction system consisting of two bonded, load-bearing elements that resist shearing strengths, it is important to determine the shearing stresses in the contact zone. If interdependent dilatation exists by definition in either one of the bonded elements, stresses caused by shearing forces cannot be measured but can only be assessed in theory. This also applies to residual stresses caused by shrinkage with and without creep effects.

Maximum shearing stresses (τ) of 2.95 and 2.72 MPa (428 and 395 lbf/in^2) respectively were assessed in the two test slabs after cracking had occurred, under carrying load and dead weight, by determining the course of the transverse strength according to the elastic or plastic method (3). Data produced by other authors on cracking caused by shearing force (τ rupture) in bonded

Figure 3. Three major bridge-paving systems used in Switzerland.

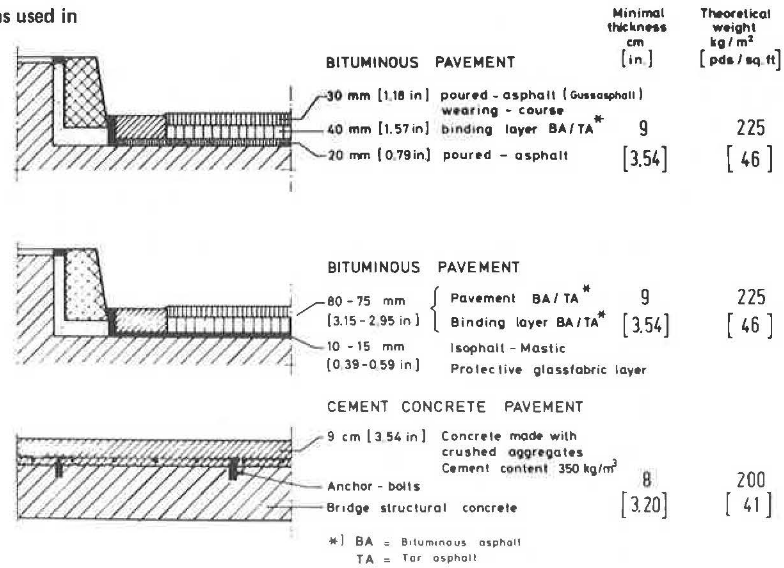


Figure 4. Two systems of bonded concrete pavement construction.

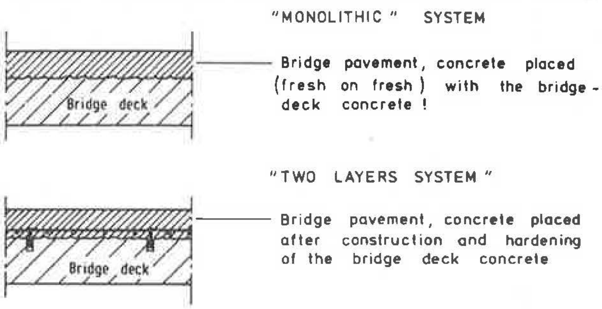


Figure 5. Characteristics of transverse stresses.

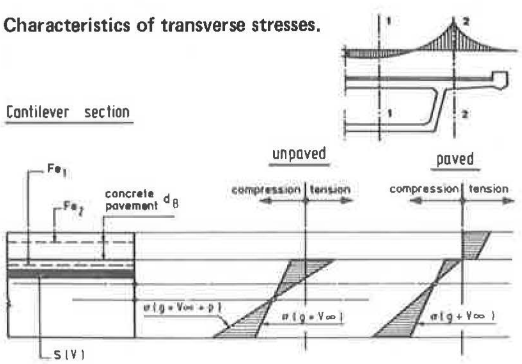


Figure 6. Characteristics of longitudinal stresses.

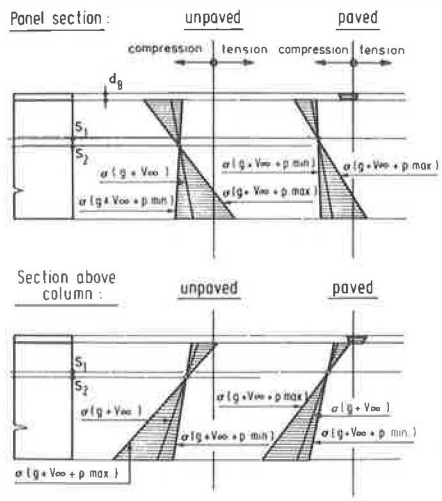
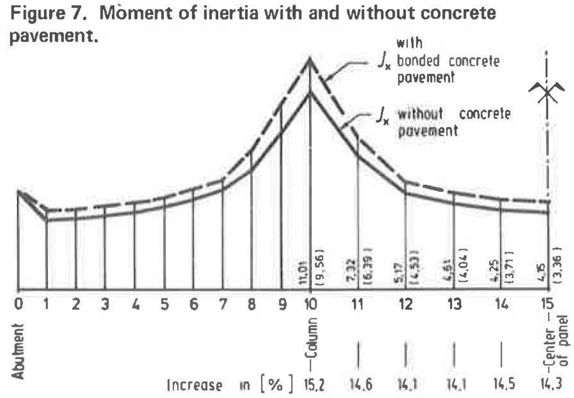


Figure 7. Moment of inertia with and without concrete pavement.



systems with cracked cross sections contradict these findings. The table below gives test results reported by various other sources for the shearing strength of bonded concrete elements of different ages (1 MPa = 145 lbf/in^2):

Source	Shear Strength (MPa)
ACI-ASCE (4) (smooth to rough bonding surface)	0.55 to 2.21
Hanson (5) (smooth to rough bonding surface)	2.07 to 2.45
Saemann and Washa (6)	2.45
Basler and Witta (7)	2.45

To assess the actual bonding strength in bridge structures, values obtained from uniaxial strength measurements have been published by Wilk (8). These values were obtained from EMPA tests on 7.2-cm (2.8-in) diameter cones. The tests were made to establish the uniaxial tensile strength of the bond when it is not influenced by anchors. The results are shown in Figure 9. Figures 10 through 13 show some interfaces of fractures that resulted from the same tests.

Corrosion Tests

Except for some light damage caused by scaling, no other damage was found on the bridge pavements constructed according to the two-course bonded construction system (Figure 4). These results show that efficient protection is provided by the two-course system against the corrosive action of deicing agents.

Quantitative data on the sealing efficiency of concrete pavements were obtained by taking drilled-out test cores, each of a diameter ϕ = 7.2 cm (2.83 in), from eight bridges (Table 1) at the end of the winter of 1974 and measuring their specific chloride contents at various depths. The range of the values measured is shown by the hatched zone in Figure 14, which indicates that the highest chloride content measured in the contact zone does not exceed 0.025 percent by weight—the limit set by German Industrial Standard Specifications (DIN) No. 1045 for the chloride content in prestressed concrete. This is of primary importance in view of the fact that, immediately underneath the pavements, bridge decks in Switzerland are generally constructed with non-air-entrained concrete and in most cases, therefore, are without frost-thaw-salt resistance. This is proved by the results of many microscopic pore analyses, morphological examinations of concrete struc-

Figure 8. Disposition of loads on two test slabs.

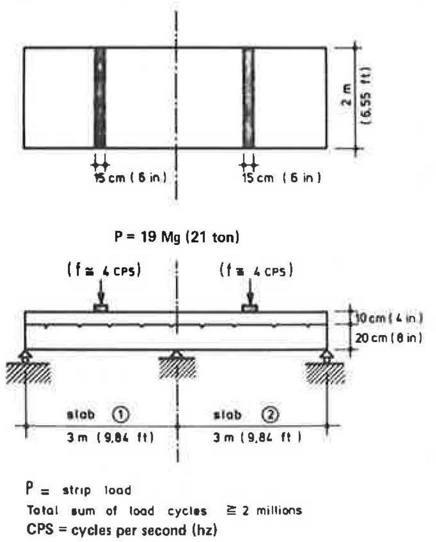


Figure 9. Distribution of uniaxial tensile strength (β_z).

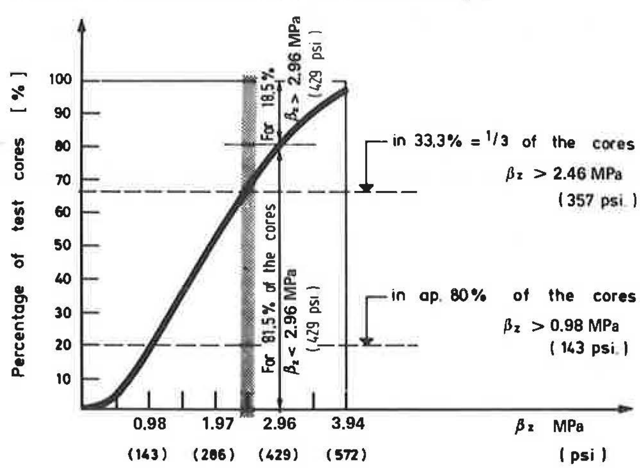


Figure 10. Results of uniaxial tensile strength tests: rupture running across whole mortar course [β_z = 0.69 MPa (100 lbf/in^2)].

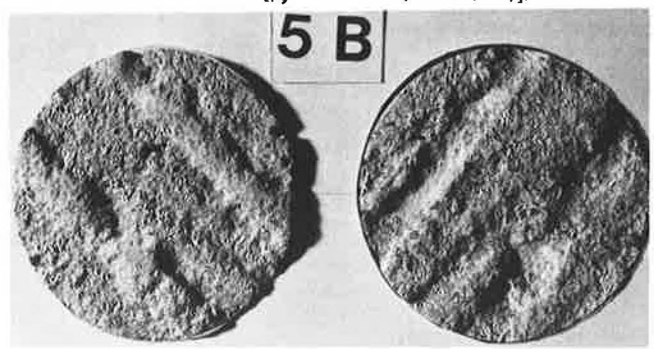


Figure 11. Results of uniaxial tensile strength tests: rupture running across whole mortar course [β_z = 1.08 MPa (157 lbf/in^2)].

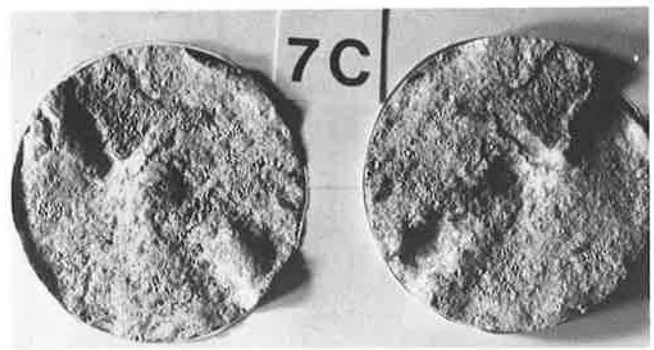


Figure 12. Results of uniaxial tensile strength tests: rupture running across many coarse and fine aggregates with no fine mortar in contact zone [β_z = 2.38 MPa (345 lbf/in^2)].

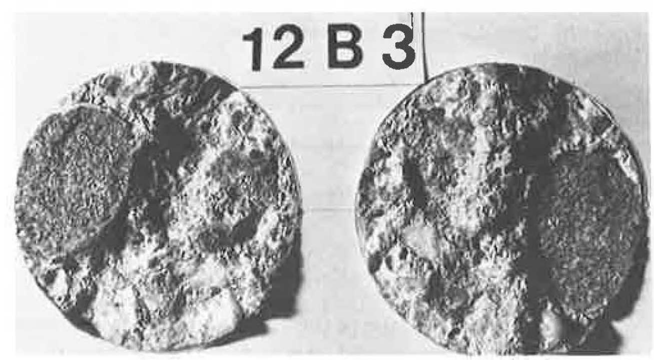


Figure 13. Results of uniaxial tensile strength tests: characteristic surface of rupture [β_z = 3.04 MPa (441 lbf/in^2)].

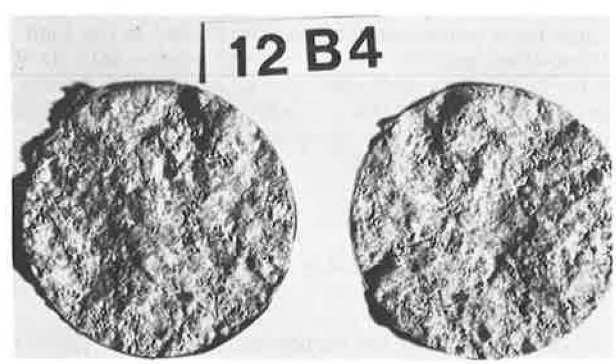


Table 1. Location and construction data for bonded cement concrete bridge pavements on which corrosion tests were conducted.

Bridge or Viaduct	Construction Period	Length of Bridge (m)	Pavement Thickness (cm)	Spacing of Transverse Joints (m)	Age of Pavement at First Salt Treatment (years)
Rheinbrücke N13 Grisons, St. Gall	1962	210	8 to 10	5	1
Zwillingsbrücke N13, St. Gall	1962 to 1964	125	8 to 10	5 to 6	1
Lehnenviadukt N13, St. Gall	1962 to 1964	800	8 to 10	5 to 6	1
RA No. 20, Pont de Larrevoin	1964 to 1965	212	8 to 10	8	
Kanalbrücke N1, Aargau	1966 to 1967	110	8 to 10	5 to 6	1
Goldachviadukt N1, St. Gall	1971 to 1973	480	8 to 10	Day joints only	1
Nónnentobelbrücke T13, St. Gall	1972 to 1974	140	8 to 10	No joints	1
Attinghausenviadukt N2, Uri	1973	160	8 to 10	No joints	

Note: 1 m = 3.3 ft; 1 cm = 0.39 in.

Figure 14. Chloride content limits measured in concrete pavements of eight bridges.

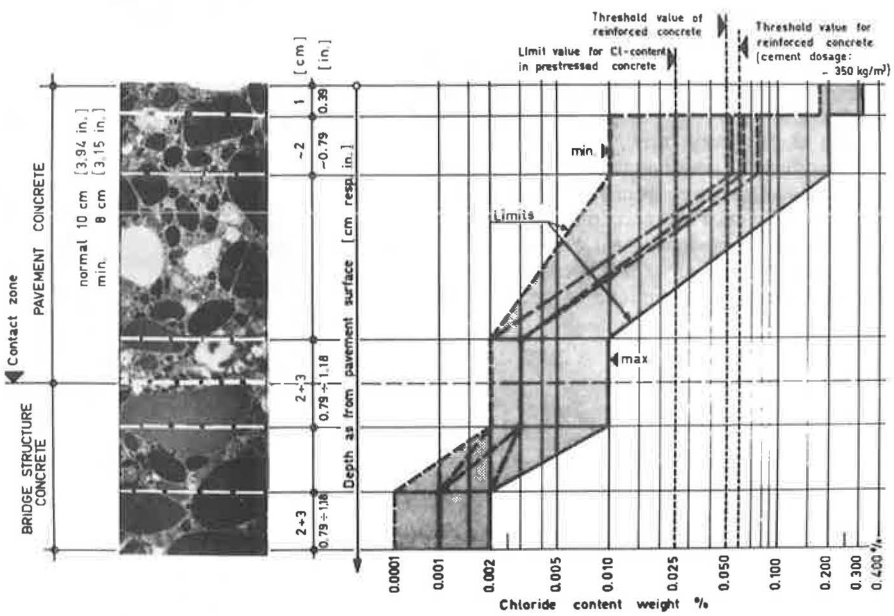
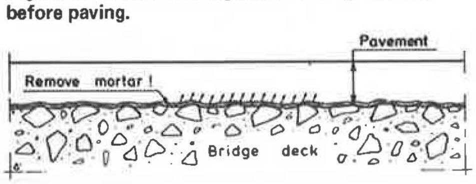


Figure 15. Desirable roughness of bridge surface before paving.



ture (i.e., of the quality of the bond between aggregates and matrix and cracks and capillarity), and tests of frost-thaw-salt resistance using rapid cycles according to the Dobrolubov-Romer (D-R) method (Figures 1 and 2). These three types of tests and their results are discussed elsewhere (11).

In morphological analysis, the higher the total index number for a certain structure, the higher is the loss in frost-thaw-salt resistance. In the rapid-cycle (D-R) test for frost-thaw-salt resistance, concrete is frost-thaw-salt resistant if the permanent longitudinal dilatation of its prismatic test specimen reaches 1 percent after 360 or more cycles.

CONSTRUCTION RULES

Rules for constructing bonded bridge-deck pavement are as follows:

1. The surface of the structural concrete of the bridge deck must be absolutely sound, show a high degree of roughness (Figure 15), be scrupulously clean, and be kept wet for 48 h before pavement construction.
2. The deck must not be vibrated during paving (for instance, by heavy vehicle traffic) because concrete passes through a very critical phase during the first 4 to 12 h after placement. At this stage, deformation capacity is at its absolute minimum (Figure 16), and any disturbance causing deflections stronger than those allowed can result in irreversible fractures.
3. Immediately before the concrete pavement is placed, the bridge-deck surface is treated with a synthetic resin emulsion (on a polyvinyl or latex base) to decrease the elasticity modules below their normal level in the contact zone and thus reduce the stresses caused by differential shrinkage (16). Mortar left after the removal of the coarse aggregates from the pavement concrete is then rubbed into the bridge deck surface by means of brooms. The thinner the cement paste coating is, the better the bond will be (Figures 9 through 12).
4. To counteract the effect of the shearing stresses caused by shrinkage and creep, the pavement panels are fastened to the bridge by means of 2 to 3-m (6.5 to 9.8-ft) anchors placed along their edges (Figures 3, 4, and 17).
5. The pavement thickness must not be less than 8 cm (3.15 in).
6. In zones of negative flexural moments, the pavement reinforcement must be increased according to careful calculations.
7. Adequate bridge pavement curing is of primary

Figure 16. Capacity of strain at failure of freshly placed concrete.

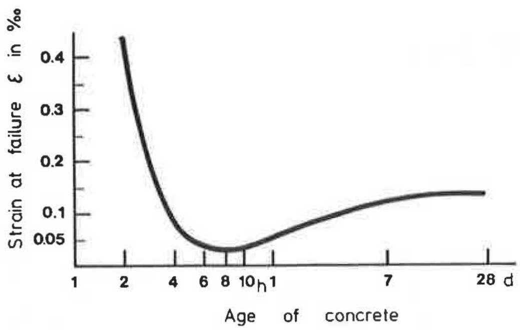
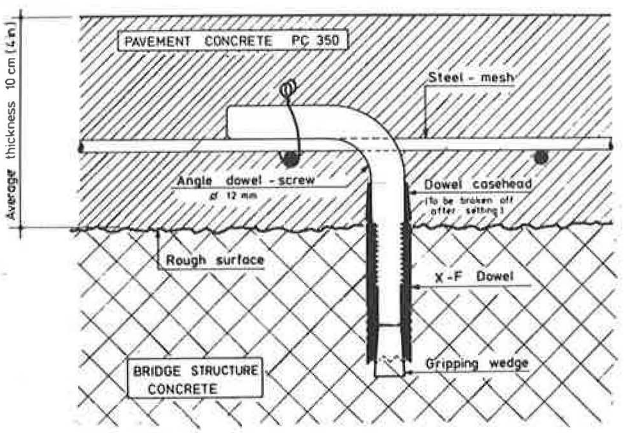


Figure 17. Type of anchor used to fasten pavement panels.



importance. In addition to the application of a curing compound, it is advisable to protect the concrete for at least 48 h after placing by means of wet burlap.

CONCLUSIONS

Fatigue tests confirmed the results of previous on-site measurements (12, 13, 14), showing that, in the case of a two-course bonded paving system subject to the maximum cyclical static loading allowed, the pavement will always perform its share of the load-bearing function of the structure—provided, of course, that the technical rules (4) have been observed during construction. This joint load bearing by the bridge and the pavement must therefore be fully considered in structural design.

Bridges must be designed with full knowledge of all the advantages offered by a bonded pavement. In its turn, the pavement must be designed according to its load-bearing contribution to the bridge. Because of its load-bearing capacity, a bonded pavement offers cost-saving possibilities that cannot be offered by pavements constructed according to other design systems (14). In fact, bridge construction costs, which of course vary according to the project design and the longitudinal and transverse stiffnesses required (Figure 7), could at times be reduced by an amount equivalent to the paving costs.

Protection of the bridge structure by a frost-thaw-salt-resistant pavement of the best possible impermeability is needed. Measured chloride contents and penetration depths (8) indicate that bonded concrete pavements offer effective and durable protection against frost-thaw-salt penetration without the use of special insulation layers.

The long-term behavior of the bond in certain bridge structures can be controlled by means of ultrasonic equipment (15).

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Correlation of Laboratory Cutting Data With Tunnel-Boring Machine Performance

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Results of rock-cutting experiments conducted under constant normal force and constant penetration are discussed. Independent as well as interacting groove-spacing regions are identified and classified through determination of optimum and critical spacings. Specific energy, muck weight, groove depth, cutting coefficient, and size distribution of muck for the independent and interacting grooves of laboratory cutting are examined, and forward thrust, torque, advance rate, and muck size distribution in field operations are presented and discussed. Satisfactory correlation is obtained between laboratory cutting results and field performance data.

Although significant developments in underground excavation have occurred in the past two decades (3), the state of the art is not keeping pace today with developments in other related fields. Improved methods of underground excavation are highly desirable for developing new mineral reserves and opening up underground avenues of transportation for men and materials.

The continuing search for improved tunneling methods usually includes evaluation in the field of each new innovation or development. The major drawbacks of this approach include excessive cost and the duplication of time and effort that results from setting up a system to operate under varying values of controlling parameters. Only limited progress has been made by using this approach.

Less costly and quicker results can be achieved by assessing innovative equipment designs in the laboratory under instrumentation. Series of tests can be repeated until conclusive evidence of the merits of a specific design is established. Testing new tunneling techniques in the laboratory can make possible significant progress toward the development of technology for rapid and efficient underground excavation. The observations and recommendations presented in this paper are aimed at bringing that goal closer.

EXPERIMENTAL PROCEDURES AND EQUIPMENT

Laboratory rock-cutting experiments can be performed in either of two modes: constant penetration or constant normal force. In constant penetration, the cutter is fixed in its position relative to the rock surface as it traverses the surface. Thus, the penetration or groove depth is preset at the beginning of the test to, say, 1 mm (0.04 in), and normal force and cutting force—and possibly the groove width resulting from such penetration—are measured and recorded. In the constant normal force mode, the cutter traverses the face of the rock as it is being forced against the surface by a hydraulic cylinder that produces a nearly constant normal force. In these tests, normal force is preset during the test, and groove depth, groove width, and cutting force are recorded or measured as a function of normal force.

Laboratory rock-cutting data were produced at the Colorado School of Mines (CSM) by using a modified milling machine equipped with a 15-cm (6-in) disk cutter with a constant penetration ranging from 0.75 to 3 mm (0.03 to 0.12 in) that traversed the rock specimen at a speed of 8.2 mm/s (0.33 in/s). Laboratory rock-cutting data were produced at the Twin Cities Mining Research Center (TCMRC) by using a specially designed constant-thrust machine equipped with a 17.5-cm (7-in) cutter traversing under a normal thrust of 31 kN (7000 lbf) and at a speed of 7.5 cm/s (3 in/s).

The CSM experiments were performed on cores 15 cm (6 in) in diameter, obtained from three tunnel sites: the Nast tunnel in Basalt, Colorado; the Lawrence Avenue tunnel in Chicago; and the Climax tunnel in Climax, Colorado. Table 1 shows some of the geologic and strength data for these rocks. The TCMRC experiments were performed primarily on blocks obtained from commercial quarries marketing marble, limestone, granite, and quartzite. The specimens measured approximately 61 by 61 by 20 cm (24 by 24 by 8 in).

In the laboratory rock-cutting experiments, the normal and horizontal forces applied to the cutter shaft were determined by sensing the outputs of two properly aligned strain-gauge systems. Normal force and cutting force were then computed from the strain-gauge output recordings. The cutting force recorded by the strip-chart recorder is the force applied at the cutter shaft (Figure 1). This value is sufficient for comparing data obtained by the same machine. If results obtained by two laboratory cutting machines are to be compared, then the cutter diameter and the loading geometry will affect the results in such a way that direct comparisons are not possible unless proper corrections are made. For the purpose of consistency, in this paper the force applied to the rock surface is used to calculate the values of specific energy and cutting coefficient. The force at the rock surface, or the rock-resistance force, is determined by multiplying the force measured at the shaft by the ratio of the lever arms for the shaft and the cutter tip. The cutting coefficient was obtained by dividing corrected cutting force by normal force. The total work done by the corrected cutting force was divided by the weight of the total muck to obtain the value of specific energy for that groove. Data on groove geometry and muck size distribution were also determined for each test. Figure 2 shows the appearance of grooves in limestone.

Field data collected on the operation of boring machines (1, 7) included total forward thrust, cutterhead torque, and muck size distribution.

RESULTS

The performance of a rolling rock-cutting element in a specific rock generally depends on normal force, groove depth, cutter diameter, included angle of disk, cutting speed, groove spacing, and cutter sharpness (2, 4, 5). All of these parameters are interrelated so that if one parameter is increased the cutting performance may be kept consistent by decreasing another appropriate parameter. In some cases, two or three parameters can be changed systematically without affecting the overall cutting performance. Rad and McGarry (4) and Rad and

Table 1. Properties of rock specimens used in experiments of Colorado School of Mines.

Rock	Geologic Description	Compressive Strength (MPa)	Apparent Density	Shore Hardness
Tennessee marble[a]	Holston limestone	71.7	2.69	49.5
Valders limestone[a]	Cordell dolomite (member Manistique)	108.2	2.55	56.2
Gray granite[a]	St. Cloud gray, Granodiorite gneiss	183.4	2.72	84.4
Barre granite[b]	Barre granite	220	2.64	102
Sioux quartzite[a]	Sioux quartzite	560	2.64	89.1
Nast tunnel[c]	Fine-grained granite	89.7 to 241.3	2.42 to 2.66	—
Lawrence Avenue tunnel[c]	Fine-grained limestone	55.2	2.81	46

Note: 1 MPa = 145 lb/in^2.

[a] From Rad and Olson (6). [b] From Rad and McGarry (5). [c] From Haller, Pattison, and Shimizer (1).

Figure 1. Instrumentation for linear cutter apparatus.

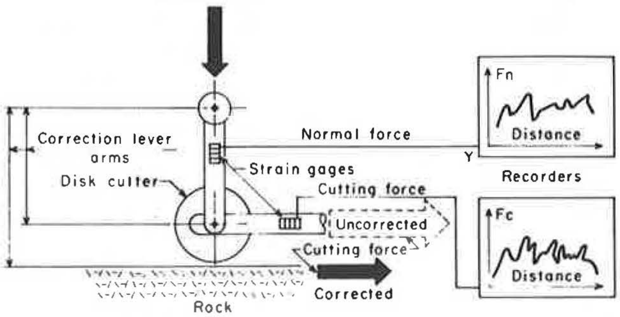
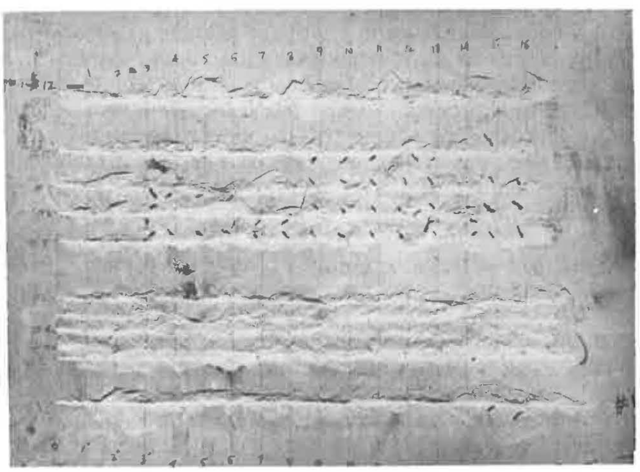


Figure 2. Appearance of grooves in limestone.



Olson (5) have shown that, for the purpose of studying specific aspects of the cutting process, several of these parameters can be systematically reduced from their field value without noticeably affecting results such as values of specific energy, cutting coefficient, and muck size distribution.

One of the more important characteristics of cutter design is the circumstances under which the cutter operates optimally and those under which it does not. Rad and Olson (5) conducted a study of cutting performance as a function of spacing and identified the three basic performance regions. The results indicated that increasing the spacing is analogous to increasing the speed, diameter, and included angle of the cutter and to decreasing normal thrust and groove depth. Such findings indicate that the concepts discussed here are applicable to laboratory and field studies regardless of what parameters are changed to obtain the various performance levels.

Performance Regions

For a fixed cutter diameter and fixed normal force, there exists a critical spacing for parallel grooves. Critical spacing is the closest distance two neighboring grooves can be to each other without interacting (Figure 3). Stated another way, critical spacing is the largest spacing at which there is interaction between neighboring grooves. At spacings slightly greater than critical spacing the grooves become independent, and at spacings slightly smaller than critical spacing there is chipping between adjacent grooves. The chips, formed by breakout between grooves, are considerably larger than those produced by independent grooves. In the interacting regions, the amount of muck is greater than that of independent regions and the value of specific boring energy is less than that of independent specific energy, as shown in Figure 4. By definition, specific energy and muck size at critical spacing are the same as those for all independent grooves with greater spacings.

For each set of cutting conditions, there is an optimum spacing at which the highest amount of muck and the lowest value of specific energy are obtained (Figure 5). Any decrease or increase in groove spacing from the optimum spacing will result in a reduction in cutting efficiency.

As the groove spacing decreases below the critical spacing, the average chip size increases from that produced at independent spacings (6). However, if the spacing is less than optimum, the maximum chip size is controlled by the groove spacing and is thus smaller for smaller spacings (4, 6). Identification of the preoptimum region was facilitated by arbitrarily calling the smallest spacing used in each series minute spacing and reporting the values of such factors as muck specific energy for this spacing.

Although a cutter operating in the preoptimum range is subjected to relatively high lateral forces, it is occasionally desirable to design a machine so that the cutters operate in the preoptimum region. In such a design, if cutting conditions worsen (normal thrust drops, the cutter tip wears, or there is a transition into harder rock), the cutter will gradually move into the postoptimum cutting region, which is very efficient. If, however, the cutter is initially operating in the postoptimum region, it cannot continue to operate satisfactorily because such a worsening of cutting conditions will move the cutter from the postoptimum region very close to or into the independent region. It becomes apparent, therefore, that cutters on boring machines intended for use on uniform rock type should be designed to operate at the optimum point or in the postoptimum region and machines intended for use on variable rock type should be designed to operate in the preoptimum region in softer formations such as marble and shale and in the postoptimum region in harder formations such as granite and quartzite.

Figure 3. Performance regions for gray granite defined by muck weight.

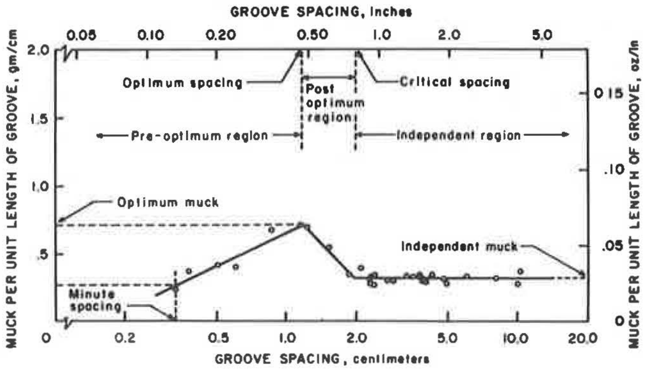


Figure 4. Performance regions for gray granite defined by specific boring energy.

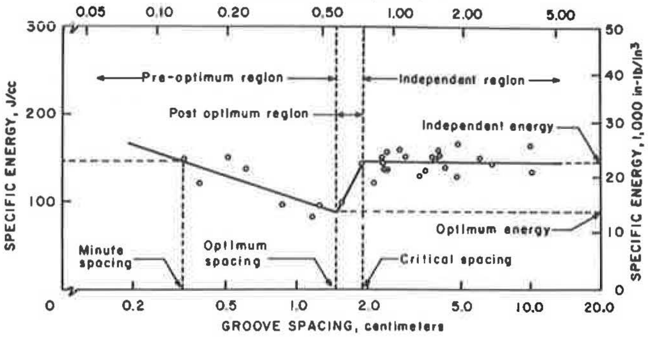
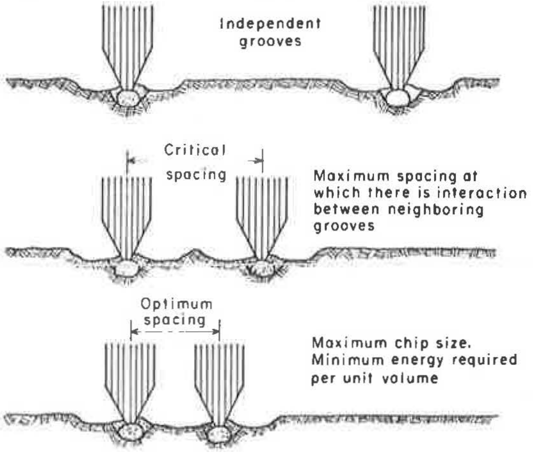


Figure 5. Optimum and critical groove spacings.



Normal Thrust

Experiments (2, 4) have shown that increasing the thrust or decreasing the cutter diameter results in deeper grooves and larger chips (Figure 5). This supports the theory (4, 5, 6) that critical and optimum spacings increase with normal force. As shown in Figure 6, the amount of muck removed for each groove increases smoothly but in a nonlinear pattern throughout the range of values investigated in these experiments. Coarser material was obtained at higher normal forces although independent specific energy did not vary substantially with increasing thrust (2).

Steady State

In actual field conditions, the boring-machine cutters repeatedly traverse the same circular path and keep breaking chips away from the roughened advance face. At the steady state thus reached by the boring-machine cutters, two factors contribute to the muck-removal process: (a) The cutters continue to penetrate farther into the groove on each pass by removing additional muck from the groove, and (b) neighboring grooves almost always interact with each other, which causes large chips to be formed.

Results obtained by Rad and McGarry (4) and Rad and Olson (5, 6) have shown that the interaction between the two neighboring grooves is the major contributing factor. Repeated passes in independent grooves resulted in successively smaller amounts of muck (4). Repeated passes in interacting grooves become more efficient until they reach a steady state corresponding to the cutting conditions (normal force, cutter diameter, and so on).

Figure 7 shows values for specific energy obtained from multiple passes on Barre granite. A series of 15 parallel, equidistant grooves were made on the smooth surface of the specimen, one groove at a time; this constituted one pass. Subsequent passes were made by repeating this process. The value of specific energy decreased on the second and third passes and reached the steady state on the sixth pass. The muck weights for these passes are shown in Figure 8. Although the scatter in the muck data is quite significant, it can be seen that muck weight also reaches the steady state at the sixth pass. The optimum spacing at the steady state is 1.5 times that found for smooth surface grooves (4). At the steady-state optimum spacing, the amount of muck increased 20 percent and the value of specific energy decreased to 50 percent of that found for the optimum cutting conditions for the smooth surface grooves (4).

Another interesting observation was that it takes a smaller number of passes to reach the steady state in postoptimum and preoptimum regions than in cases where the spacing is near critical (5). In other words, the number of passes required to reach the steady state was found to be somewhat proportional to groove spacing.

CORRELATION OF LABORATORY DATA WITH FIELD PERFORMANCE

The values of specific energy and advance rate obtained for field operations were compared with their corresponding laboratory parameters as functions of cutting coefficient and advance rate. A comparison of muck size distribution was also conducted for field and laboratory data.

Specific Energy

The value of specific energy for laboratory experiments is obtained by determining the work done by the rock-resistance force at the cutter tip and then dividing it by the volume of material removed. The cutting-coefficient value is the ratio of rock-resistance force to normal force, as shown in Figure 1. Essentially the same principle is applied in determining specific energy and cutting coefficient for field operations: (a) The value of specific energy is the amount of work done by the cutterhead to remove the unit volume of muck, and (b) the cutting-coefficient value is the cutting-coefficient average for all cutters.

The value of specific energy for a boring machine is obtained by using the following equation (Equations 1 and 2 were formulated in customary units, and thus no SI equivalents are given in the term definitions):

$$E_s = 2NT/0.60\,R \tag{1}$$

where

E_s = specific energy (in·lbf/in^3),
N = speed of rotation of cutterhead (rpm),
T = torque required by the cutterhead (lbf·ft), and
R = advance rate (ft/h).

The cutting coefficient is obtained from the boring-machine parameters by using the following equation (7):

$$C = (4/D) \cdot (T/F) \tag{2}$$

where

C = cutting coefficient,
D = tunnel diameter (ft), and
F = total thrust (lbf).

Figure 9 shows the variation of specific energy with cutting coefficient for single-pass grooves at the optimum point or at the critical or minute spacings (5) and for steady-state grooves (7). The data points for a single pass represent a large set of data: 23 grooves in marble, 25 grooves in limestone, 30 grooves in granite, and 27 grooves in quartzite. The data for the steady-state mode were obtained from experiments on eight specimens from the Nast, Lawrence, and Climax tunnels. As a first approximation, the behavior in the steady-state mode can be approximated by a straight line. The behavior in the preoptimum and postoptimum regions of the single-pass experiments can be approximated by lines drawn between the conditions of critical and optimum spacing and optimum and minute spacing respectively. The behavior in the independent region is represented by the critical point because by definition the cutting characteristics of the independent grooves are the same as those of the critical groove. These approximations are confirmed by the behavior of individual data points in the interacting regions, which have been left out for the sake of clarity.

Critical specific energy, optimum specific energy, and minute-spacing specific energy decrease as the cutting coefficient increases. The single-pass data further show that, if experiments are performed in one rock type, specific energy decreases as the cutting coefficient increases while conditions change from critical to optimum. If, however, the cutting conditions continue to change in the same direction, no further increase in the cutting coefficient is realized and it begins to decrease as specific energy increases. Because the preoptimum and postoptimum segments of these variations both follow an inverse linear relation, the trend in the variation of specific energy with cutting coefficient may not be used to indicate whether the cutting conditions are preoptimum or postoptimum unless the data cover a good portion of both these regions.

Figure 10 shows the variation of specific energy with cutting coefficient for the Nast tunnel operation. Figures 8 and 9 show very close agreement between field data and laboratory data: Both show an inverse linear trend. It is interesting that the change of cutting conditions in the field was obtained by varying the forward thrust. The change in cutting-coefficient values in the field reflects an increase in the value of forward thrust rather than a decrease in the value of torque. The data points shown in Figure 7 represent torque values of 75 to 84 kN·m (102 000 to 114 000 lbf·ft), whereas the value of thrust varies from 1.78 to 2.32 MN (401 000 to 522 000 lbf).

Figure 11 shows the variation of specific energy with muck per unit length of groove for single-pass experiments. The data points represent the results obtained from over 100 grooves. For each of the three cutting modes—critical, optimum, and minute—the values of specific energy decrease as the amount of muck per unit length of groove increases. For the same rock, as muck increases from the critical to the optimum value, specific energy decreases. In the preoptimum region, however, the muck decreases and the specific energy increases. For each rock, the optimum condition is the one where the highest amount of muck is produced at the lowest value of specific energy. Because the preoptimum and postoptimum segments of these variations both follow an inverse linear relation, the trend in the variation of specific energy with muck weight may not be used to identify whether cutting conditions are preoptimum or postoptimum unless the data cover both of these performance regions.

Figure 6. Groove depth versus normal thrust for independent grooves.

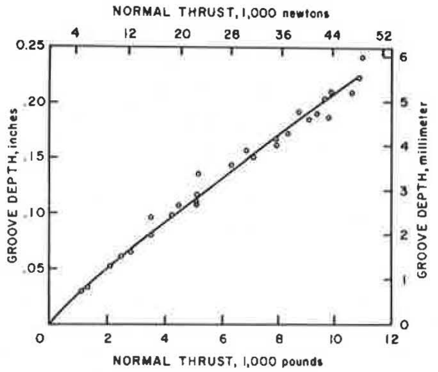


Figure 7. Steady-state specific energy.

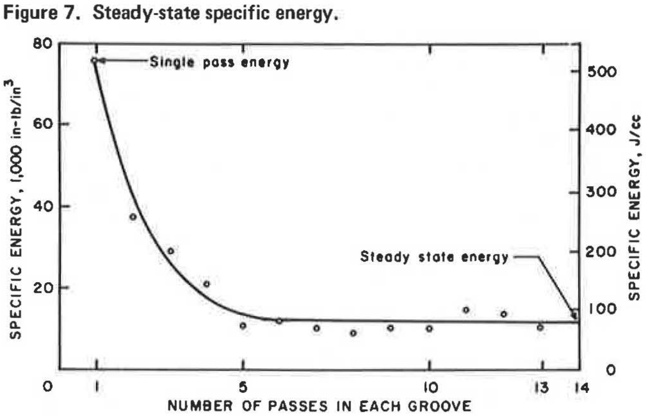


Figure 8. Steady-state muck generation.

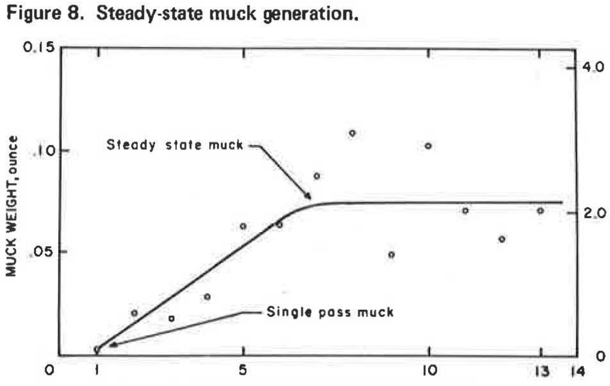


Figure 9. Specific energy versus cutting coefficient for laboratory experiments.

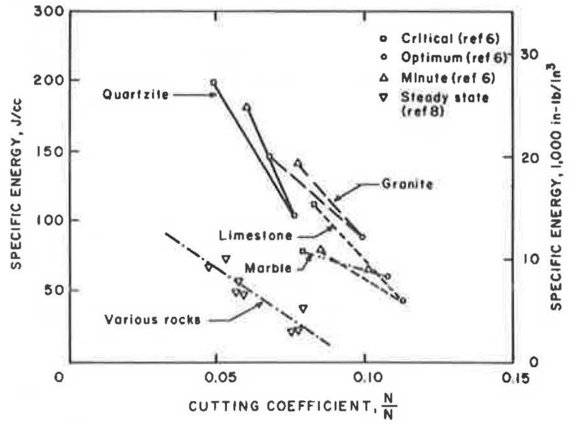


Figure 10. Specific energy versus cutting coefficient for Nast Tunnel operation.

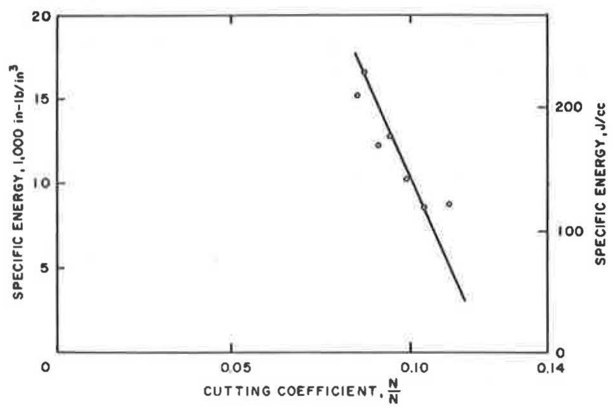


Figure 12 shows the variation of specific energy with groove depth for critical, optimum, and minute groove spacing and steady-state grooves. For each of these four cutting modes, the value of specific energy decreases as groove depth increases. The single-pass data further show that, if experiments are performed in one rock type, specific energy decreases as groove depth increases from that of independent spacing to that of optimum spacing. Specific energy increases if groove depth continues to increase in the preoptimum region. The variation of specific energy with groove depth can be used effectively to identify the mode of cutting in a given rock. Such identification is relatively simple because, for each rock, the slopes for preoptimum and postoptimum regions are positive and negative respectively.

The two major factors in muck removal in a boring-machine operation are cutter penetration and rock breakout between cutters. It can therefore be seen that advance rate in field operations corresponds with some combination of laboratory values for groove depth and muck weight. Figure 13 shows specific energy as a function of boring rate for the Nast and Lawrence Avenue tunnel operations. The material in the Lawrence Avenue tunnel is relatively uniform and—judging by values of specific energy, advance rate, and muck size distribution—the boring machine is operating in the postoptimum region. The data thus correlate well with the postoptimum segment of single-pass laboratory experiments on limestone. The material of the Nast tunnel, however, has varied significantly over the length of the tunnel so that, for the purposes of this discussion, it cannot be assumed that rock properties and rock strength stayed constant over this length. All indicators are that the Nast tunnel machine was operating in the preoptimum mode. Therefore, it appears that the variation of specific energy for the Nast tunnel can be more closely correlated with the variation of specific energy of minute spacing points plotted for various rocks.

Advance Rate

Groove depth and muck weight are the laboratory parameters that correspond to boring-machine rate (4). Figure 14 shows the variation of groove depth with cutting coefficient for single-pass and steady-state experiments. Single-pass results are given for critical, optimum, and minute groove spacing. For each of these three cutting modes, the value of groove depth increases as the cutting coefficient increases. For the same rock type, as the cutting coefficient increases from the critical value through the postoptimum region, groove depth also increases. In the postoptimum region, granite has a negative slope and other rock types have positive slopes. The variation of groove depth with cutting coefficient can be used to identify the cutting mode only if the range covers a major portion of both preoptimum and postoptimum regions. Such identification is based on the slope of the line in Figure 13 that represents the preoptimum and postoptimum regions.

Figure 15 shows the variation of muck weight per groove length with cutting coefficient for single-pass grooves, including the representative points for critical, optimum, and minute spacing. In the postoptimum region, the muck value increases with increasing cutting coefficient for the same rock. In the preoptimum region, both muck and cutting-coefficient values decrease. The variation of muck with cutting coefficient in both the preoptimum and postoptimum regions has positive slopes; therefore, it may not be a good indicator of the cutting mode because in some cases the difference in slope may not be significant.

Figure 16 shows the variation of boring rate with cutting coefficient for the Nast tunnel. This variation closely approximates a linear pattern and agrees with the laboratory data shown in Figures 13 and 14. Although other indicators point out that this machine operates in the preoptimum region, this variation cannot be used for such conclusions because the slopes of the preoptimum and postoptimum regions are both positive.

Figure 17 shows the variation in advance rate with forward thrust for the Lawrence Avenue and Nast tunnel operations. If one considers the principle that increasing thrust and decreasing spacing are analogous (4) and compares Figures 3 and 16, it becomes apparent that the Lawrence Avenue operation is in the postoptimum mode and the Nast tunnel operation is in the preoptimum mode. With increasing thrust, the rate of advance in the Nast tunnel decreases; in the Lawrence Avenue tunnel it increases. The former compares with the postoptimum region and the latter with the preoptimum region shown in Figure 3.

Muck Size Distribution

The muck obtained from single-pass or steady-state laboratory experiments may not be expected to have the same size distribution as that produced by a boring machine. It does, however, provide information that can be used to predict and analyze field situations with much more accuracy and by the help of more indicators. Generally, the material produced at the optimum spacing is substantially coarser than that obtained at the independent spacing. Plots of muck size distribution for the entire spec-

Figure 11. Specific energy versus muck weight for laboratory experiments.

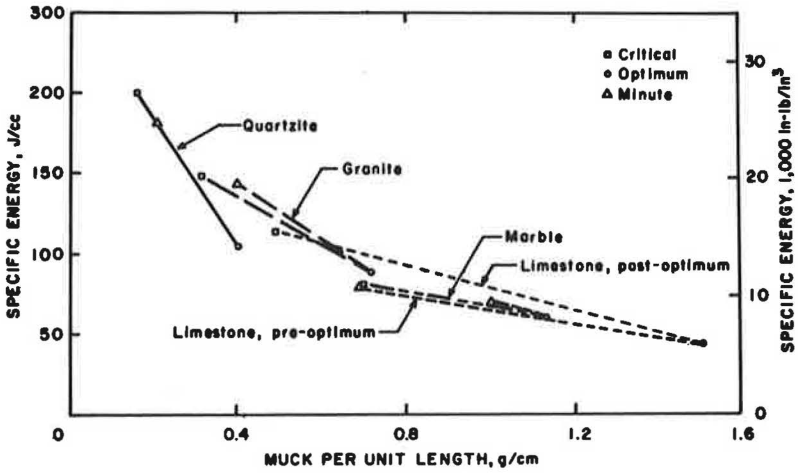


Figure 12. Specific energy versus groove depth for laboratory experiments.

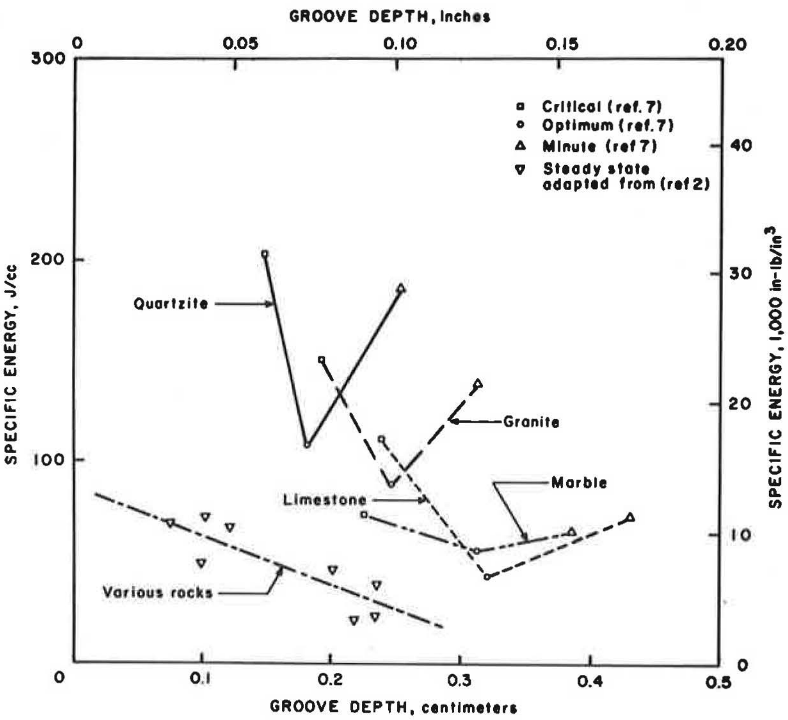
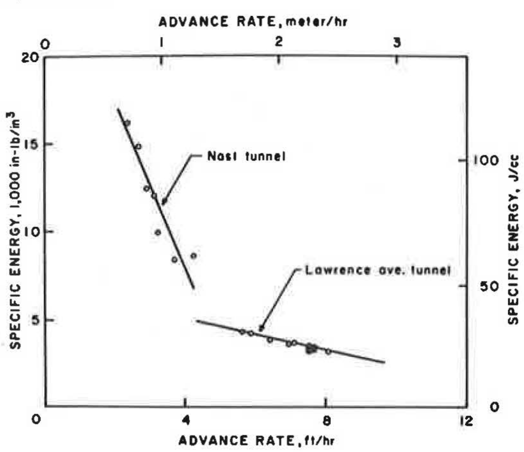


Figure 13. Specific energy versus advance rate for field operations.



trum of spacings (6) have indicated that the curves show a general coarsening of muck as the spacing is reduced from critical toward optimum (arrow a in Figure 18). If the spacing is reduced below the optimum spacing, the muck size distribution becomes fine again. For very small spacings, muck size distribution becomes similar to the distribution of muck obtained from independent grooves (arrow b in Figure 18). It has also been found that at the optimum spacing rocks of higher compressive strength produce a finer size distribution than more cuttable rocks of lower compressive strength (5). Comparison of field data and laboratory data shows that near-optimum conditions are obtained if about one-third of the pieces that compose the muck are larger than 40 percent of the distance between the neighboring grooves or the studs of the cutters (7).

Figure 19 shows the size distribution of muck obtained from the Lawrence tunnel operation and from two different locations in the Nast tunnel (the samples from the two

Figure 14. Groove depth versus cutting coefficient for laboratory experiments.

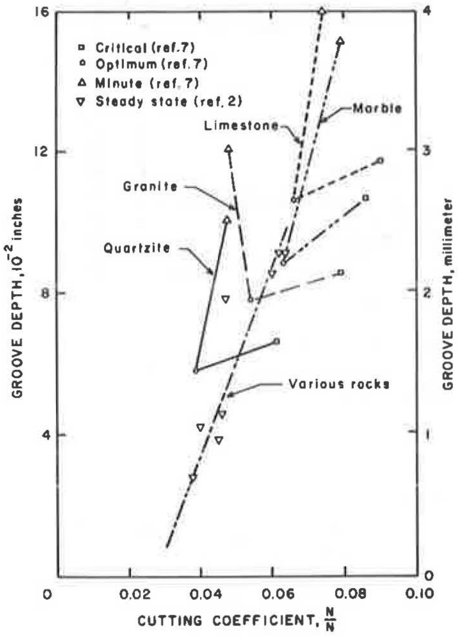


Figure 15. Muck weight versus cutting coefficient for laboratory experiments.

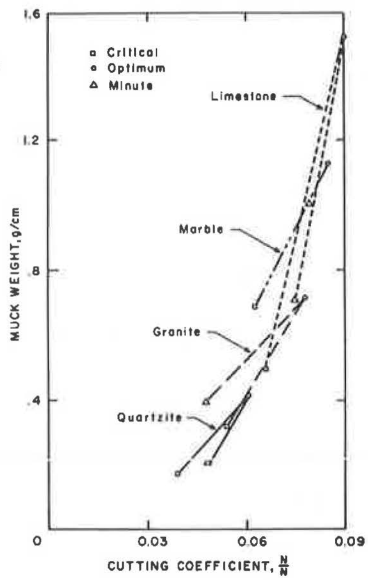


Figure 16. Advance rate versus cutting coefficient for Nast Tunnel operations.

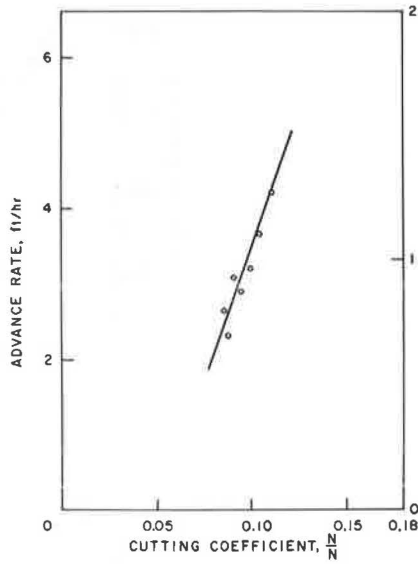
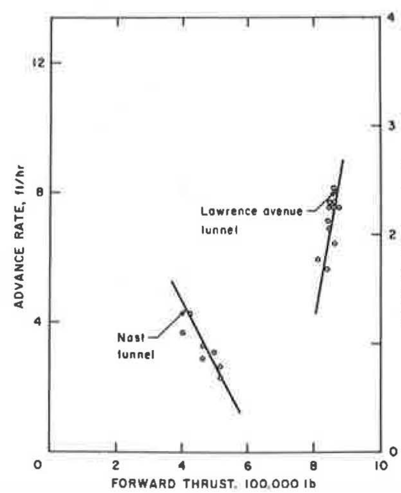


Figure 17. Advance rate versus forward thrust for field operations.



locations in the Nast tunnel are of the same rock type but of different compressive strengths). The Lawrence Avenue muck is much coarser than that from the Nast tunnel. The reason for such a difference is not immediately obvious but is probably a combination of differences in rock type and structure, thrust, cutter type, and speed of rotation.

Interestingly, the size distribution of the muck obtained from the Nast tunnel indicates that the material obtained from the stronger rock is coarser. This observation confirms other observations suggesting that the Nast tunnel boring machine is operating in the preoptimum region and as the rocks get stronger the conditions become closer to optimum and so coarser material is produced.

CONCLUSIONS

Laboratory experiments in tunnel boring conducted under the constant normal force mode or the constant penetration mode are capable of producing results that are beneficial to the design and operation of boring machines. Continuously monitoring thrust, torque, advance rate, and muck size distribution in boring machine operations will provide indicators by which the performance of the machine can be systematically assessed. The values for advance rate and cutting coefficient for boring machines can be represented in the laboratory by groove depth, muck weight, and cutting coefficient. Steady-state laboratory experiments result in values of specific energy higher than those found in field operations if the field boring conditions are postoptimum. Tunnel boring machines may be operated in either the preoptimum or postoptimum region. The former is suitable for highly variable rock whereas the latter is suitable for relatively uniform rock.

Figure 18. Variation in muck size distribution with groove spacing.

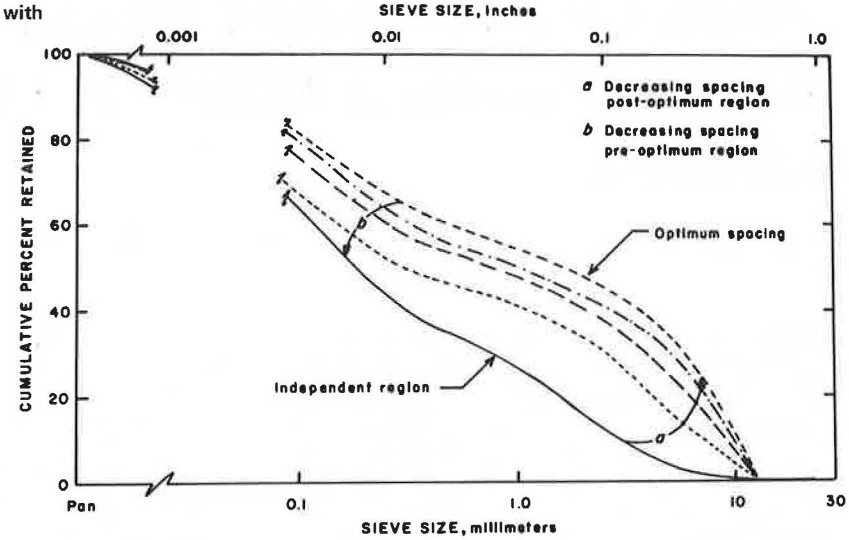
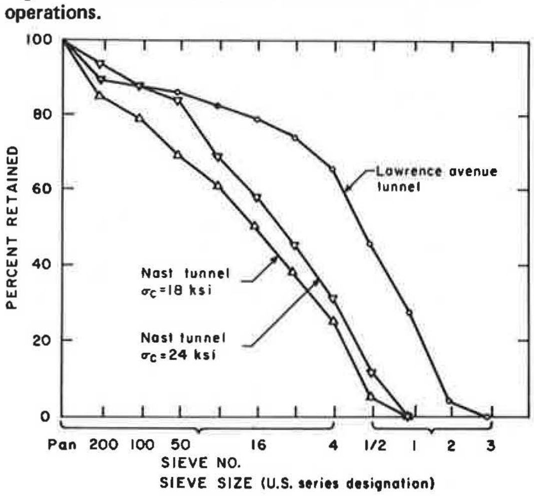


Figure 19. Size distribution of muck obtained in field operations.



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Statistical Quality Control Procedures for Airfield Pavement Materials and Construction

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The interaction of materials, construction, and the environment makes projections of pavement performance difficult. To increase the confidence with which these projections can be made, a simplified statistical quality control plan for airfield pavement materials and construction was developed. By the application of statistics, a relatively small number of randomly collected samples can be used to evaluate the quality of materials and construction. Test results based on a specific number of samples can be extended with the desired confidence limits based on probability theory to evaluate an entire lot of material. This statistical quality control plan evaluates pavement quality much better than do conventional specifications. It also describes how to handle materials or construction of borderline acceptability. When test results indicate that material or construction meets the desired quality, it is accepted at 100 percent payment; when it fails to meet the minimum requirements, it is rejected. When material or construction is found to be below the desired quality but above the minimum requirements, the job is paid for at a reduced cost. The evaluation is done on a day-to-day basis so that the engineer and the contractor know the acceptability of the job as it progresses.

In pavement construction, because poor-quality material and inadequate construction control can nullify the benefits of rigorous pavement design, quality control of material and construction is required if in-place pavement is to perform properly. In conventional methods of evaluating pavement material and construction quality, many samples are obtained and a number of tests are conducted. These tests indicate whether the samples pass or fail, but the results can be extrapolated with only limited confidence to indicate the level of quality of the pavement in general.

Confidence in the quality of pavement material and construction can be greatly increased by the use of random sampling and the application of statistical procedures. Such procedures, however, should result in conclusions that are compatible with sound engineering judgment. Statistics should be used to define what is acceptable and unacceptable in such a way that one engineer evaluating a job will reach the same conclusion as any other engineer who evaluates the quality of the finished product.

The objective of this paper is to present, in a general way, a plan for applying statistical procedures to the evaluation of airfield pavement materials and the type and method of pavement construction. This plan should define appropriate control parameters, sampling procedures, testing procedures, target values of measured parameters, and tolerances. In addition, the plan should provide for a reduction in job payment for noncompliance with specified quality. A detailed application is presented for bituminous concrete density.

STATISTICAL TERMS

Test results on a small amount of material can be used to estimate the properties of all of the material being evaluated (the population) within a certain degree of accuracy if the distribution function of the population is known. Data on pavement quality control generally fit a normal distribution curve (bell-shaped distribution function). The characteristics of a population can be better estimated by increasing the number of samples tested, but in most cases expense and time restrict the number of samples. An optimum sample number should be established to maintain accuracy while limiting time and expense.

A normal distribution curve for a population is described by two variables—the mean and the standard deviation. The population mean (μ) indicates the location of the peak of the curve; the population standard deviation (σ) indicates the width of the bell-shaped curve. A low value for σ indicates a narrow curve and a well-controlled product.

Given μ and σ, the percentage of a population falling within any given range can be computed. For instance, $\mu \pm \sigma$, $\mu \pm 2\sigma$, and $\mu \pm 3\sigma$ contain approximately 68, 95, and 99.7 percent of the total population respectively. If, for example, the specification limits of the population were set at 98 to 100 units and it was determined that μ and σ were 99 and 1 respectively, 32 percent of the population could be expected to fall outside the specification limits because 68 percent would be obtained between 99 $\pm$ 1 ($\mu \pm \sigma$) units. Thus, if specifications required that all test results be between 98 and 100 units, the specification limit would probably not be met in all cases.

Where a finite number of random samples are obtained, the mean is an unbiased estimator of the population mean and is represented by $\overline{X}$. Similarly, the random-sample standard deviation represented by $\hat{\sigma}$ is an unbiased estimator of the population standard deviation. The sample mean ($\overline{X}$), which is simply the arithmetic mean, is defined by

$$\overline{X} = \sum_{i=1}^{n} X_i \Big/ n \tag{1}$$

where X_i is an individual test value (the test result of the i th sample) and n is the total number of tests. The sample variance is defined by

$$\hat{\sigma}^2 = \sum_{i=1}^{n} (X_i - \overline{X})^2 \Big/ (n - 1) \tag{2}$$

The sample standard deviation is equal to the square root of the sample variance; i.e.,

$$\hat{\sigma} = \sqrt{\sum_{i=1}^{n} (X_i - \overline{X})^2 \Big/ (n - 1)} \tag{3}$$

If individual test results are grouped to form sets, the standard deviation of the sets (the standard error of the mean) will be less than the standard deviation of the individual results. The standard error of the mean is determined by

$$\overline{\sigma} = \sigma/\sqrt{n} \tag{4}$$

where σ is the standard deviation of the population and n is the number of tests to be grouped. From Equation 4 it can be seen that the distribution of sets of data is not as scattered as that of individual data points. If the results for four tests were averaged in the example above, the standard error of the mean for the set would be $\bar{\sigma} = \sigma/\sqrt{n} = \sigma/\sqrt{4} = \sigma/2$. Thus, given a population mean (μ) of 99 and σ of 1, only 5 percent of the grouped data would be expected to fall outside the specification limits because $\mu \pm \bar{\sigma}$ would be equivalent to $\mu \pm 2\sigma$.

Estimates of both the mean and the variance are needed to evaluate a population properly. The mean can be estimated by dividing the sum of the results by the number of tests conducted on random samples. The standard deviation for the population can also be estimated from these results.

One method of measuring the variation of test results consists of evaluating the following equation to determine the sample standard deviation:

$$\hat{\sigma} = \sqrt{\left[\sum_{i=1}^{n}(X_i - \bar{X})^2\right]\bigg/(n-1)} = \sqrt{\left[\sum_{i=1}^{n}(X_i)^2 - \left(\sum_{i=1}^{n} X_i\right)^2\bigg/n\right]\bigg/(n-1)} \quad (5)$$

FIELD QUALITY CONTROL DATA

Ideally, data used for the development of statistical limits should be obtained by visiting individual jobs before construction begins, setting up a statistical quality control plan, and obtaining the test results during construction. If such a method were used, however, it would take years to obtain the needed data. To collect data within a reasonable time frame, a number of agency offices were visited to obtain pavement quality control data for airfield jobs. The pavements chosen had been constructed within the past 10 years, appeared to be performing satisfactorily, and were recommended as having been constructed by capable contractors with trained personnel and good equipment. This method of obtaining data from which to estimate variations in pavement arbitrarily limits the analysis by excluding poor-quality pavements, but setting good quality as the target is desirable.

Airfield-pavement data were collected at 30 locations throughout the United States. The type of data collected and the number of locations for which they were collected are given below:

Data	Number of Locations
Subgrade	
Density	5
Water content	4
Lime-treated subgrade	
Density	2
Water content	2
Subbase	
Density	5
Water content	4
Base	
Density	8
Water content	7
Asphaltic concrete	
Aggregate gradation	19
Asphalt content	14
Percentage density	17
Portland cement concrete	
Slump	11
Air content	10
Flexural strength	11

Only data on bituminous concrete are presented here.

Field sieve analysis data collected for this study showed that a relation could be developed between percentage passing and the $\hat{\sigma}$ of percentage passing. McMahon and others have also developed such a data presentation (1). Figure 1 is a plot of percentage passing versus the standard deviation ($\hat{\sigma}$) of percentage passing for hot-bin gradations and gradations determined from aggregate recovered from bituminous mixture. These curves, which are the best fit, second-degree curves through the data points, can be used to determine the limits for the $\hat{\sigma}$ of percentage passing for each sieve.

Table 1 gives data obtained on asphalt content and mat and joint density. All asphalt contents were determined by using the rotorex method. The data show that the $\hat{\sigma}$ of asphalt content can be expected to be near 0.20 percent. The average mat density obtained in the field was approximately 98 percent of the laboratory density, and the average standard deviation for mat density was 1.0 percent.

Deterioration at joints is one of the most significant problems in asphalt pavement performance. The problem can be minimized by good workmanship in compacting, which ensures maximum possible density. On the average, joint densities are lower than mat densities. On the three jobs for which such data were obtained, the average joint density was approximately 1 percent below the average mat density, and the $\hat{\sigma}$ for joint densities was approximately 1.5.

QUALITY CONTROL PLAN

Basic Requirements

Several items must be defined for a successful statistical quality control plan (2). These items include (a) the size of a lot, (b) the number of tests for each lot, (c) the point of random sampling, (d) the sampling method, (e) the size of the sample (the number of measurements or test portion), (f) the test or measurement method, (g) the target (desired) value of the measured characteristic, (h) the realistic tolerance on the target value, and (i) action to be taken in case of noncompliance with requirements. In addition, in controlling a process, it is advantageous to use control charts. When these charts are used properly, they can indicate when a process is going out of control and allow the contractor to make the necessary adjustments. The use of control charts is discussed in greater detail later in this paper.

Description of a Lot

In a statistical quality control plan, the total job is divided into lots, each of which consists of a specified number of sublots. Each lot is considered to be a separate job and is accepted or rejected on that basis. Handling the quality control procedure in this way benefits both the contractor and the engineer by enabling them to identify lots quickly as acceptable or unacceptable.

The size of a lot (the quantity of production or the production time) can be selected based on two factors: (a) the approximate amount of daily production by the contractor and (b) the number of tests that would normally be conducted per day if conventional control procedures were used. By using this procedure, the expense of testing and the desired confidence level can be weighed and the appropriate lot size set.

If adjustments are necessary at some point in the production of a lot, the lot should be evaluated at that point and a new lot started after the adjustments are made. When lots are small, minor adjustments can be made when production of each lot is completed.

To minimize the expense of testing while maintaining a satisfactory level of confidence, it is recommended that the test results obtained from four samples selected at random from four equal sublots be used to evaluate each lot. When conditions indicate that more testing is needed, the lot size should be reduced.

Sampling

If a statistical analysis is to be unbiased, the data used for the analysis must be obtained from random samples. Ideally, in random sampling each sample has an equal chance of being selected. Probably the most convenient of the several methods for obtaining random samples is that using a random number table. As already stated, the lot should be divided into four equal sublots and one random sample should be obtained from each sublot. The time or the quantity of material between tests will thus be spaced. Another method of random sampling consists of taking four random samples from the entire lot. The disadvantage of this method, however, is that all samples could be taken after minimal production of materials or construction.

It is also recommended that, whenever possible, all tests be conducted on samples obtained from in-place construction so that there will be no problem obtaining additional samples for testing. If the testing is performed on samples obtained during production, there is no way to obtain additional random samples by the same sampling process after the lot is completed.

Tolerances

Specification tolerances allow for variation about an adopted job requirement and, on a statistical basis, vary with the number of samples in a lot. Tolerances should allow normal material and testing variations to be acceptable and should be set so that a finished pavement will have an acceptable level of quality.

It has been shown that the standard error of the mean can be computed by using Equation 4. Thus, from $\bar{\sigma} = \sigma/\sqrt{n}$, the distribution of a population has a standard deviation twice as high as that of the distribution for the average of four samples. Tables showing distribution of variances indicate that a distribution of $\hat{\sigma}$ for two samples can be expected to have a standard deviation of 1.27 times the distribution of $\hat{\sigma}$ for four samples. Thus, the tolerance limits for $\hat{\sigma}$ for two samples should be set at 1.27 times that for four samples.

Reduced Payment System

A reduced payment system or sliding scale pay factor (SSPF) is a useful feature to include in a statistical quality control system. In effect, the SSPF rates payment to correspond with the quality of material or construction. A satisfactory job would be paid at 100 percent of the bid price; a less than satisfactory job would be paid at less than the bid price; and a completely unsatisfactory pavement would be removed and replaced with satisfactory material at no extra cost to the user agency. Factors to consider in applying the SSPF to a less than satisfactory pavement are how much the pavement life has been reduced, the expected additional cost for maintenance, and the associated reduction in performance.

The majority of agencies use a tabular format for the SSPF. The SSPF, however, is graduated, and there is

Figure 1. Best fit curves for standard deviation versus percentage aggregate passing for hot-bin and extraction gradations.

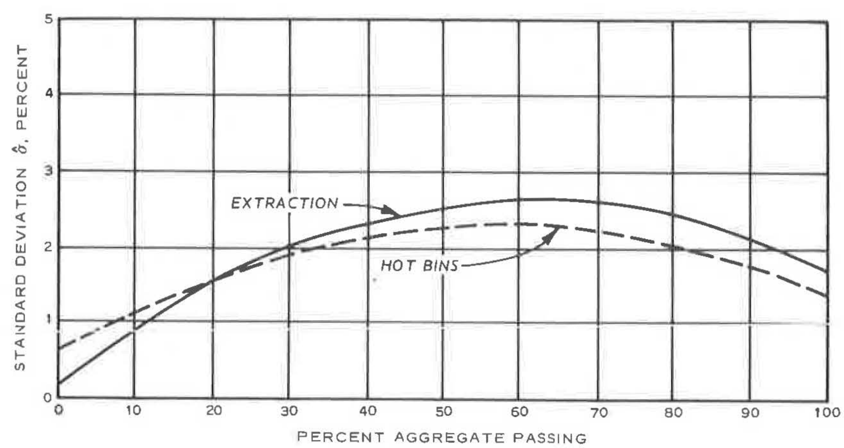


Table 1. Statistical data for asphalt concrete.

Asphalt Content			Mat Density			Joint Density		
Number of Samples	Mean	Standard Deviation	Number of Samples	Mean	Standard Deviation	Number of Samples	Mean	Standard Deviation
49	6.2	0.38	39	98.9	0.60	21	93.4	2.60
93	6.6	0.21	85	99.2	0.80	48	97.6	0.80
48	5.4	0.26	65	99.0	0.60	72	95.7	1.20
50	7.3	0.07	91	98.4	0.80			
189	5.3	0.11	96	98.6	1.40			
10	6.0	0.11	25	98.8	0.70			
74	5.0	0.20	26	98.8	1.30			
20	5.8	0.23	8	99.0	0.60			
20	6.1	0.11	30	99.5	0.80			
30	5.9	0.33	15	96.0	0.80			
5	5.7	0.18	99	97.2	1.00			
39	5.0	0.24	79	97.2	1.40			
38	5.2	0.27	9	96.8	0.80			
7	4.6	0.12	21	95.1	2.00			
69	5.7	0.25	48	97.8	0.40			
			21	96.0	1.90			
			72	96.8	1.40			

a significant decrease in payment at each step. The SSPF could also be represented as a continuous function or curve, which would allow an infinite number of SSPFs to be selected. Experience has shown, however, that taking points from a curve can result in controversy. In an attempt to take advantage of the tabular format and the continuous nature of a curve, it was decided to expand the size of the usual SSPF tables so that the steps in payment would be smaller.

Control Charts

The control chart is essentially a historical way of representing normally distributed data (bell-shaped curve). The target value of a control chart represents the μ (population average) of a normal curve. The upper and lower control limits are determined from σ and the degree of confidence desired. Therefore, any rules that would apply to normal curves would apply to control charts.

Test results plotted on control charts provide a simple, easily interpreted way of evaluating a continuing production or construction process. The charts can be used to identify trends and thus to identify and correct problems before they become severe.

The measures of variability that can be effectively evaluated on control charts are (a) individual values, (b) the moving average, and (c) the moving range. Any of these measures can indicate when the process has gone out of control. A process is considered out of control when one test value on any chart is above the upper control limit (UCL) or below the lower control limit (LCL).

The control limits are essentially a desired confidence interval or a range of values that, on the basis of a given sample, has a specified probability of including the true value. For example, a confidence level of 95 percent for a given test or process indicates that as many as 5 out of every 100 measurements can fall outside the control limits when the process is within control.

Two factors affect the selection of the confidence level:

1. The confidence level should be set so that test results falling outside these limits would be cause for classifying the process as out of control.
2. The confidence level should protect the buyer by ensuring that the material being bought is of acceptable quality.

Any significant changes that are made to the process should be marked on the control charts. By noting the process changes, most assignable causes for fluctuation in the control charts will be identified.

The control charts for individual measurements depend only on desired confidence and standard deviation. The control charts for average and range measurements depend also on the number of samples from which the range and the average were determined.

The UCL and the LCL for individual, average, and range measurements can be computed from the information given in the tables below by using the following formulas. For individual values, UCL = JMF + Kσ and LCL = JMF - Kσ; for average values, UCL = JMF + KAσ and LCL = JMF - KAσ; and for range values, central value = KBσ and UCL = KCσ where JMF = job mix formula, K = multiplication factor for desired level of confidence, and A, B, and C = constants.

Number of Samples	Factors That Determine: Control Limits of Mean Values	Central Value for Range	Control Limits for Range Values
2	0.71	0.38	1.23
3	0.58	0.56	1.45
4	0.50	0.69	1.57
5	0.45	0.78	1.64
6	0.41	0.84	1.69
7	0.38	0.90	1.73
8	0.35	0.95	1.77
9	0.33	0.99	1.80
10	0.32	1.03	1.82

Confidence Level	Multiplication Factor for Desired Confidence Level	Confidence Level	Multiplication Factor for Desired Confidence Level
50	0.67	92	1.75
55	0.76	93	1.81
60	0.84	94	1.88
65	0.93	95	1.96
70	1.04	96	2.05
75	1.15	97	2.17
80	1.28	98	2.33
85	1.44	99	2.58
90	1.65	99.5	2.81
91	1.70	99.9	3.09

To demonstrate the use of control charts, assume the following percentages of mat density were obtained for a bituminous concrete pavement:

	Lot 1	Lot 2	Lot 3	Lot 4	Lot 5
	95.2	98.4	99.5	98.1	100.0
	96.4	99.1	98.0	96.8	99.1
	97.4	97.3	97.5	99.8	98.3
	96.6	98.1	–	98.6	98.9
Average	96.6	98.2	98.3	98.3	99.1
Range	2.6	1.8	2.0	3.0	1.7

Assume the allowable σ to be 1.0 and the requirement for average density to be 98 to 100 percent. The upper control limits and lower control limits can be determined for 98 percent confidence limits as follows: For individual values,

UCL = JMF - Kσ = 100 + 2.33 (1) = 102.33
LCL = JMF - Kσ = 98 - 2.33 (1) = 95.67

For average values (n = 4),

UCL = JMF + KAσ = 100 + 2.33(0.50)(1) = 101.17
LCL = JMF - KAσ = 98 - 2.33(0.50)(1) = 96.83

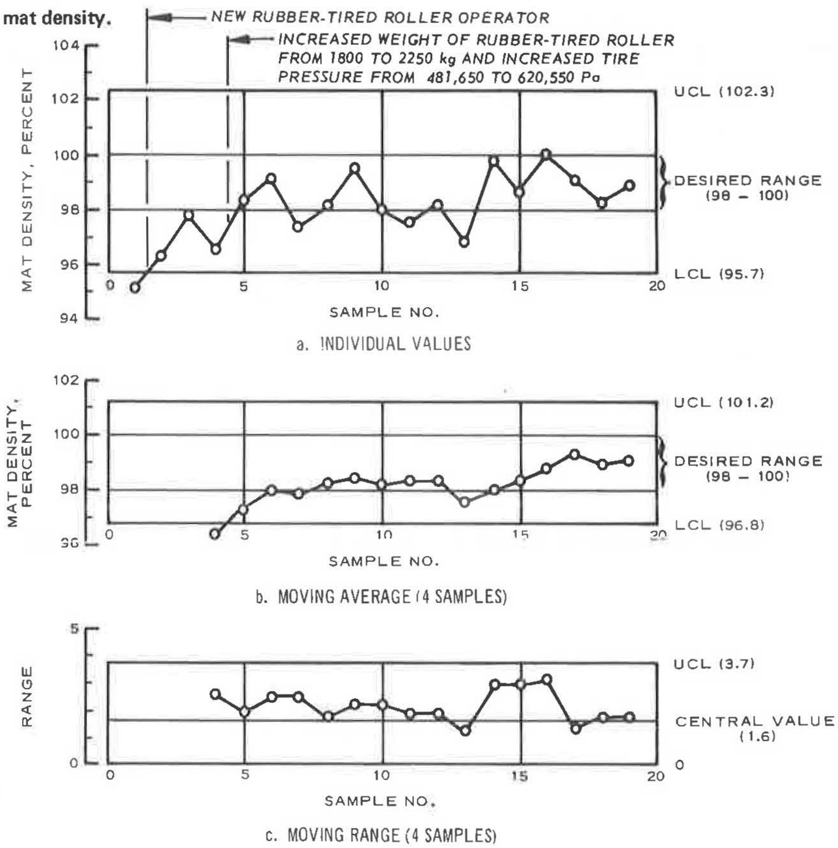
For range values (n = 4),

Central value = KBσ = 2.33(0.69)(1) = 1.61
UCL = KCσ = 2.33(1.57)(1) = 3.66

Figure 2 demonstrates the use of control charts for the above data. The first test result was obtained and plotted in Figure 2a. It can be seen that this measurement fell below the lower control limit. At this point, it should have been concluded that the process was out of control and the operation should have been stopped and the cause for the deficiency identified and corrected.

The second, third, and fourth samples were measured and found to be within the control limits. However, the average of the first four samples was below the lower control limit. At this point, the weight of the rubber-tired roller used in compacting the mat was increased from 18.1 to 22.7 Mg (20 to 25 tons), and the tire pressure was increased from 483 to 621 kPa (70 to 90 lbf/in^2). After these changes, the density measurements

Figure 2. Control chart for mat density.



NOTE: 1 kg = 1.102×10^{-3} TON AND 1 Pa = 1.450×10^{-4} psi.

were in control for the remainder of the job.

The moving average and the moving range are determined for the last four samples tested. For instance, for sample 6, the average and the range should be determined for samples 3 through 6. Plotting the moving average and the moving range allows more data points to be plotted and indicates trends sooner so that necessary adjustments can be made.

Check Tests

Retesting is not normally done; however, it may be necessary to obtain check tests during a job. Density may be retested for one of the following reasons:

1. The contractor or the user agency believes that the average test results from a lot are not representative of the average mat or joint density of the lot,
2. Samples are damaged before density testing, or
3. The lot has been rerolled after it was determined that the lot density was low.

Whenever the contractor or the user agency believes that the test results (mat or joint densities) measured from a lot are not representative of that lot, additional testing of the lot is justified. In such a case, if a lot has four sublots, three additional random samples are obtained from each of the four sublots. Each sublot is then evaluated as a lot by making a total of four tests. In no case are any of the test results excluded in the analysis of the lot density. No additional density testing is performed on this lot.

Whenever the contractor and the user agency agree that a sample has been damaged before testing, the sample is discarded and an additional random sample from the same sublot is obtained. Once a sample has been tested, the only causes for rejecting the test results are equipment failure or an improper testing procedure or both. When a sample or a test result is rejected, the sample number and the reason for rejection should be documented.

When a lot has been checked and determined to have low density, it is unlikely that rerolling of the lot will increase the density significantly; rerolling may be performed, however, at the option of the contractor. If a lot is rerolled, the first set of density results should be discarded. After rerolling, the lot is sampled again; two random samples (two mat and two joint samples) are taken from each sublot. The density results from sublots 1 and 2 are combined and evaluated as a lot, as are the density results from sublots 3 and 4. Thus, a lot that has been rerolled is divided into two lots for evaluation. No additional density testing is performed on this lot. The density results obtained before rerolling are not used in the final evaluation of the lot.

Summary of Procedure

The recommended step-by-step application of statistical quality control to pavement density is as follows:

1. Lot size should be set according to daily production. This lot size would be the most convenient to use in the field. In no case, however, should the lot size exceed 18.2 Gg (2000 tons).
2. Each lot should be subdivided into four equal sublots and a random sample selected for each sublot. As a result, four random samples should be taken from the mat and four from the joint of each lot. If it is felt that this is not a sufficient number of tests to evaluate the

Table 2. Percentage payment for mat and joint density.

Payment (%)	Average Density for Four Cores (%) Mat	Joint	Payment (%)	Average Density for Four Cores (%) Mat	Joint
100.0	98.0 to 100.0	97.0 to 100.0	93.0	96.4	95.4
100.0	97.9	96.9	92.1	96.3	95.3
99.9	97.8	96.8	91.1	96.2, 100.6	95.2, 100.6
99.8	97.7, 100.1	96.7, 100.1	90.1	96.1	95.1
99.6	97.6	96.6	89.0	96.0	95.0
99.4	97.5	96.5	87.8	95.9, 100.7	94.9, 100.7
99.1	97.4, 100.2	96.4, 100.2	86.6	95.8	94.8
98.7	97.3	96.3	85.4	95.7	94.7
98.3	97.2	96.2	84.1	95.6, 100.8	94.6, 100.8
97.8	97.1, 100.3	96.1, 100.3	82.7	95.5	94.5
97.3	97.0	96.0	81.3	95.4	94.4
96.7	96.9	95.9	79.8	95.3, 100.9	94.3, 100.9
96.1	96.8, 100.4	95.8, 100.4	78.2	95.2	94.2
95.4	96.7	95.7	76.7	95.1	94.1
94.7	96.6	95.6	75.0	95.0, 101.0	94.0, 101.0
93.8	96.5, 100.5	95.5, 100.5	0	<95.0, >101.0	<94.0, >101.0

pavement in general, the lot size should be decreased, but it is recommended that four samples be obtained from each lot.

3. Density measurements must be taken from the in-place pavement only.

4. All samples for a given job should be taken at random and be approximately the same size, and a table of random numbers or some other approved method for selecting random sample locations should be used.

5. A sample should be cut perpendicular to the surface for the full thickness of the pavement and should weigh at least 750 g (26 oz).

6. Field density is expressed as a percentage of the laboratory density. The average density of two sets of laboratory-prepared cores should be considered as the laboratory density of the lot. Two sets of laboratory-compacted cores (normally, for Marshall tests, three cores compose one set) can be prepared in a day by experienced technicians. If for any reason only one set of laboratory-compacted samples is prepared for a given day (lot), then the average density of this set is used as the laboratory density. If no laboratory-compacted samples are prepared for a given day, then the laboratory density for the previous day is used as the laboratory density. But in no event should a prior lot laboratory density be used if any changes in material or mix proportions have been made.

7. The recommended average field density is 99 percent of the laboratory density for the mat and 98.5 percent of the laboratory density for the joints.

8. The average standard deviation for individual mat samples is 1.0 percent, and the standard deviation for the average of four samples is half that or 0.5 percent. If the overall average density is 99 percent, 95 percent of all the tests (the average of four samples) will be between 98 and 100 percent. It is recommended that ±1 percent be set as the tolerance on the target value for the mat samples. Thus, 98 to 100 percent density will be acceptable for 100 percent payment. For the joint samples, a standard deviation of 1.5 percent can be expected, and a target value of 98.6 percent is recommended. A 97 to 100 percent joint density is recommended as acceptable for 100 percent payment.

9. As already noted, in a well-constructed job up to 5 percent of the lots could fail to meet density requirements. When field density is not in compliance, the payment for that lot should be determined from data given in Table 2. The lower percentage payment of joint or mat density will be the percentage payment for the lot. When the density of the asphalt concrete in a lot is such that there is no payment for that lot, the contracting officer has the option to require the contractor to remove and replace the asphalt concrete in that lot. When the lot is removed and replaced, it should be paid for at the percentage payment determined for the density of the final in-place asphalt concrete. When the unsatisfactory asphalt concrete is left in place, no payment is made to the contractor for that lot.

ACTUAL APPLICATION OF STATISTICAL QUALITY CONTROL REQUIREMENTS

After the large quantity of data from paving projects were collected and analyzed, some doubt remained as to how well the statistical quality control procedures would work on an actual paving job. In the summer of 1976, the runway at Shemya Air Force Base in the Aleutian Islands, Alaska, was overlaid with 5 cm (2 in) of asphalt concrete. The statistical quality control procedures were applied to the density requirements for this job. During construction of the job, temperatures ranged from 4 to 13°C (40 to 55°F), the wind was blowing constantly at speeds of 16 to 48 km/h (10 to 30 mph), and the weather was cloudy or foggy about 90 percent of the time.

A lot was set equal to one day's production. The specifications required that the average mat density be 98 percent or greater and the average joint density be 97 percent or greater for 100 percent payment. When the average mat density was <95 percent or the average joint density was <94 percent, the lot was removed and replaced or paid for at 25 percent of the bid price, as determined by the engineer. If the average density was between the acceptance and rejection limits, the pavement was purchased at a reduced cost. The payment for the mat density was determined from a straight-line reduction that ranged from 100 percent payment at 98 percent density to 70 percent payment at 95 percent density. For the joint density, the straight-line reduction ranged from 100 percent payment at 97 percent density to 85 percent payment at 94 percent density.

Equipment furnished by the contractor included a 7.6-m-wide (25-ft) paver and seven rollers consisting of one vibratory roller; one three-wheel tricycle roller; two 32-Mg (35-ton) rubber-tired rollers; one three-wheel, three-axle roller; and two tandem, steel-wheel rollers. Skirts were placed around the rubber-tired rollers to help hold the heat in and keep the tires from picking up the asphalt mix. All seven rollers were used in the rolling operation, but the same density could probably have been achieved by using four, or at most five, rollers.

Because of difficulty in achieving dense longitudinal joints, much time and effort were spent working on these joints. The longitudinal joints were continuously rolled by one of the steel-wheel rollers. All free edges were cut back to expose a dense vertical edge.

Density and percentage payment for each lot are given below (1 Mg = 1.1 tons):

Lot	Megagrams Placed	Average Mat Density	Average Joint Density	Payment (%)
1	547	98.5	–	100
2	1120	97.9	96.1	95.5
3	494	97.7	96.9	97.0
4	1280	97.9	95.2	91.0
5	318	96.1	94.1	81.0
6	708	98.4	97.4	100
7	1498	98.4	97.2	100
8	1362	98.4	97.2	100
9	600	98.3	98.0	100
10	1450	98.3	95.5	93.5
11	1590	98.6	97.4	100
12	1797	99.1	97.7	100
13	1325	98.3	97.3	100
14	1000	97.7	96.6	97.0

Average and standard deviation values for the entire job for aggregate gradation, asphalt content, and density are given below:

	Shemya		Previous Jobs	
Item	Average	Standard Deviation	Average	Standard Deviation
Density, %				
Mat	98.2	1.07	97.8	1.0
Joint	96.7	1.43	96.8	1.5
Asphalt content, %	6.7	0.33	–	0.20
Gradation, % passing sieve size				
12.7 mm (½ in)	100	0	95 to 100	2.0
9.52 mm (⅜ in)	96.8	1.2	95 to 100	2.0
4.76 mm (No. 4)	72.2	2.1	30 to 95	2.5
2.38 mm (No. 8)	51.7	1.4	30 to 95	2.5
0.047 mm (No. 16)	38.3	1.2	30 to 95	2.5
0.023 mm (No. 30)	28.4	1.5	20 to 30	2.0
0.297 mm (No. 50)	16.7	1.2	10 to 20	1.5
0.149 mm (No. 100)	9.4	1.3	0 to 10	1.0
0.074 mm (No. 200)	6.9	1.0	0 to 10	1.0

Most of the data obtained from the Shemya job compared reasonably well with data obtained on previous jobs. Given the satisfactory material and construction properties obtained on this job under poor weather conditions, it is believed that density and other quality parameters can be better controlled and evaluated by using statistical rather than conventional specifications.

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Quality Assurance in Bridge Construction in Canada: A Study of Inspection and Testing Programs

J. Ryell and L. Bowering, Ontario Ministry of Transportation and Communications

A study is summarized of inspection times and concrete testing programs on 21 contracts of the Ontario Ministry of Transportation and Communications containing 41 bridges built in 1974. The study assessed the degree of uniformity in the Ministry's work with respect to inspection and field testing programs and compares such programs, where appropriate, with recommended practices and specification requirements. Data are presented on inspection times for reinforcing steel, formwork and falsework, concrete placement, erection of beams, stressing and grouting of cables, deck waterproofing, and other operations. Test programs on fresh and hardened concrete are discussed. The study notes a wide variation in the effect of inspection on comparable segments of different structures and attributes this partly to the individual inspector's perception of the scope and nature of the required inspection effort. A practical bridge-inspection manual and checklists that will establish uniform standards of inspection and provide the basis to build more accountability into inspection work are required. It is concluded that inspection and testing programs, particularly on critical segments of the bridge structure such as the deck, require substantial improvement.

In 1974, a project was carried out by the Ontario Ministry of Transportation and Communications (MTC) to evaluate the adequacy of current MTC quality control of bridge construction, particularly of bridge-deck construction. One part of the project and the subject of this report consisted of a study of current inspection and testing programs by means of data collection from 21 active contracts. The 41 structures contained in these contracts included a number of design types and were distributed in each of the five regions of the province of Ontario.

The purpose of the study was to assess the degree of uniformity that exists throughout the province with respect to inspection and field testing programs and to compare such programs, where appropriate, with recommended practices and specification requirements. The data collection was carried out on all MTC contracts that contained active structural work for the period September 1 to December 13, 1974.

Tests on slump, air content, temperature, and compressive strength of concrete were summarized from the concrete construction report filled out daily by the inspector and from the report on concrete strengths issued by the testing laboratory. Compliance was based on the requirements of MTC specifications in effect at the time of the study and on the contract drawings. The adequacy of the number and frequency of field tests was judged by comparison with the table contained on the back of the inspector's concrete construction report (Figure 1), which is formulated in customary rather than SI units of measurement.

Figure 1. Form for field testing of concrete from inspector's concrete construction report.

LEGEND: < = LESS THAN
> = GREATER THAN

TESTING OF CONCRETE (1) – NUMBERS AND FREQUENCY OF FIELD TESTS

LABORATORY SERVICES OFFICE

LOCATION	STRUCTURES						PAVEMENTS & BASE (Exposed Concrete (7))			CURB & GUTTER (Exp. Concrete (7))		CULVERTS & MISC. WORK		PLANT MADE PRECAST BEAMS, PILES, ETC. (8)
	FOUNDATIONS, WALLS, PIERS, COLUMNS, ABUTMENTS			DECK SLABS, APPROACH SLABS, BRIDGE CURBS & SIDEWALKS (Exposed Concrete (7))										
SPECIFIED 28 DAY COMPRESSIVE STRENGTH OF CONCRETE P.S.I.	2,500, 3,000 4,000 & 5,000 (11)			3,000, 4,000 & 5,000 (11)			3,000 & 3,500			3,000		3,000		4,000 AND GREATER
QUANTITY OF CONCRETE PLACED PER DAY CUBIC YARDS (2)	< 100	100 TO 500	> 500	< 100	100 TO 500	> 500	< 500	500 TO 1,000	> 1,000	< 100	> 100	< 100	> 100	
TESTS														
28-DAY CYLINDERS (3) (4) (10)	1 SET PER DAY (6)	2 SETS PER DAY	3 SETS PER DAY	2 SETS PER DAY (6)	3 SETS PER DAY	4 SETS PER DAY	1 SET PER DAY (6)	2 SETS PER DAY	3 SETS PER DAY	1 SET PER DAY (6)	2 SETS PER DAY	1 SET PER DAY (6)	2 SETS PER DAY	AS DIRECTED BY THE ENGINEER, TO BE RELATED TO THE NUMBER OF UNITS CAST, AND THE QUANTITY OF CONCRETE PLACED, PER DAY
10-DAY FLEXURAL BEAMS (3) (5) (10)	—	—	—	—	—	—	1 BEAM PER DAY	2 BEAMS PER DAY	3 BEAMS PER DAY	—	—	—	—	—
AIR TEST (7) READY-MIX AND CENTRAL-MIX PLANT	AFTER SATISFACTORY CONTROL (9) IS ESTABLISHED 1 TEST FOR EACH 5 TRUCKLOADS OF CONCRETE			1 TEST FOR EACH TRUCKLOAD OF CONCRETE WHEN RATE OF PLACEMENT IS 50 CU. YDS. PER HOUR OR LESS. WHEN RATE OF PLACEMENT EXCEEDS 50 CU. YDS. PER HOUR AND AFTER, SATISFACTORY CONTROL (9) IS ESTABLISHED. 1 TEST FOR EACH 3 TRUCKLOADS OF CONCRETE.			AFTER SATISFACTORY CONTROL (9) IS ESTABLISHED 1 TEST FOR EACH 3 TRUCKLOADS OF CONCRETE.			1 TEST FOR EACH TRUCKLOAD OF CONCRETE.		AFTER SATISFACTORY CONTROL (9) IS ESTABLISHED 1 TEST FOR EACH 5 TRUCKLOADS OF CONCRETE.		AFTER SATISFACTORY CONTROL (9) IS ESTABLISHED 1 TEST FOR EACH 5 TRUCK LOADS OF CONCRETE
SITE-MIX	AFTER SATISFACTORY CONTROL (9) IS ESTABLISHED 1 TEST FOR EACH 10 TO 25 CUBIC YARDS CONCRETE DEPENDING ON SIZE OF MIXER ETC.													
SLUMP TEST	1. AT THE BEGINNING OF A CONCRETE PLACEMENT OPERATION AND AT SUBSEQUENT INTERVALS TO INSURE THAT THE SPECIFICATION REQUIREMENTS ARE MET. 2. WHENEVER CYLINDERS OR FLEXURAL BEAMS ARE MADE.													
YIELD	1. WHENEVER THE YIELD OF A CONCRETE MIX IS CHALLENGED. 2. TWICE PER DAY ON CONCRETE PAVEMENT OR CONCRETE BASE. 3. WHENEVER MATERIALS OR MIX PROPORTIONS ARE CHANGED.													
TEMPERATURE	1. IN HOT OR COLD WEATHER CONDITIONS, AT FREQUENT INTERVALS UNTIL SATISFACTORY CONTROL (9) IS ESTABLISHED. 2. WHENEVER CYLINDERS OR FLEXURAL BEAMS ARE MADE.													
ACCELERATED STRENGTH TESTING (3)	AS SPECIFIED IN MEMORANDUM OB-C-69-5 DATED OCT. 8, 1969, 'ACCELERATED STRENGTH TESTING'.													

NOTES:

(1) THE NUMBER AND FREQUENCY OF THE TESTS INDICATED REPRESENTS A REASONABLE LEVEL OF TESTING FOR CONCRETE ORIGINATING FROM PLANTS WITH A GOOD STANDARD OF QUALITY CONTROL. THE FREQUENCY OF TESTING SHOULD BE INCREASED FOR CONCRETE PRODUCED BY PLANTS WITH LOWER STANDARDS OF CONTROL.

(2) THE CONCRETE QUANTITIES SHOWN REFER TO ALL CONCRETE OF THE SAME CLASS, WITH THE SAME MIX SERIAL NUMBER, ORIGINATING FROM THE SAME CONCRETE PLANT FOR THE SAME CONTRACT. WHERE MORE THAN ONE MAJOR CONCRETE PLACEMENT OPERATION OCCURS ON THE SAME CONTRACT DURING THE SAME DAY E.G. ,THE CONSTRUCTION OF A SECOND BRIDGE DECK, EACH OPERATION SHALL BE TESTED AT THE FREQUENCY SHOWN.

(3) 28 DAY CYLINDERS AND 10 DAY FLEXURAL BEAMS ARE TO BE MADE AT THE FREQUENCY SHOWN TO DETERMINE WHETHER THE CONCRETE MEETS THE REQUIREMENTS OF THE STRENGTH SPECIFICATION. IN ADDITION JOB CURED CYLINDERS AND FLEXURAL BEAMS WILL FREQUENTLY BE REQUIRED BY THE ENGINEER TO DETERMINE CONCRETE STRENGTH FOR THE PURPOSE OF FORM REMOVAL, STRESSING, OPENING TO TRAFFIC ETC. 28 DAY STANDARD CURED COMPRESSIVE STRENGTH RESULTS CAN BE PREDICTED WITH REASONABLE ACCURACY USING THE AUTOGENOUS CURED ACCELERATED TEST PROCEDURE.

(4) A COMPRESSIVE STRENGTH TEST IS THE AVERAGE STRENGTH OF TWO STANDARD 6 IN. x 12 IN. CYLINDERS.

(5) A FLEXURAL BEAM TEST IS THE AVERAGE STRENGTH OF TWO BREAKS IN A STANDARD 6 IN. x 6 IN. x 36 IN. BEAM.

(6) IN VERY SMALL VOLUME PLACEMENT OPERATIONS, CYLINDERS NEED NOT BE MADE EACH DAY.

(7) EXPOSED CONCRETE* MUST BE PROPERLY AIR ENTRAINED AND SUFFICIENT TESTING CARRIED OUT TO ENSURE THAT THIS IS ACHIEVED. ON MANY CONCRETE PLACEMENT OPERATIONS, PARTICULARLY WHERE THE PROCEDURE IS KNOWN TO HAVE LESS THAN EXCELLENT STANDARDS OF CONTROL IT IS NECESSARY AND POSSIBLE TO CHECK EACH LOAD OF CONCRETE FOR AIR CONTENT. ON WORK SUCH AS PAVEMENTS AND LARGE BRIDGE DECKS WHERE VERY RAPID RATES OF CONCRETE PLACEMENT OCCUR , I.E., > 50 CUBIC YARDS PER HOUR, IT IS UNPRACTICAL TO CONTINUE SUCH A FREQUENCY OF TESTING FOR VERY LONG. UNDER SUCH CONDITIONS AND ONCE SATISFACTORY CONTROL HAS BEEN ESTABLISHED, THE FREQUENCY OF TESTING CAN BE REDUCED FOR AS LONG AS EACH TEST RESULT FALLS WITHIN SPECIFICATION REQUIREMENTS. WHEN A TEST RESULT FALLS OUTSIDE THE SPECIFIED LIMITS THE TESTING FREQUENCY SHOULD REVERT TO ONE TEST PER LOAD OF CONCRETE UNTIL SATISFACTORY CONTROL IS RE-ESTABLISHED.

(8) QUALITY CONTROL TESTING SHALL BE CARRIED OUT BY CONTRACTOR AND SUPERVISED BY M.T.C.

(9) SATISFACTORY CONTROL IS CONSIDERED TO HAVE BEEN ESTABLISHED WHEN TESTS ON FIVE CONSECUTIVE TRUCKLOADS OR BATCHES (SEE NOTE (2)) OF CONCRETE ARE WITHIN SPECIFICATION REQUIREMENTS.

(10) SEE ATTACHED SHEET 'METHOD OF RANDOM SELECTION OF CONCRETE SAMPLE ETC

(11) THE ENGINEER MAY DIRECT THAT THE FREQUENCY OF TESTING FOR 5,000 P.S.I. CONCRETE BE INCREASED ON SOME OCCASIONS.

* CONCRETE SUBJECTED TO FREEZE-THAW CONDITIONS WITH DE-ICING SALTS PRESENT E.G., BRIDGE DECKS, PAVEMENTS, CURB AND GUTTER SECTIONS WALLS AND COLUMNS AFFECTED BY ROAD SPLASH.

Figure 2. Daily report on bridge inspection.

DAILY REPORT ON BRIDGE INSPECTION

DISTRICT NUMBER 6 CONTRACT NUMBER 74-20

SITE NUMBER 131-74 DATE July 16, 1974

INSPECTOR'S NAME T. L. Brown CLASSIFICATION H. 1. A. 3.

INSPECTION RECORD

*FUNCTION	*LOCATION	TIME (Hrs)
2	A-North	3.50
1	B-#1	1.
15		0.50
17		3.00

*See reverse side of form for codes.

RECORD OF VISITS TO BRIDGE SITE

NAME	SECTION	POSITION	TIME (Hrs)	REASON
M. Smith	District	Prov.Supervisor	0.50	General Insp.
L. Jones	Region	Sup.Inspector	2.50	Conc.Supply
P. Green	District	Const.Engineer	3.00	Construction Schedule Review

Mail white copy of form to: L. Bowering (416/248-3965), Engineering Research and Development Branch, Ministry of Transportation and Communications, Ontario, 1201 Wilson Avenue, Downsview, Ontario, M3M 1J8.

METHOD OF COMPILING THE DAILY REPORT ON BRIDGE INSPECTION

1/ Every day that the contractor works, each bridge inspector completes one form for each bridge inspected. (It is possible that some forms will indicate no inspection for a particular day. This may be due to the fact that construction activity is suspended because of delays or inclement weather. Also, forms will indicate no inspection when the Inspector is not physically on-site).

2/ Record the bridge location information, the date, your name and classification.

3/ In the Inspection Record section, under the function column, insert the code number that describes the work you are performing. If your work does not fit into one of the categories listed on the reverse side of the form, insert one of the unassigned numbers and write a brief description of your work on the reverse side of the form, beside the number you have selected. Code number 18, designated as Other, will include time spent on phases of the contract other than the bridge, time spent on administration - e.g. time sheets, coffee breaks, travel time, etc. We are not interested in a breakdown of this time.

4/ Under the Location column, insert the code letter that indicates the section of the bridge you are inspecting. Beside the code letter, designate which abutment, pier, etc. if there is more than one. For example, write N for north abutment and No. 1 for pier number one.

5/ Record your time to the nearest half hour.

6/ In the Record of Visits to Bridge Site section, record the name, section and position title of all individuals that visit the site. Since other district inspectional staff will be compiling their own form, their visits will not be recorded in this section.

7/ Record the duration of all visits to the nearest half hour.

8/ Under the Reason column, briefly state the problem that precipitated the visit. An unsolicited visit will be referred to as a general inspection.

9/ Enquiries regarding the form may be directed to: L. Bowering at 248-3965 or 248-3966.

FUNCTION CODE

1/ Check excavation.
2/ Check reinforcing steel placement.
3/ Inspect concrete placement.
4/ Perform quality control tests on concrete.
5/ Inspect formwork.
6/ Inspect falsework.
7/ Check bearing placement.
8/ Check stressing cable placement.
9/ Inspect stressing operation.
10/ Inspect grouting operation.
11/ Inspect beam erection.
12/ Inspect waterproofing operation.
13/ Inspect paving operation.
14/ Inspect pile-driving operation.
15/ Instruct contractor's staff.
16/ Compile diary and inspectional forms.
17/ Visits to the concrete plant and other suppliers.
18/ Other
19/
20/

LOCATION CODE

A/ Abutment Footings.
B/ Pier Footings.
C/ Abutments.
D/ Piers.
E/ Wingwalls.
F/ Deck.
G/ Ballast Walls.
H/ Curbs.
J/ Parapet Walls.

Inspection time was summarized by using the daily report on bridge inspection form shown in Figure 2. All bridge inspectors completed one report for each day that the contractor worked. This form recorded the hours spent on each inspection function—steel placement, stressing operation, pile driving, and so on—for each segment of the structure as well as all noninspection time. Visits to the site by personnel from the regional, head, and district offices were also recorded on this form.

A strict comparison between the inspection effort actually made and what is required is not possible: No formal standards exist within the MTC for inspection duties on modern bridge construction. Some guidance on various facets of inspection is contained in documents such as MTC inspection and construction manuals, miscellaneous technical notes distributed at training courses, and memorandums originating from the Operations Division.

The project supervisor for research and development visited each contract site at least once during the early part of the project to outline the purpose and the aims of the study, to promote interest in the program, and to ensure that the daily report on bridge inspection was being correctly filled out by each inspector.

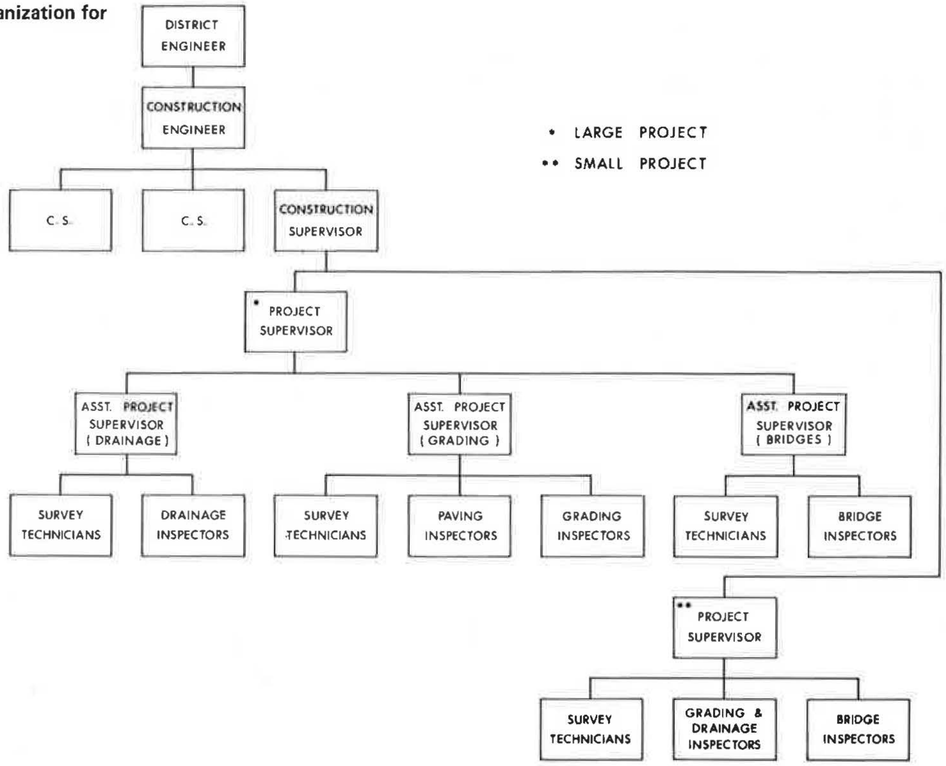
MTC DISTRICT STRUCTURE

At the time of the study, administration of construction contracts was the responsibility of 18 district offices spread over the 1 068 588 km^2 (412 582 miles^2) of Ontario. The organizational structure of these offices is shown in Figure 3. In most cases, only the district engineer and the construction engineer were professional engineers.

In addition to district offices there are five regional offices in Ontario that provide technical assistance and advice in matters of construction and quality assurance. Each region typically has (a) a quality control engineer who is assisted by several specialized supervising inspectors and (b) a fully equipped materials testing laboratory.

The head office of MTC in Toronto also provides various technical services to the district engineer's staff. The more important head-office functions in relation to the quality assurance system are represented

Figure 3. MTC district-level organization for construction.



by a structures office that is responsible for a specialized technical service such as cable stressing and approval of falsework plans; a quality control office that is concerned with technical advice on materials technology, inspection, and testing; and a laboratory services office that carries out concrete mix designs and testing and approval of products and materials.

TYPES OF BRIDGE DECKS

In this study, bridge decks are considered in two categories:

1. Thin-slab decks are single-course, reinforced concrete slabs supported on steel or precast concrete beams. In most cases the slab thickness is 19 cm (7.5 in) and contains four layers of reinforcing steel. The layout of formwork and reinforcing steel for such decks is relatively straightforward.
2. Thick-slab decks are reinforced concrete slabs posttensioned in a longitudinal and transverse direction. In most cases the slabs contain voids to reduce dead weight. When round metal voids are used, the concrete slab is placed in a single operation. When wooden, rectangular void forms are used, the concrete is placed in two operations. The overall slab depth for most structures in this group varies from 0.6 to 1.5 m (2 to 5 ft).

The layout of voids, reinforcing steel, and stressing cables in thick-slab decks is complex and congested, particularly around cable anchorages and above supporting columns.

All bridge decks were waterproofed and paved with bituminous materials.

DETAILS OF THE STUDY

In analyzing the data it is important to recognize the inherent limitations of the study and their effect on the conclusions.

The short data collection period resulted in only partial coverage of the construction of the structures. In some cases the various segments of a structure are not entirely covered; e.g., a portion of the substructure—a column or an abutment—was completed prior to the study or was not constructed during the study period. If a particularly complex or relatively simple portion were completed during the study but not the remainder of the structure, a false impression of the effectiveness of the inspection may be derived.

Having the inspectors record and report their own daily tasks may have an inflationary effect on inspection time. The knowledge that one's performance is under observation will generally produce an increased effort. In addition, actual inspection time may be "padded" to indicate superior performance or to hide portions of the working day when inspection activity is not required. To minimize these factors, it was requested that the forms be sent directly to the project supervisor for research and development. This procedure eliminated scrutiny of forms by field or district office supervisors and any influence this would have on the results.

Some idea of the overall effectiveness of the inspection effort can be gained by reviewing the inspection time spent on various phases of bridge construction. Several factors contribute to the ratio between the time spent and the inspection accomplished. The ability and the experience of the inspector are probably the most important factors. A fully experienced, capable inspector will achieve the objectives in a minimum of time. An inept inspector will not produce the desired results regardless of the time spent.

Staffing is also a factor. Overstaffing or understaffing of a contract will result in a corresponding variance in the inspection time for each operation. The influence this situation would have on the quality of inspection cannot be determined from this study, but a reasonable assumption would be that understaffing would result in a decline in the quality of inspection and overstaffing would have little if any effect.

Many inspection and testing operations are directly related to the contractor's speed and ability. When a contractor is capable and desires to produce a quality product, the inspector need only check the finished product for adherence to tolerances and specifications. If the contractor lacks experience or attempts to cut corners, some operations may have to be performed a second time so that the inspector can demonstrate the proper procedure. In this case, the inspector may have to observe the entire operation, solving problems and corecting errors as they occur.

Inspection times related to a specific structure or part of a structure can thus be expected to vary widely across the province, but such variation will usually not indicate unsatisfactory work by either the contractor or the MTC staff.

Testing programs can also be expected to vary between contracts but to vary much less than inspection times. The number and the frequency of field tests of concrete required by the MTC were developed over many years of bridge-construction experience and appear rational when they are compared with the requirements of other authorities and established national and international standards. Situations are not likely to arise in which the number and frequency of tests should be much less than those called for in the concrete construction report, but the nature of the contractor's operation may often require an increased frequency of testing.

Analysis of the daily report on bridge inspection for all bridge sites indicates that 60 percent of the inspector's time is spent in the field on inspection and testing duties, 7 percent in the office compiling diaries and records and studying contract documents, and 33 percent on work other than inspection and testing, off-site visits to concrete plants, vacations and sick leave, and nonproductive time attributable to inclement weather or lack of work.

Inspection and Testing

Placement of Reinforcing Steel

Establishing the time required to properly inspect the placement of a megagram of reinforcing steel is difficult. Variables that affect inspection time include (a) the complexity of the reinforcing steel configuration, (b) the skill of the contractor's staff, (c) the ability of the inspector, and (d) the inspector's concept of the inspection procedures. From the range of inspection times shown in Figures 4, 5, and 6, it is evident that these variables have considerable influence. Among these variables, the inspector's view of the scope and nature of the inspection effort is probably largely responsible for any wide variation. Some inspectors act as foremen, instructing the contractor's staff at the site during the entire operation. Others check the steel after the contractor has placed it and then advise the contractor of deficiencies.

Checking the steel for a typical three-span, thin-slab bridge deck in this study represents an average inspection time of almost 2 d. This at least indicates that on MTC contracts, as they are presently staffed, adequate inspection time is available and is utilized as the basis for a reasonable inspection effort.

The longer times and greater variability of inspections of reinforcement in the substructure appear to be predictable; such work is often more complex and more difficult to inspect because of restricted access, and contains relatively small quantities of steel. In contrast, some bridge decks contain large quantities of reinforcement but normally require less inspection time because of the standard size and spacing of reinforcing bars.

The main improvements in reinforcement inspection that seem to be required are (a) establishment within the MTC of uniform standards covering the scope of the inspection effort, (b) providing the inspector with clearer placement drawings, and (c) avoiding situations in which the inspector supervises steel placement on behalf of the main contractor.

Concrete Placed in the Deck

Originally, inspection and testing of concrete were to be recorded as two operations, but the inspectors often experienced difficulty in separating the time and reported the total time under one heading. In the interest of uniformity, therefore, these two operations have been combined for reporting purposes. Figure 7 summarizes the inspection and testing time on 14 deck pours. As may be expected, the time spent on inspection and testing varies widely; it averages 8.6 h/100 m^3 (131 yd^3) of concrete for the 10 thin-slab decks and 5.2 h for the 4 thick-slab decks.

Establishing an optimum inspection time for a typical bridge deck is difficult because of major differences in such factors as structural complexity, concrete placement rate, and contractor's operation. Generally, however, the program of concrete testing required for a bridge deck and the need for continuous inspection of the placing, consolidating, and finishing operations call for at least three inspectors for the duration of the concreting operation.

The required inspection and testing time for 191 m^3 (250 yd^3) of concrete placed over 8 h in a thin-slab deck would be 12.6 h/100 m^3 (131 yd^3) of concrete; for 535 m^3 (700 yd^3) of concrete placed over 10 h in a thick-slab deck, an inspection time of 5.6 h/100 m^3 of concrete would be required. This amount of inspection and testing time was not put into the majority of deck slabs for which data are available; in some cases the time that was spent appears to have been inadequate. It is possible in the more extreme cases that unreported help was given by survey technicians and other MTC staff. But, because each contract was asked to report all inspection time, it must be assumed that a reasonable inspection effort was not made on many of the deck slabs studied. Shortage of inspection staff may be partly responsible for this underinspection.

Cylinder Tests

The requirements contained in the concrete construction report for the making and testing of 28-d cylinders are related to the size of the concrete placement operation and allow little latitude in number and frequency of tests. It is recognized that 34-MPa (5000-lbf/in^2) concrete may require an increased amount of testing at the discretion of the engineer, but no advice is given on the number of specimens to be made for very large concreting operations, i.e., those in excess of 765 m^3 (1000 yd^3). Table 1 gives concrete compressive strength data for 14 study sites.

In total, 80 sets of 28-d cylinders representing 5757 m^3 (7529 yd^3) of deck concrete were made or 1 set for each 72 m^3 (94 yd^3). Approximately half of the bridge decks were represented by adequate cylinder testing programs. There was a tendency to make more cylinders than were asked for on 34-MPa (5000-lbf/in^2) concrete and fewer than were specified on most 28-MPa (4000-lbf/in^2) deck slabs. On one thin-slab deck only 1 set was made for 253 m^3 (331 yd^3) of concrete, which is clearly inadequate.

Seventy percent of the total sets of cylinders, or 186 sets, were tested at ages other than 28 d; this apparently reflects a demand for data on the early strength of concrete for purposes of stressing and falsework removal. Fabrication and testing of such cylinders are very expensive processes. Because the data are usually needed by the contractor before the next step in construction can proceed, it seems logical that the cost of such testing should be borne by the contractor. This study did not indicate that the approved concrete suppliers were carrying out the program of quality control testing required in MTC specifications.

The basic 28-d cylinder strength requirement—that at least 90 percent of tests must meet or exceed the minimum specified strength—was complied with on the 10 deck slabs in which 28-MPa (4000-lbf/in^2) concrete was used. The fact that no cylinder test fell below the minimum strength indicates the relative ease of attaining specified strength levels for this class of concrete. This was not the case for 34-MPa (5000-lbf/in^2) concrete. Two 34-MPa deck slabs met the specified strength requirements, but two fell far short. The two failures occurred at two sites that were included in one contract; 62 and 29 percent of the tests fell below 34 MPa. A detailed review of the 28-d tests for site 5-219 at which 62 percent failures occurred (Table 1) indicates that almost half of the tests exceeding 34 MPa had slump values and air contents far below the average for the total number of tests reported. In all probability, a high percentage of the concrete in this deck did not meet specification requirements.

Although the results of the strength tests on these two decks were not satisfactory, it should be noted that an impressive amount of concrete testing was carried out before the concrete deck was placed. The mix proportions tested in the laboratory indicated a 28-d strength of 45.7 MPa (6630 lbf/in^2). This testing was followed by a substantial program of field trial batches

Figure 4. Inspection time on placement of reinforcing steel: bridge decks.

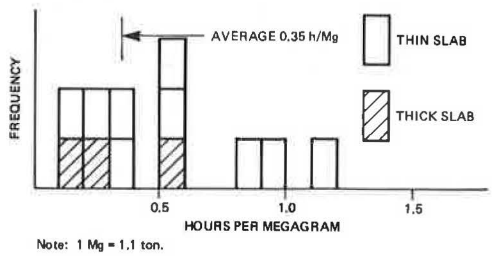


Figure 5. Inspection time on placement of reinforcing steel: footings.

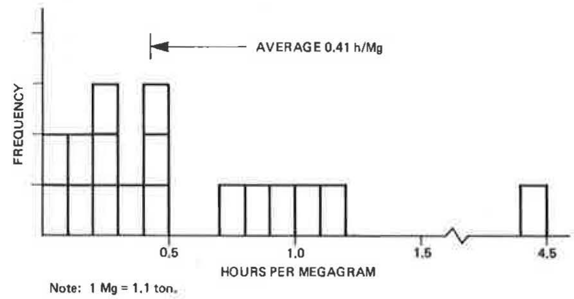


Figure 6. Inspection time on placement of reinforcing steel: substructure.

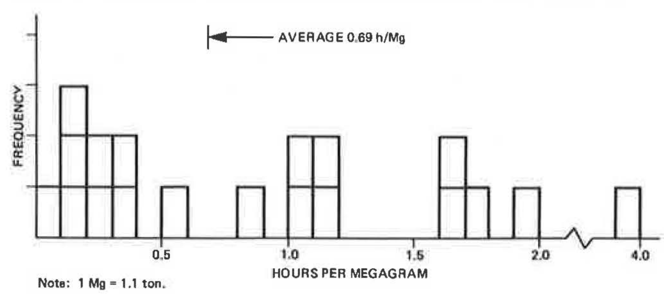


Figure 7. Inspection and testing time on bridge-deck concrete.

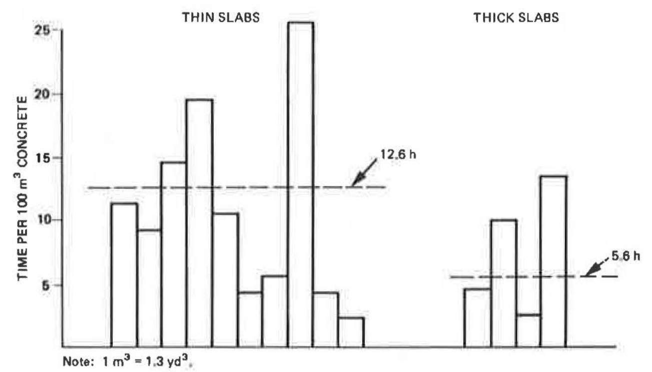


Table 1. Concrete compressive strength data.

Site Number	Concrete Class (MPa)	Concrete Quantity (m^3)	Compressive Strength Tests: Total Number of Sets Made	28-d Tests: Made	28-d Tests: Required[a]	Mean 28-d Strength (MPa)	Percentage Failure (28 d)
22-277	28	69	4	1	2	32.3	0
14-347	28	306 (2 pours)	12	4	6	40.7	0
14-348	28	78	9	3	3	34.5	0
46-124	28	226 (2 pours)	13	6	6	36.8	0
5-219	34	995	32	21	4	33.7	62.0
5-220	34	349	12	7	3	35.3	29.0
37-154	28	57	3	1	2	40.5	0
3-306	34	1970 (4 pours)	48	18	13	41.2	0
36-82	28	253	4	1	3	36.2	0
37-991	28	138	3	2	3	37.0	0
40-02	28	114 (5 pours)	14	5	10	35.3	0
33-291	34	516	16	5	4	40.2	0
37-963	28	240	6	2	3	32.7	0
37-1005	28	445 (2 pours)	10	4	6	40.4	0

Note: 1 MPa = 145 lbf/in^2; 1 m^3 = 1.3 yd^3.

[a]Based on requirements of concrete construction report. For 34-MPa (5000-lbf/in^2) concrete, the frequency of testing may be greater than that specified.

with a mean 28-d strength of 41.2 MPa (5970 lbf/in^2) for 12 separate mixes. Concrete slump, air content, and temperature of the field trial batches were all comparable to anticipated values for the bridge-deck construction. It is difficult to explain at this point why the designated mix proportions used on two bridge decks produced concrete with a compressive strength lower than earlier field trial batches by 5.9 and 7.4 MPa (850 and 1080 lbf/in^2) respectively, but it emphasizes the need for rapid on-site tests and concrete plant controls that will ensure concrete mixes of the specified proportions.

On this contract, 7-d cylinder tests from one site were available before the construction of the deck at the other site, and the test results did indicate a potential problem with the specified 28-d tests. The use of accelerated concrete strength tests, a technique available to MTC inspectors, would have provided additional time to correct the problem before the second deck slab was constructed. As noted elsewhere, the average concrete placement temperature exceeded the specified maximum—certainly a factor contributing to lower strength.

Slump Tests

The required number of slump tests is based on testing at the beginning of the pour, at the point at which concrete cylinders are made, and at subsequent intervals to ensure that the specification requirements are met. In calculating the number of tests required (Table 1), satisfactory testing is assumed for the first five loads of concrete. A slump test has been added to this for each set of cylinders made. Because large numbers of cylinders for other than the 28-d test are often made and cylinders could be made from concrete in the first five loads, the required number of tests may be high in some cases.

An adequate number of slump tests were done on the 14 bridge decks included in the study, although in five cases the number was significantly lower than that required. The mean value of slump tests reported, contract by contract, was in each case greater than the maximum slump of 6.4 cm (2.5 in) specified for bridge decks. In recent years there has been a tendency in construction of bridge decks to accept a 7.6-cm (3-in) slump as a reasonable target value. In view of reinforcement congestion in some decks and the capability of typical bridge-deck finishing machines, the existing specification requirements on the consistency of bridge-deck concrete may be slightly unrealistic.

At site 33-291, a mean slump value of 15.5 cm (6.1 in) is reported for 58 tests (Table 1). The mean slump value of the concrete on this deck was obviously excessive although it is inflated by the inclusion of several very high tests from rejected loads and the concrete was apparently accepted on discharge from the truck at higher than specified values because of drying out on a long conveyor belt transportation system. The air content of the concrete on this particular deck-slab placement was apparently deliberately kept low at least partly to compensate for the effect on concrete strength of the high slump. The mean air content of 4.5 percent from 53 tests indicates that more than half of the concrete represented by the tests was below specification requirements, which is obviously not acceptable for MTC bridge decks.

Air Tests

The required number of air tests is based on the rate of concrete placement and the number of truckloads of concrete delivered (Table 2). The concrete quantity has been used in conjunction with the inspection and testing time to determine if the rate of pour exceeds 38 m^3/h (50 yd^3/h). Because inspection time may be spent on prepour preparation, postpour finishing, and curing and protection procedures, the calculated pour rate may not reflect actual conditions. At placement rates lower than 38 m^3/h, an air test is specified for each load of concrete. At rates in excess of 38 m^3/h, it has been assumed that testing the first five loads will establish satisfactory control and one test every third load thereafter will maintain control. The size of the load also affects the number of air tests required. Because it is difficult to determine when larger loads were used, all calculations are based on 5.3-m^3 (7-yd^3) loads.

The required frequency of air tests stated in the concrete construction report (Figure 1) reflects MTC policy: All concrete in a bridge deck must be properly air entrained to prevent premature deterioration by freezing and thawing in the presence of deicing salts. Size and speed of bridge-deck placement operations require that a substantial effort be made to test the air content of the plastic concrete.

Apart from the one deck example already referred to, reported mean values for air content indicate that most tested concrete is properly air entrained. Only maximum, minimum, average, and number of tests are reported on the concrete construction report. It is apparent that on some very large deck-placement operations the district inspection staff is able to make the sizable number of air tests required. But this is not always the case; in fact, an insufficient number of air tests were made on 11 of the 14 deck slabs. In one example (site 5-220), seven tests for 349 m^3 (457 yd^3) of concrete are totally inadequate regardless of the capacity of the ready-mix truck or the rate of concrete placement.

Temperature Tests

Data given in Table 2 indicate that a sufficient number of temperature tests are being made for the purpose of determining conformity with specification requirements (this characteristic should show little variation in a given concrete-placement operation). For 34-MPa (5000-lbf/in^2) concrete, in conditions in which the maximum concrete temperature requirement was 24°C (75°F) regardless of weather conditions, two of the four decks reported mean concrete temperatures greater than that specified. The study included several deck slabs placed under cold or near-cold weather conditions. At site 36-82, the mean concrete temperature reported from 20 tests [16°C (60° F)] coincided with the minimum specified value. This and lower minimum values reported indicate that some concrete was placed at too low a temperature.

General

Including results on rejected loads of concrete in the test summary appears to defeat the purpose of the concrete construction report. If an evaluation by the ready-mix supplier is required, an additional form tabulating the results of tests on rejected loads would serve this purpose better. The inspector's report would then more accurately indicate the quality of the concrete placed.

During the review of the concrete construction report, several omissions were found, e.g., volume of concrete placed, mean air temperature, number of tests performed. These omissions appear to be a result of carelessness on the part of the inspector compiling the form. Eliminating errors and omissions at the site office by

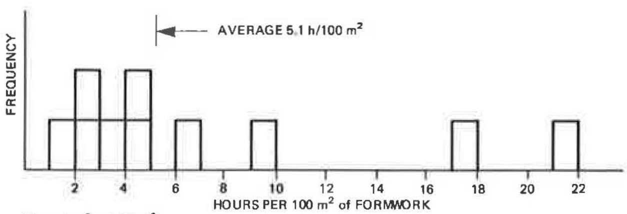
Table 2. Results of concrete slump, air, and temperature tests.

		Slump				Air			Temperature		
Site Number	Quantity of Concrete (m^3)	Tests Made	Tests Required	Tests per 100 m^3 of Concrete	Mean Value (cm)	Tests Made	Tests Required	Mean Value (%)	Tests Made	Tests per 100 m^3 of Concrete	Mean Value (°C)
22-277	69	5	9	7.2	7.6	8	13	6.0	8	11.6	21.7
14-347	306 (2 pours)	35	22	11.4	7.9	39	56	6.0	38	12.4	21.1
14-348	78	13	14	16.7	7.4	13	14	6.0	13	10.1	18.9
46-124	226 (2 pours)	32	23	14.2	7.4	32	42	6.0	32	14.2	21.1
5-219	995	9	37	0.9	8.9	43	65	4.8 to 7.0[a]	40	4.0	25.6
5-220	349	5	17	1.4	7.6	7	25	5.0	8	2.3	24.4
37-154	57	3	8	5.3	7.6	3	11	6.0	1	1.8	17.8
3-306	1970 (4 pours)	69	67	3.5	7.4	233	157	5.5	183	9.3	13.3
36-82	253	12	9	4.7	8.4	14	47	6.2	20	7.9	15.6
37-991	138	13	8	9.4	8.9	13	26	6.3	13	9.4	23.3
40-02	114 (5 pours)	22	22	19.3	6.9	22	22	6.1	15	13.1	22.2
33-291	516	58	21	11.2	15.5	53	36	4.5	48	9.3	18.9
37-963	240	6	11	2.5	7.6	12	18	5.5	8	3.3	23.9
37-1005	445 (2 pours)	18	20	4.0	7.6	18	30	5.8	18	4.0	18.9

Note: 1 m^3 = 1.3 yd^3; 1 cm = 0.39 in; 1°C = (1°F - 32)/1.8.

[a]Mean value was not calculated in the concrete construction report.

Figure 8. Inspection time on bridge-deck formwork.



Note: 1 m^2 = 1.19 yd^2.

requiring that the supervising inspector or the project supervisor check the forms is advisable.

Bridge-Deck Formwork and Falsework

In this paper, falsework is defined as steel or timber shores, bracings, jacks, and foundations used to support the formwork. Formwork is defined as the plywood mold into which the fresh concrete is placed. Falsework is generally not used in the construction of thin-slab decks supported on steel or precast concrete beams.

Erection of formwork or falsework on many structures included in this project had started before the study began. To assess the value of an inspection effort on only a portion of falsework or formwork is practically impossible. Thus, only a limited number of complete falsework erections and formwork operations were recorded.

Figure 8 summarizes the inspection time reported for formwork on 10 bridge decks. The deck slabs varied from small, thin-slab structures with a soffit area of less than 170 m^2 (200 yd^2) to very large, thick-slab, posttensioned structures with more than 20 times this area of formwork.

The average inspection time of 5.1 h/100 m^2 (120 yd^2) of formwork represents 8 inspection days for a typical three-lane bridge 91 m (300 ft) in length. This amount of inspection time probably reflects the absence of other work to be inspected on the structure rather than the demands of the formwork. Inspection times for formwork, as may be expected, varied widely; more time was spent on slab-on-beam decks than on voided posttensioned structures. Camber problems associated with simply supported beams generally require more inspection. Adequate inspection time is clearly available for checking the quality and accuracy of formwork construction.

The following table gives inspection times for the erection of falsework for three structures in two districts (1 m^2 = 1.195 yd^2):

Site Number	Approximate Area of Deck Soffit (m^2)	Inspection Time (h)	Inspection Time per 100 m^2 (h)
3-306	2211	30.5	1.4
3-303	3156	58.5	1.9
33-291	656	23.5	3.6

This small sample indicates that a substantial effort is put into falsework inspection by the district inspector. Although the data are insufficient to support recommendations on falsework inspection, discussions with various project supervisors and inspectors revealed wide differences in the handling of such inspection. Three methods seem to be in use in the province:

1. The district inspector accepts no responsibility for falsework erection and informs the structures office when falsework erection is complete. A structural inspection engineer then approves the falsework configuration.
2. The district inspector checks conformation with the approved falsework drawings during erection. On completion, the structural inspection engineer inspects and approves the falsework.
3. The district inspector assumes full responsibility for ensuring that the falsework arrangement is in agreement with the approved plan. The structural inspection engineer is consulted only on problems that the inspector feels are beyond his or her experience and ability.

There is clearly a need to establish uniform MTC inspection procedures for falsework.

Placement of Bridge-Deck Bearings

Inspection times for bearing placement, given below, indicate reasonable effort and uniformity of inspection for the bridges considered:

Site Number	Number of Bearings	Inspection Time (h)
29-194	24	18
3-304A	10	2

Site Number	Number of Bearings	Inspection Time (h)
3-314	9	1
36-82	70	7
46-199	10	2
33-291	6	1
37-963	80	3.25
30-432	24	3

The fact that inspection time was not reported for bearing placement at three sites seems to indicate that some district inspectors do not concern themselves with this process. The items that require the inspector's attention during bearing placement must be clarified.

Placement of Stressing Cable and Stressing and Grouting Operations

Head-office staff play an important if varied role in on-site inspection of cable placement, stressing, and grouting. The wide variations in inspection time expended by district-level staff reflect somewhat their varying degrees of responsibility. There may be good reasons for a particular inspection arrangement—e.g., a district may not have an inspector experienced in prestressed concrete—but the situation has led to some confusion on site as to what the district inspector is responsible for.

Data on inspection time for these operations, which were available for only four posttensioned deck slabs in three districts, are given below:

Operation	Site Number	Number of Transverse and Longitudinal Cables	Inspection Time (h)	Inspection Time per Cable (h)
Placement of stressing cables	5-219	91	24.5	0.27
	5-220	39	18	0.46
	3-303	126	5	0.04
	33-291	36	12.5	0.35
Stressing	5-219	91	18.5	0.20
	5-220	39	19.5	0.50
	33-291	36	62	1.72
Grouting	5-219	91	12	0.13
	5-220	39	13	0.33
	3-303	126	3	0.02
	33-291	36	27	0.75

The variation in the involvement of the district inspector is evident: Three hours to inspect the grouting of 126 cable ducts on site 3-303 is clearly partial inspection whereas 27 h to inspect 36 cables on site 33-291 may represent continuous inspection of the grouting operation. Cable placement in three decks received close attention from the district inspectors, but at site 3-303 much less inspection time was reported.

The proper checking of cable stressing requires continuous presence of the inspector to ensure that the specified strand elongations and jack pressures are achieved. Thus, inspection time depends directly on the speed and the efficiency of the contractor's operation. Stressing problems occur frequently, and many days of inspection time are sometimes needed. The inspection time spent on the three structures for which stressing operations were reported indicates full-time or almost full-time checking of the contractor's work.

Erection of Concrete Beams

The table below for the four contracts that reported data indicates that a reasonable inspection effort was put into the erection of precast concrete beams:

Site Number	Number of Beams	Inspection Time (h)	Inspection Time per Beam (h)
22-277	5	9.5	1.90
32-136	20	16	0.80
36-82	35	7	0.20
37-963	40	34.25	0.86

Waterproofing of Deck

The inspection times reported for bridge-deck waterproofing generally indicate reasonable uniformity. Since the daily report on bridge inspection was compiled by bridge inspectors, the lack of reported time on paving operations has indicated that this phase of the work is probably handled by the road paving inspector. This may be the best approach provided the road inspector is made aware that the waterproofing membrane must not be damaged during paving. Data for inspection time on waterproofing operations are given below (1 m^2 = 1.2 yd^2):

Site Number	Quantity (m^2)	Inspection Time (h)	Inspection Time per 100 m^2 (h)
22-275	2421	12.5	0.43
22-276	1007	6	0.50
46-124	924	7	0.63
5-219	1091	6	0.46
37-154	455	2	0.37
37-866	962	11	0.96
3-304A	2229	19	0.71
3-314	951	13	1.14
40-02	396	14	2.97

Excavation and Pile-Driving Operation

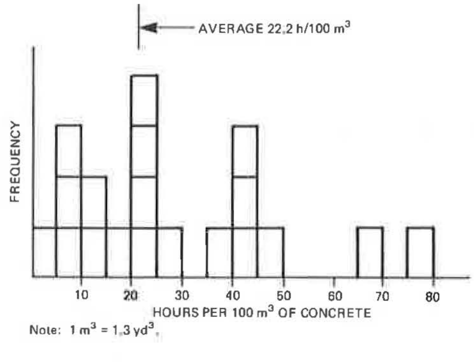
Considerable variation in the site conditions encountered during footing construction contributes to the difficulty of both construction and inspection. Although different soil conditions, dewatering problems, shoring and protection requirements, and location of the footing grade with respect to the original topography all affect footing construction, it is difficult to determine how much each affects the variation in inspection. Detailed examination of the footing excavation and pile-driving operations would probably reveal unique problems and methods of construction, which would often account for differences in inspection time. Although reasonable inspection time seems to have been spent on these operations, it is clear that some personnel assigned to inspect the driving of piles had little experience in such construction.

Footings and Substructure

As anticipated, considerably more inspection time per unit is required for the relatively small footing and substructure pours. Reasons for this include (a) similar prepour preparation and postpour curing and finishing operations distributed over smaller volumes; (b) less accessible sites, the necessary use of less efficient placement techniques, and rate-of-pour restrictions that result in slower concrete placement rates; and (c) problems of specification interpretation and construction techniques that are usually resolved before construction of the deck.

In some cases, trial batches to determine the suitability of 34-MPa (5000-lbf/in^2) bridge-deck concrete mix designs have been placed in footings and substructure sections. Additional inspection and testing have been done in these areas. Test results indicate that most 20.7- and 27.6-MPa (3000- and 4000-lbf/in^2) concrete

Figure 9. Inspection time on formwork of bridge substructure.



placed in footings and substructure conformed to the specification requirements. With few exceptions, the inspection effort and testing programs for concrete in bridge footings and substructure appear to be quite adequate. The following table summarizes this inspection and testing for all sites (1 m^3 = 1.3 yd^3):

Item	Total Concrete (m^3)	Mean Quantity of Concrete Represented by One Test (m^3): 28-d Cylinder	Air Content	Slump	Inspection and Testing Time per 100 m^3 of Concrete (h)
Footings	2194	38	11	13	11.8
Substructure	3609	40	10	14	13.3
Deck slabs	5757	72	11	19	6.3

Bridge Substructure and Formwork

In reviewing these data it is important to realize that many different substructure types are represented and thus wide variations in inspection times can be expected (Figure 9). Because the various structures represent a wide range of forming and inspection problems, it is difficult to make meaningful comparisons of inspection times. However, some contracts report what appears to be an excessive amount of inspection; three sites each with formwork quantities for less than 76 m^3 (100 yd^3) of substructure concrete, completed during the study, indicate an average inspection time of 3 person days for this quantity of formwork. The average formwork inspection time noted—22 h/100 m^3 (131 yd^3) of concrete—represents a much greater inspection effort than that for bridge-deck formwork. This increased inspection time reflects the more complex nature of vertical formwork and the greater area per unit of volume of supported concrete.

Site Visits

Bridge inspectors were asked to record the nature and the duration of site visits of MTC personnel above the inspector level during each bridge-deck concreting operation to determine the help and assistance normally available. District office construction supervisors, construction engineers, and district engineers spent an insignificant amount of time on site during placing of the deck concrete. However, on many contracts, the project supervisor, assistant project supervisor, or a supervising inspector was present, often for most of the working day. The influence of the regional office engineering staff is clear: A supervising inspector from that section was present on most sizable deck concreting operations. Head-office staff were seldom present during the concreting of a bridge deck. If difficult problems arise during concreting operations of a deck slab, the bridge inspector can normally seek help from the project supervisor or the regional supervising concrete inspector, but guidance from MTC construction engineers or quality control engineers can only be obtained by telephone.

CONCLUSIONS AND RECOMMENDATIONS

The quality of a structure depends largely on workmanship in construction. The best of materials and design practice cannot be effective unless the construction is well performed. Competent inspection and adequate testing programs are necessary throughout the progress of the work to ensure a satisfactory finished product.

Assuring adequate structural quality through the prevention of construction faults may be more difficult for concrete bridges than for other civil engineering structures. The whole construction "team" is together for a relatively short period of time. Repetitive operations seldom occur on a bridge as they do on, say, a high-rise structure; on many contracts the contractor is performing a different operation from day to day. There is no "ground floor" on which to sort out construction problems. On some segments of the structure—for example, a posttensioned, voided deck slab—huge quantities of concrete are placed in a single day around a complex arrangement of voids, cables, and reinforcement. Once this concrete production, delivery, placement, and finishing operation gets under way, understandable pressure is put on the on-site staff against making decisions that will impede or halt the work.

This study of current inspection and testing programs, although somewhat limited in scope chiefly because the qualitative aspect of inspection is almost impossible to measure, leads to the following conclusions and recommendations:

1. The wide variation in inspection effort noted on comparable segments of different structures depends partly on the inspector's own interpretation of the scope and the nature of the required inspection effort. A practical bridge-inspection manual that explains in detail what and how structures should be inspected is needed as the basis for a uniform standard of quality. This manual plus appropriate checklists should be the basis for building more accountability into inspection work.
2. Improved manpower planning procedures are necessary to ensure that the required inspection effort and concrete testing programs are carried out. Further studies are needed to determine the optimum time required to complete inspection and testing programs on the various segments of a structure.
3. Site drawings, in particular those detailing reinforcing bars, are not easily interpreted by many inspectors. Simpler drawings are needed.
4. More use should be made of accelerated curing procedures for concrete test cylinders, particularly for prepour trial batches and laboratory mix designs. There is also an urgent need to develop reliable, practical tests for the rapid analysis of fresh concrete. Otherwise, extra controls such as mandatory printout of concrete proportions are required at ready-mix plants.
5. Current quality assurance problems in materials and workmanship can be minimized by strict adherence to existing specifications. There is a need to develop a broad attitude among the construction staff of the owner and the contractor that specifications, testing procedures, and other construction controls are something more than guidelines—that such documents must be the basis of construction, testing, and acceptance of the work.

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Development of Price-Adjustment Systems for Statistically Based Highway Construction Specifications

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This paper presents a methodology that can be used to develop price-adjustment systems for use in statistically based highway construction specifications. Three approaches are proposed for the development of a price-adjustment system: (a) the serviceability approach, (b) the cost of production approach, and (c) the operating characteristic curve approach. The three approaches are discussed and compared, and their most appropriate applications are recommended. A fourth approach, the cost of quality control approach, is also discussed but is not fully developed because of the limited cost data available.

The advent in recent years of statistically based highway construction specifications has resulted in many highway agencies incorporating various systems of price adjustment into their specifications so as to take into consideration the degree to which the finished product conforms to requirements. A price-adjustment system forms an important part of the specifications, but few guidelines have been made available for the development of an equitable system. As a result, many of the initial price-adjustment systems have been developed primarily through judgment, and further adjustments that have been made have been dictated by experience under actual contract conditions. Unfortunately, such an evolution often occurs at the expense of one or more of the contractual parties that have agreed to proceed under the specifications. In addition, because the desired operating level may not have been decided in advance, it becomes difficult to determine objectively just when an equitable price-adjustment system has been attained.

This paper presents a methodology that may be used by highway agencies to establish effective price-adjustment systems for their statistically based specifications. The methodology can be applied in conjunction with the development and the implementation of a new set of specifications, or it can be applied as a means of establishing the adequacy of the price-adjustment system currently used by the agency.

SPECIFICATIONS

The suitability or effectiveness of a price-adjustment schedule, viewed by itself, is impossible to determine. (Here, a price-adjustment schedule is defined as a tabular, graphical, or formular representation that establishes, for a given material characteristic, the payment factors associated with estimated quality levels of that characteristic; a price-adjustment system, on the other hand, is composed of the various schedules that together are used to determine the contractor's final payment for a given material.) The schedule is but a small part of the specifications, and its merit can only be determined by viewing it in the entire context of the specifications.

Welborn (1), in a report summarizing the statistically based bituminous concrete specifications of several agencies, points out that many differences exist in the specifications in the following areas: responsibilities assigned to the agency and to the contractor, mix design criteria, quality control tolerances, and acceptance plans (specifically, acceptance quality characteristics, basis for acceptance, lot size, and number of samples). When the implementation of statistically based specifications for other materials (e.g., portland cement concrete paving, structural concrete, and soils) becomes as extensive as it is for bituminous concrete today, these specifications too will probably vary widely from agency to agency.

A price-adjustment system must be tailor-made for the specifications in which it will be contained. Therefore, a necessary first step in the design of a price-adjustment system is that of examining and understanding the specifications. Several questions about the agency's statistically based specifications must be answered before a price-adjustment system can be developed:

1. What are the quality characteristics that are to be used as a basis for acceptance? What properties do these characteristics measure? Are all the desired properties taken into account? How are the characteristics related?
2. What procedures are to be used to determine the degree of acceptability of material submitted by the contractor?
3. How have the acceptance limits been determined? What is the relation between the acceptance limits and the statistical parameters of the material?
4. What risks are being taken by the contractor? What risks are being taken by the highway agency? What are the operating characteristic curves of the acceptance plans? (This set of questions applies only to the case in which the adequacy of a price-adjustment system already contained in the specifications is to be determined.)

Acceptance Quality Characteristics

Each quality characteristic that is to be used for acceptance must provide a measure of one or more of the significant properties that are desired in a completed product. Not all of the desired properties must be taken into account by the acceptance quality characteristics; some of them can be adequately controlled by requirements or guidelines placed on the contractor's daily process-control activities. But all properties that are considered desirable must be adequately controlled either through acceptance or process-control testing of the quality characteristics.

Because of the large number of quality characteristics that can be used in various combinations as acceptance characteristics, the number of possible price-adjustment systems is also large. In other words, once a decision is made as to what properties are to be controlled, the designer has a number of quality characteristics from which to choose in establishing the basis for acceptance. In addition, as will be discussed later in this paper, several rational approaches for the establishment of a price-adjustment schedule for a given acceptance characteristic may be available to the designer. The choice of acceptance characteristics that are to be used should therefore take into account the effects that will be created by the implementation of the overall price-adjustment system. Two of the more important effects that should be considered are (a) the interdependency among acceptance quality characteristics (which may need to be minimized) and (b) effects on the relation among the highway agency, the contractor, and the material supplier (a relation that should be optimized).

Acceptance Plans

An acceptance plan is generally defined as an agreed-on method of making measurements for the purpose of determining the acceptability of a lot of material or construction. Although many different statistically based acceptance plans are currently in use, they should all have a common denominator in that they should each define

1. Lot size,
2. Number of samples or measurements,
3. Sampling or measurement procedure,
4. Point(s) of sampling or measurement,
5. Numerical value of acceptance or specification limit(s), and
6. Method of evaluating acceptability.

Each of these items affects the subsequent development of individual price-adjustment schedules. For example, if the number of measurements is small, the uncertainty associated with an estimate of lot quality will be greater than if the number of measurements is large. On the other hand, if the specification limits are wide (in the case where both a lower and an upper limit exist), more material can be expected to fall within those limits and the price-adjustment schedule would appear to be strict in comparison to one based on narrow specification limits. It is important, therefore, that each of the above items be known before an attempt is made at establishing a price-adjustment schedule.

Acceptance Limits and Statistical Parameters

There are basically two approaches for establishing realistic acceptance limits (2, 3). The first approach involves assigning numerical values based solely on engineering requirements. This means that the permissible range for the specific quality characteristic must be determined. It may be possible in some cases to derive the permissible range by means of a theoretical or experimental procedure, but this is often difficult. A more practical approach for setting realistic acceptance limits is to measure the properties (statistical parameters) whose existing construction is satisfactory. Limits are then established so that the material quality will at least stay at the present acceptable level. This is the approach used by most highway agencies that have adopted statistically based specifications.

There are two generally recognized methods for determining the statistical parameters of acceptable construction: (a) Statistical analyses can be performed on historical data, or (b) a separate sampling system can be installed to obtain data under a controlled procedure. If statistical analyses are to be performed on historical data, caution must be exercised in making sure that the data were actually obtained by random sampling. Because few historical data have been collected by random sampling, many highway agencies have introduced separate sampling systems on typical projects under construction. If the data collection system is properly controlled, this method will result in more up-to-date and reliable data.

When the statistical parameters that typify existing construction are known for a given acceptance characteristic, they can be related to the acceptance limit(s) to establish a reference by means of which various levels of quality can be compared. For example, if the typical relative density of bituminous concrete pavement is 100 percent with a standard deviation of 2 percent, then, by calculating the appropriate area under a normal distribution, 97.72 percent of the tests on submitted lots that are of a quality equivalent to that of existing construction can be expected to result in a relative density greater than 96 percent.

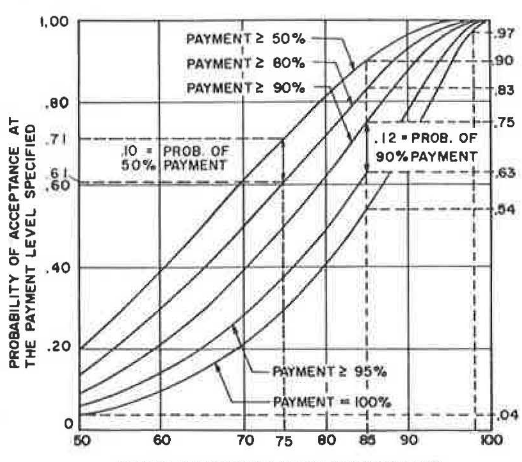
Operating Characteristic Curves

The most common technique used in evaluating the risks that are part of any acceptance plan is the development of operating characteristic (OC) curves. If the adequacy of a price-adjustment system that already exists in the specifications must be checked, it is important to investigate the OC curves.

In its most basic form, an OC curve relates the quality of a lot to the probability of its acceptance. However, when a price-adjustment schedule is part of the acceptance plan, the probability of acceptance associated with various levels of quality does not provide sufficient insight into the risks involved in the acceptance plan. In such a case, it is far more meaningful to think not only in terms of the probability of acceptance of a lot but also in terms of the probability of a lot being assigned any one of the specified payment reductions (or increases). A set of OC curves should thus be drawn to relate the quality of a lot to the probability of its acceptance at each of the specified payment levels. A discussion of the development of such sets of OC curves can be found elsewhere (4).

An example of the type of information that can be provided by OC curves is shown in Figure 1. The set of OC curves shown in the figure was developed from the price-adjustment schedule used by the Pennsylvania Department of Transportation (PennDOT) in its acceptance plan for the density of bituminous concrete pavement. The price-adjustment schedule is given below. For acceptance purposes, a lot of material is divided into five sublots (i.e., n = 5).

Figure 1. Operating characteristic curves for PennDOT acceptance plan for bituminous concrete pavement density.



Estimated Percentage of Lot Above Density Limit	Percentage of Contract Price to Be Paid	Estimated Percentage of Lot Above Density Limit	Percentage of Contract Price to Be Paid
85 to 100	100	70 to 74	80
80 to 84	95	65 to 69	50
75 to 79	90	<65	–

According to the PennDOT schedule, when <65 percent of a lot is estimated to be above the density limit, the lot shall be removed and replaced to meet specification requirements as ordered by the engineer. Alternatively, the contractor and the engineer may agree in writing that, for practical purposes, the lot should be removed and should be paid for at 50 percent of contract unit price.

From Figure 1, it can be seen that a bituminous concrete lot that actually has 85 percent of material above the specified density limit will be accepted 54 percent of the time at full payment, 63 percent of the time at a payment equal to or greater than 95 percent, 75 percent of the time at a payment equal to or greater than 90 percent, and so on. The probability of the lot receiving exactly 90 percent payment, for instance, is found by subtracting the probability of receiving a payment equal to or greater than 95 percent from the probability of receiving a payment equal to or greater than 90 percent.

In Pennsylvania, when statistically based bituminous concrete specifications were being developed, it was determined that a lot having 98 percent of material above the specified density limit typified the level of quality that had been incorporated in acceptable construction in Pennsylvania in the past. Figure 1 shows that lots submitted at this typical quality level will be accepted at full payment approximately 97 percent of the time and will practically never be rejected. Therefore, if the contractor submits the type of material that has normally been submitted in Pennsylvania, the chances are very good that he or she will receive full payment.

The information provided by Figure 1 can also be used to plot the relation between the quality of the contractor's material and the expected payment (or average payment over the long run). The development of this relation for the Pennsylvania density schedule is given in Table 1, and the resulting curve is shown in Figure 2. It can be seen that a contractor who is providing material of a quality typically found in Pennsylvania construction (i.e., lots in which 98 percent of the material is above the specified density limit) can expect an average payment of 99.7 percent. At the other end of the scale, a contractor who is providing material that contains 50 percent material above the limit, for instance, can expect an average payment of 35.6 percent. Other expected payments for various quality levels can be obtained in the same way. The expected payments in this example may be misleading, however, especially at low quality levels, unless the assumptions made in Table 1 regarding the 50 percent payment level are understood. A more detailed discussion of this particular expected-payment curve can be found elsewhere (5).

The OC curves and the curve of expected payment can be used as a basis for determining whether the acceptance plan (with the price-adjustment schedule) is reasonable. Because the answer depends largely on the philosophy of the highway agency, several questions should be posed in analyzing the curves: Is the highway agency satisfied with the performance of roads that have been constructed at the typical quality level? Does the highway agency want the same quality or a higher or lower quality than that provided in the past? If higher (or lower), how much higher (or lower)? What does the highway agency consider to be unacceptable material in relation to the quality characteristic in question? What does the highway agency consider to be good material? What risks are the highway agency and the industry willing to take?

If it is determined that the price-adjustment schedule is not reasonable, then one or more of the following changes can be made:

1. Change the sample size.
2. Loosen or tighten the acceptance or specification limit(s).
3. Increase or decrease payment for a given quality level.
4. Increase or decrease the number of payment levels.

The exact modification to be made depends on the nature of the inadequacy. After the modification has been made, new OC curves and a new curve of expected payment should be drawn and evaluated.

PRICE-ADJUSTMENT APPROACHES

Serviceability

Perhaps the most logical basis for establishing a defensible price-adjustment schedule is the selection of an adjustment in unit price that is commensurate with the expected serviceability or performance of the furnished product. If the acceptance characteristic can be strongly correlated with the serviceability or the performance of the pavement, it may be desirable to relate the payment reductions to the expected losses in pavement performance. For example, assume that it is undesirable to accept bituminous concrete that will result in a pavement with a service life that is less than 75 percent of the design life. If pavement thickness can be shown to be directly correlated with service life, then the value of thickness that is to be considered unacceptable can be determined. Thus, if a 12.7-mm (0.5-in) thickness deficiency reduces service life by 25 percent, bituminous concrete pavement that has a deficiency of 12.7 mm or more should not be accepted. Furthermore, for pavements that are deficient by less than 12.7 mm, a price-adjustment factor related to the expected percentage of design life may be used.

Several state highway agencies have used a serviceability approach for one or more of their price-adjustment schedules. Two examples are the thickness schedule for bituminous concrete pavements of the New Jersey Department of Transportation (6) and the smoothness schedule for portland cement concrete pavements of the New York State Department of Transportation (7).

Developing a price-adjustment schedule based on serviceability is, however, not always possible, for two reasons: (a) The desired precise correlation between the quality characteristic and pavement performance does not exist in many cases, and (b) performance data relating the quality characteristic to the maintenance-free life of pavements are often not available. Therefore, although the serviceability approach is highly desirable, any methodology for the development of a price-adjustment system should also consider alternative approaches.

Cost of Production

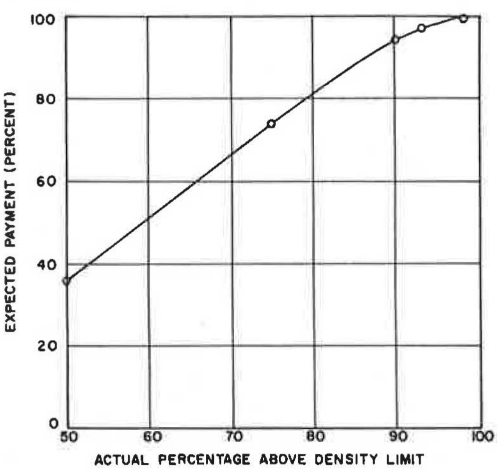
The cost of production approach is limited in use to only a few quality characteristics. In this approach, the payment reduction should be greater than the reduction in cost that results when lower quality material is being produced. For example, if the design thickness of a

Table 1. Calculations of expected payment for PennDOT acceptance plan for bituminous concrete pavement density.

Percentage of Lot Above Density Limit	Percentage Probability of Receiving Payment of 100 Percent	95 Percent	90 Percent	80 Percent	50 Percent[a]	0 Percent	Expected Payment[b] (¢)
50	0.04	0.02	0.03	0.05	0.06 + 0.40	0.40	4.0 + 1.9 + 2.7 + 4.0 + 23.0 + 0 = 35.6
75	0.30	0.08	0.12	0.11	0.10 + 0.26	0.03	30.0 + 7.6 + 10.8 + 8.8 + 18.0 + 0 = 75.2
90	0.70	0.09	0.09	0.05	0.03 + 0.04	0	70.0 + 8.6 + 8.1 + 4.0 + 3.5 + 0 = 94.2
93	0.82	0.07	0.06	0.02	0.02 + 0.01	0	82.0 + 6.6 + 5.4 + 1.6 + 1.5 + 0 = 97.1
98	0.97	0.01	0.01	0.01	0	0	97.0 + 1.0 + 0.9 + 0.8 + 0 + 0 = 99.7

[a] The probability of receiving 50 percent payment is the sum of two probabilities: the probability of obtaining a quality estimate that requires 50 percent payment and the probability of receiving 50 percent payment when removal is an alternative. To determine the second probability, it is assumed that material that is actually 50 percent above the density limit will be paid for at half price in 50 percent of the cases where a decision must be made between removal and acceptance; material that is 75 percent above the density limit will be paid for at half price in 90 percent of such cases; and material that is more than 75 percent above the density limit will be paid for at half price in 100 percent of such cases.
[b] Σ(payment) x (probability of receiving payment).

Figure 2. Expected payment curve for PennDOT acceptance plan for bituminous concrete pavement density.



bituminous concrete wearing course is 50.8 mm (2 in) and the average thickness attained is found to be 43.2 mm (1.7 in), the contractor can be said to have incorporated only 85 percent [(1.7 ÷ 2)100] of the material required to do the work. The cost of production is thus assumed to be 15 percent less than it would have been had the required thickness been provided. The contractor's payment should therefore be reduced by 15 percent of the cost of the material. If the payment reduction is any less than 15 percent of the cost of the material, it may be more beneficial for the contractor to place deficient thickness and accept the payment reduction. To make certain that the contractor will not be tempted to place a deficient thickness, the price reduction should be greater than 15 percent of the cost of the material.

In most cases, however, the cost of the material is actually unknown to the purchasing agency. The agency knows only the item bid price, which includes the cost of labor, equipment, and overhead as well as the cost of the material. Although the cost of material has decreased for the contractor who is producing material of deficient thickness, the other costs in the item bid price may have remained essentially constant. If it can be assumed that the cost of material in the example given above is half of the total cost, then the appropriate price reduction should be at least 7.5 percent of the item bid price. A factor of one-half seems to be reasonable for bituminous concrete wearing course (8).

The price reduction thus depends on two ratios:

1. The fractional deficiency ratio (i.e., the deficiency divided by the required amount, or 0.3 ÷ 2.0 = 0.15 in the example); and
2. The ratio of the cost of the material to the item bid price (0.5 in the example).

If these two ratios are multiplied, a minimum price reduction of 0.075 or 7.5 percent of the item bid price is obtained.

Besides being used for the thickness acceptance characteristic as discussed above, the cost of production approach can also be used in bituminous concrete pavements for a characteristic such as asphalt content and in portland cement concrete pavements for thickness and the quantity of portland cement used (bags per cubic meter). It should be noted, however, that the cost of production approach can only be used for acceptance characteristics that have a single (lower) acceptance limit. In the case of asphalt content, the upper acceptance limit would have to be eliminated and replaced in the specifications by an additional acceptance characteristic such as skid resistance, which would function to prevent an excess in asphalt content.

Operating Characteristic Curve

It was indicated earlier that OC curves and curves of expected payment can be used to determine the desirability of a given acceptance procedure and its related price-adjustment schedule. These curves may also be used directly during the course of the development of a price-adjustment schedule. This approach is referred to here as the OC curve approach.

The OC curve approach can be used to develop the entire acceptance plan or the price-adjustment portion of the plan only. If it is used in the development of the entire acceptance plan, two points must be defined on the OC curve graph. In other words, the agency must establish the probability for accepting (or rejecting) two different levels of quality for material. The two quality levels should preferably not be spaced too close to each other. In defining the two points, it is easiest for the agency to think in terms of the probability desired for rejecting material that has been designated as good and the probability desired for accepting material that has been designated as poor. Only one OC curve can pass through the two points—the curve that identifies the sample size and the acceptance limit(s) that are to be used.

Once the sample size and the acceptance limit(s) have been established (whether or not OC curves have been considered to that point), the agency can use OC curves and curves of expected payment to develop a reasonable schedule. A trial schedule should be devised. In the first step, the price reductions can either be designed to increase sharply with decreasing quality or they can be designed to increase linearly with decreasing quality.

Next, OC curves (for the various levels of payment) and curves of expected payment can be drawn. If both the agency and the contractors are pleased with the curves, then the schedule can be incorporated into the specifications. If not, then either the price-adjustment schedule or the acceptance plan must be modified by means of the changes suggested earlier in this paper (changing the sample size, loosening or tightening the acceptance or specification limits, increasing or decreasing payment for a given quality level, and increasing or decreasing the number of payment levels). After the appropriate modification has been made, new OC curves and a new curve of expected payment are drawn and the process is repeated until both contractual parties are satisfied.

Cost of Quality Control

The development of a price-adjustment system that is based on the contractor's cost of quality control was also investigated during the course of this research (4, 5, 9). A price-adjustment system based on the cost of quality control would logically relate the reduction in payment for inferior material to the contractor's reduced spending on quality control. Inherent in the development of such a system is the assumption that there is a direct relation between the contractor's cost of quality control and the quality of the resulting construction.

In an effort to determine whether this approach could be implemented, an attempt was made to gather the data necessary to determine the relation that existed between what a contractor spent on quality control of a project and the resulting quality of that project. This relation seemed to be obscured by variable project conditions—such as weather and the distance from the plant to the project—that influence quality but cannot readily be associated with the cost of quality control. Furthermore, cost data for the Pennsylvania situation were unavailable on a project-by-project basis. Nonetheless, this approach has a certain intuitive appeal and may be found to be workable as more cost data become available.

DEVELOPING AND IMPLEMENTING A PRICE-ADJUSTMENT SYSTEM

Factors for Consideration

Incorporating price-adjustment tables or formulas into specifications is not a simple matter. Even if a rational method could be used to determine price adjustments, many issues must still be considered before the payment factors can be declared acceptable. For instance, the price-adjustment system must be consistent with the highway agency's philosophy regarding the level of quality desired in future construction. In addition, the effect of combining individual price-adjustment schedules to form a price-adjustment system must be considered.

The preceding discussion of the various approaches to price adjustment may have implied that a price-adjustment schedule that involves only judgment is undesirable, but this is not necessarily true. A rational schedule, of course, would be ideal, but it is not essential. DiCocco (10) states that the main function of reduced payment is to provide an alternative means of enforcing the contractual agreement. An entirely rational price-adjustment schedule is not always required to enforce the contractual agreement. Any arbitrary price-adjustment schedule, as long as it is reasonable, can serve that purpose.

Because the four price-adjustment approaches discussed here are based on sound mathematical procedures, they each represent a rational (and therefore defensible) method of determining price adjustments. But when two or more of these approaches can be applied to the same quality characteristic, they will probably not result in the same price adjustment. In the case of pavement thickness, for example, the serviceability, cost of production, and OC curve approaches are all applicable (the cost of quality control approach may also be applicable, but it is disregarded here because it could not be fully developed at the time the research was performed). The question that logically follows is, Which approach should be applied when two or more approaches are possible?

The authors believe that all possible approaches should be considered for a given acceptance characteristic. In the pavement-thickness example mentioned above, a comparison should first be made between the serviceability approach and the cost of production approach. If the price reduction based on the cost of production turns out to be larger than the price reduction based on serviceability, perhaps it is in the interest of the highway agency to use the cost of production approach because the smaller price reduction for serviceability would allow the contractor to benefit by producing deficient material. But it may also be argued that, if at all possible, the highway agency should be primarily concerned with the serviceability approach. In this case, if a contractor can provide the required serviceability at a reduced cost of production, it is an incentive for him or her to do so.

The selection of either the serviceability or the cost of production approach, however, does not ensure that the resulting schedule will be either reasonable or readily acceptable by the industry. It is therefore recommended that the OC curve approach also be adopted (and that either the serviceability or the cost of production approach be used as a trial schedule). Although the OC curve approach cannot be said to be as rational as the other approaches, its primary benefit is that it provides a means by which the highway agency can define the quality level the agency desires. It is also more likely to yield a schedule that will be accepted by all parties involved.

Methodology

In view of the material that has been discussed, the following methodology is suggested for the development of a price-adjustment system:

1. Acceptance characteristics should be chosen so as to ensure that the desirable properties of the material are evaluated. The combination of acceptance characteristics and required process-control characteristics should be inclusive enough to provide adequate protection to the highway agency.
2. Individual price-adjustment schedules should be devised by considering each acceptance characteristic separately.
3. The ideal schedule is probably one that assigns a payment reduction equal to the economic consequences of reduced quality. If the acceptance characteristic being considered correlates strongly with pavement serviceability or performance, such a schedule can be developed. If not, the highway agency should consider the cost of production approach.
4. If the price-adjustment schedule is developed on the basis of serviceability, a schedule based on the cost of production method should also be developed if possible. After the results of both methods are compared, it may be beneficial from the highway agency's point of view to use the cost of production method whenever it

results in a larger price reduction. OC curves and curves of expected payment should also be developed as a check to ensure that the proposed schedule is reasonable and meets the desires of the agency.

5. If neither the serviceability method nor the cost of production method applies to the acceptance characteristic in question, the trial-and-error OC curve approach should be used.

6. The overall effect of combining the individual schedules should be considered. Adjustments should be made to the individual schedules if necessary. When adjustments are made, OC curves and curves of expected payment should be redrawn.

7. The entire system should be carefully monitored under contract conditions. Data related to cost, serviceability, and quality should be gathered continuously as a check on the design assumptions of the price-adjustment system. The effects on the relation among the highway agency, the contractor, and the material supplier should also be examined.

Implementation

Although a properly developed price-adjustment system can do much to improve the relation between the highway agency and the contractor, it must be kept in mind that a price-adjustment system is only as good as the specifications that encompass it. If the specifications are confusing or ambiguous, the price-adjustment system cannot hope to provide equitable treatment for all contractors. If the specifications are not uniformly enforced, the price-adjustment system cannot provide impartiality. For this reason, uniform interpretation and enforcement of statistically based specifications containing price-adjustment systems are more critical than they are under the conventional materials-and-methods type of specifications. The implementation of statistically based specifications containing price-adjustment systems should force highway agencies to make improvements that have been overlooked in the past.

The question of which party is to perform the required acceptance tests during the implementation of statistically based specifications is often debated. Some people reason that, because the highway agency is responsible for acceptance of the product, the highway agency should perform its own acceptance tests. This is sound reasoning only when the highway agency uses a central laboratory to perform all acceptance tests. If individual field inspectors perform the acceptance tests, the same submitted material will not necessarily have the same probability of acceptance. Testing error has been found to be the major source of variation in many quality characteristics (2, 11). For that reason, the determination of payment should not depend on the abilities of highway-agency inspectors. The contractor's technician should be allowed to perform the acceptance tests so that any penalty incurred because of testing errors will be the fault of the contractor.

Another obstacle to the implementation of statistically based specifications seems to be the fear of increased legal complications resulting from the enforcement of the price-adjustment system. Conflicts will undoubtedly arise not only between the highway agency and the contractor but also between the contractor and the subcontractors and suppliers. A contractor who is assigned a sizable price reduction is likely to contest it. The apparent inferior quality may well not be the contractor's fault. The contractor's point of view must be considered: Is the price-adjustment system placing the contractor in a situation that will create legal entanglements?

Clearly, better and more binding contractual agreements will have to be developed between contractors and suppliers. However, the type of price adjustment can also do much to improve the situation. Acceptance characteristics that can potentially create conflicts should not be used. For example, the PennDOT acceptance characteristics for bituminous concrete are asphalt content, density, thickness, and smoothness. This system minimizes legal complications because the responsibility for the control of each characteristic essentially falls on only one party. Asphalt content is clearly the responsibility of the producer. Thickness, smoothness, and density are primarily the responsibility of the contractor.

A price-adjustment system cannot be successful unless all parties are satisfied. The recommendations and the comments of contractors and suppliers should be investigated. The system that is initially implemented will probably have limitations that will only be discovered under contract conditions. Because of constantly changing construction conditions, the effectiveness of the price-adjustment system is also subject to change. The price-adjustment system will need to be reevaluated periodically because of new developments in testing and construction procedures and changes in prices. The highway agency must therefore be flexible enough to correct inequities that become apparent during implementation of the price-adjustment system.

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The contents of this paper reflect our views, and we are responsible for the facts and the accuracy of the data presented. The contents do not necessarily reflect the official views or policies of the Pennsylvania Department of Transportation or the Federal Highway Administration. This paper does not constitute a standard, specification, or regulation.

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Statistical Quality Assurance in Highway Engineering in South Africa

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This paper examines the first large-scale application of process and acceptance control plans to a major road construction project in South Africa. The acceptance control scheme used and its background are outlined, and certain controversial features of the scheme are discussed. The variability of typical South African construction materials and processes is indicated. Some economic consequences of the use of the plan are also reported. Because the average quality of the work was well above the minimum standard required, a fully conclusive assessment of the financial advantages or disadvantages of the scheme is not possible. Because of this, a comparison was made between the acceptance decisions of the specific scheme discussed, and those of the engineering judgment approach. It is concluded that the use of the statistical method leads to more consistent interpretation of results, and the continued use of this scheme on highway projects is recommended.

Highway authorities in South Africa, like authorities in other countries, have for several years been concerned about the quality of construction work, particularly about the application of uniform standards of judgment to the acceptance or rejection of such work. The Division of National Roads of the South Africa Department of Transport—aware that differences existed in materials, construction processes, and contractors and that supervisory engineers often applied different criteria to work that did not strictly conform to specifications—decided in 1972 to incorporate statistical principles into certain road contracts. This was done so that the properties of engineering materials could be rationally defined and to assist in providing uniform criteria of judgment for acceptance or rejection decisions. The department primarily wanted to give economic encouragement to contractors who delivered uniform construction work and to reduce as far as possible the risk of having basically acceptable work rejected.

The theory and the design of the acceptance control plans adopted for use by the Division of National Roads are fully described by Kühn, Mitchell, and Smith (1). A document that explains the system and the method of implementation and also contains a typical specification is available (2). In conjunction with the Natal Roads Department, the division decided that the first major contract on which statistically oriented acceptance control procedures would be used in judging certain parameters would be a contract encompassing a portion of National Route 2 on the Durban Outer Ring Road. This $8 million contract consisted of the construction of 4.8 km of six-lane double carriageway freeway including 1.3 million m^3 of earthworks, 66 Gg of asphaltic concrete, and 121 000 m^3 of base and subbase layers.

ACCEPTANCE CONTROL IN SOUTH AFRICA

The decision to use a statistically oriented acceptance control procedure for the National Route 2 contract did not originally meet with enthusiasm from all the road engineers involved in the project. This was not surprising for two reasons: (a) Nearly every change in existing quality control procedures in road construction had met with the same reaction, and (b) most engineers do not possess an in-depth knowledge of the principles of statistics. Statistical methods are helpful, however, in solving problems of quality control and acceptance of completed work in road engineering, provided they are properly applied. It is anticipated that such procedures will come to be recognized as a definite improvement on past methods and that they will become standard practice for quality control.

In the early stages of the development of road construction in South Africa, many of the current specifications and tests were developed on an empirical basis and were largely method-type procedures both to guide contractors and to provide parameters for quality acceptance. One of the major functions of a specification was to convey technical instructions to both the contractor and the resident engineer.

It is hoped that, because of the accent on technological improvements in the contracting industry, it will soon be possible to specify only the significant characteristics of the end product in terms of measurable parameters and use a rational acceptance control procedure for the acceptance of the work. Before this goal can be reached many problems must be overcome, the most important of which is changing the practice of ill-defined joint control of both processes and acceptance by contractor and engineer to separate control of processes by the contractor and control of acceptance by the engineer.

The statistical procedures now being introduced into the South African road construction industry are not new concepts; similar procedures have been successfully applied in road construction in North America for many

Figure 1. Normal distributions of populations with a lower specification limit.

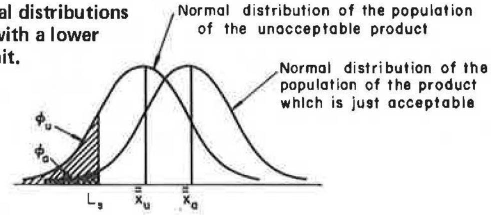
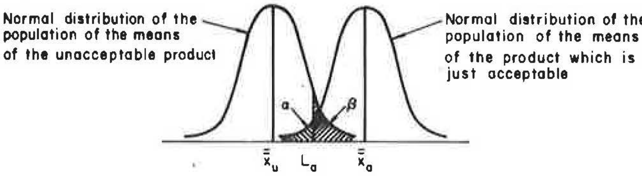


Figure 2. Normal distributions of populations of the means.



years. To quote from DeYoung (3),

> The first application of statistical quality control in American industry was made by Dr. Walter Shewhart of the Bell Telephone Laboratories during the 1920's. The development of this new methodology and its acceptance by other industries in the United States was very slow until the advent of World War II.
>
> About this time a new concept, statistical decision, was introduced. The Department of Defense, which was faced with a massive procurement program, recognized the utility of this technology and pioneered the general development and application of statistical-based process control and acceptance concepts to industrial products. This effort stimulated its application to a great variety of industrial products.
>
> The rather startling experiences with construction control at the AASHO Road Test and the institution by the Bureau of Public Roads and state highway departments of a "record sampling program" are considered to have generated the first real active effort by highway engineers to explore the use of statistical concepts as a tool for the solutions of many quality assurance problems. Dr. Robert G. Baker, former Director of the Office of Research and Development of the Bureau of Public Roads, is cited as one of those who recognized its power and aggressively promoted its use. He believed this development should contribute as much to our ability as engineers as did the advancement of the elastic theory in the 19th century and the use of computers and new construction equipment in the 20th century.

The South African procedures, of course, incorporate values for factors that have reflected construction quality in that country over the past few years.

RATIONALE OF ACCEPTANCE CONTROL PLAN

No materials and construction processes are absolutely homogeneous; i.e., they all vary according to some type of distribution (usually approximating a normal distribution). Therefore, it must be accepted that a limited number of sample test results will yield a mean and a standard deviation, which may differ from the true population mean and standard deviation. In addition, it is obviously impractical to test all possible samples that can be drawn from a population. To complicate matters even further, the test results may possibly belong to a population that is either acceptable or unacceptable in terms of the specification. The normal distributions of product populations with a lower specification limit are shown in Figure 1. In the figure,

L_s = specification limit,
ϕ_a = percentage of the material below L_s for a product that is just acceptable,
ϕ_u = percentage of the material below L_s for a product that is unacceptable,
$\bar{\bar{x}}_a$ = population mean of a product that is just acceptable in terms of the specification, and
$\bar{\bar{x}}_u$ = population mean of a product that is totally unacceptable in terms of the specification.

Because the mean value of the test results ($\bar{x}_n$) is used to assess the material, this value should be compared with the population of the means of both the acceptable and unacceptable products that have a standard deviation equal to ($\sigma/\sqrt{n}$), where σ is the true standard deviation of the population and n is the number of samples. In practice the value of the sample standard deviation (S) is used for σ because it is the best available estimated value (Figure 2).

From Figure 2 it is evident that, if the mean test result ($\bar{x}_n$) is compared with an acceptance limit (L_a) and the product is rejected because $\bar{x}_n$ is just smaller than L_a, the contractor runs a risk of α percent of being wrongly rejected because there is still an α percent probability that the true mean of the population is equal to $\bar{\bar{x}}_a$. On the other hand, if the product is accepted because the value of $\bar{x}_n$ is exactly equal to L_a, the client runs a risk of β percent of accepting an unacceptable product because there is still a β percent probability that the true mean of the population is equal to $\bar{\bar{x}}_u$. A perfect acceptance plan would be one in which these two risks, α and β, were zero. From a practical point of view this is impossible, and effort is best directed toward making these two values as low as possible and at the same time maintaining practical limits for the quality of the work.

Furthermore, if the value of $\bar{x}_n$ is lower than L_a and tends toward $\bar{\bar{x}}_u$, it is clear that the contractor's risk (α) of being wrongly rejected decreases but the client's risk (β) of accepting an unacceptable product increases. It is considered equitable, therefore, that lower payment should be made for this material if it is accepted by the client; this was the case with some of the asphaltic concrete material in the road construction contract discussed here.

The values for ϕ_a have been calculated from past as-constructed data to ensure that the standard of construction remains relatively stable in terms of previous specifications (1). Substantial information on the variability of material and construction is available from overseas investigations, particularly those conducted in the United States. Some information on this has also resulted from analyses of road construction data in South Africa. This information was verified and extended by analyzing data from some recently completed South African road construction projects. This was only a limited investigation, however, and it is imperative that much more information be analyzed if the parameters involved are to be adequately quantified.

For each of the chosen product properties and structural layers involved in a particular contract, a lot consisting of between 20 and 110 test values was analyzed and the mean, the standard deviation, and the coefficient of variation were determined. These determinations were repeated for different lots from various contracts throughout South Africa (Tables 1, 2, and 3); the number of lots (n) varied between 3 and 34.

The following quantities were among those finally obtained for each property: overall mean specification value (x_s), overall mean achieved ($\bar{x}$), standard deviation ($\sigma_{\bar{x}}$) and coefficient of variation ($v_{\bar{x}}$) for all n groups, and standard error of all standard deviations (σ_σ) and coefficients of variation (σ_v). In addition, the 50th and 70th percentiles of both the standard deviation and the coefficient of variation (σ_{50}, σ_{70}, V_{50}, and V_{70}) were calculated as well as the percentage defective (ϕ) of the achieved mean ($\bar{x}$) relative to the specified mean (x_s) by using the 70th percentile of the coefficient of variation (V_{70}). V_{70} was used because it was considered to be generally achievable under present conditions.

In the scheme used, the contractor's risk (α) is fixed at 5 percent at the acceptance limit, which means that the contractor runs a 1-in-20 risk of having a product of borderline quality rejected. The risk at the acceptance limit will be lower for a higher quality material and higher for a lower quality material.

The same principles apply to material properties with upper specification limits or double specification limits. But, in the case of properties with double

Table 1. Variability of bituminous materials.

Parameter	Number of Lots (n)	Overall Mean ($\bar{\bar{x}}$)	Coefficient of Variation of Lot Means ($V_{\bar{x}}$ percent)	50th Percentile Coefficient of Variation (V_{50} percent)	Percentage Defective From Current Specifications (ϕ based on σ_{70})
Continuously graded material					
Flow	10	2.34	15.11	14.50	33.05
Stability	10	9.87	19.32	13.00	0.21
Voids	10	4.94	19.58	16.78	37.51
Bitumen content	10	5.42	8.17	2.40	7.29
Passing sieves					
13.2 mm	10	98.51	1.69	0.90	0.27
4.75 mm	10	53.99	6.74	6.40	31.30
2.36 mm	10	36.42	8.88	8.00	40.02
0.30 mm	9	14.71	8.06	11.08	10.60
0.075 mm	10	7.07	9.35	11.00	1.76
Percentage compaction	10	96.19	0.60	1.12	16.11
Gap-graded material					
Flow	17	4.68	80.54	15.00	48.16
Stability	17	7.14	15.75	17.25	5.11
Voids	17	5.29	17.99	16.75	21.58
Sieve analysis (0.075 mm < x < 2.36 mm)	17	43.11	18.12	6.76	62.06

Table 2. Variability of unstabilized materials.

Parameter	Number of Lots (n)	Overall Mean ($\bar{\bar{x}}$)	Coefficient of Variation of Lot Means ($V_{\bar{x}}$ percent)	50th Percentile Coefficient of Variation (V_{50} percent)	Percentage Defective From Current Specifications (ϕ based on σ_{70})
Compaction of fill	16	96.63	1.99	2.88	4.05
Selected subgrade					
Compaction	14	98.20	2.69	3.25	9.73
California Bearing Ratio	8	42.68	26.07	34.50	1.58
Plasticity index	14	4.27	54.22	80.00	3.03
Liquid limit	12	15.63	36.96	62.00	36.94
Subbase					
Compaction	16	99.37	2.14	2.38	6.11
California Bearing Ratio	12	59.44	57.60	21.67	13.35
Plasticity index	13	3.50	67.25	65.00	0.32
Liquid limit	13	12.09	65.28	81.00	40.87
Compaction of base course (natural gravel)	6	99.59	0.27	1.30	20.39
Crusher-run base course					
Compaction	12	97.46	6.92	2.20	11.86
Layer thickness	34	130.71	15.31	8.30	41.10
Plasticity index	7	3.04	18.75	43.75	4.56
Liquid limit	7	17.61	13.73	36.50	13.31
Passing sieves					
2 mm	11	26.50	11.60	14.85	14.54
0.425 mm	12	14.07	24.16	17.00	10.51
0.075 mm	10	5.73	47.91	21.00	32.56

Table 3. Variability of stabilized materials.

Parameter	Number of Lots (n)	Overall Mean ($\bar{\bar{x}}$)	Coefficient of Variation of Lot Means ($V_{\bar{x}}$ percent)	50th Percentile Coefficient of Variation (V_{50} percent)	Percentage Defective From Current Specifications (ϕ based on σ_{70})
Compaction of stabilized subgrade	5	96.75	1.51	2.56	7.70
Stabilized subbase					
Compaction	19	98.18	1.57	2.15	12.81
Layer thickness	17	134.03	11.21	6.25	46.22
California Bearing Ratio	6	215.50	35.43	37.50	11.55
Plasticity index	5	5.85	79.34	23.50	9.70
Liquid limit	5	19.69	48.53	9.33	—
Stabilized natural gravel base course					
Compaction	34	99.96	2.33	2.00	12.19
California Bearing Ratio	26	306.74	24.76	30.83	9.48
Plasticity index	3	3.86	113.76	67.00	10.53
Liquid limit	4	15.97	55.15	128.00	17.06
Stabilized crusher-run base course					
Compaction	27	100.69	1.48	1.93	11.40
Layer thickness	19	113.84	15.28	9.56	10.82
Plasticity index	10	2.94	33.45	32.50	1.17
Liquid limit	10	17.12	30.18	13.00	13.03
Passing sieves					
2 mm	27	31.92	22.38	12.38	4.75
0.425 mm	28	16.39	22.46	15.00	2.35

Table 4. β risks of the client.

Property	n	ϕ_a	ϕ_u	Client's Risk (%) β_a (α = 5 percent)	β_r (α = 1 percent)	β_r (α = 0.1 percent)
Relative density of fill	4	5	20	51.54	76.43	93.11
Relative density of selected subgrade	4	10	40	34.02	60.63	84.93
Subbase						
Relative density	6	10	40	19.10	42.36	71.61
Lime or cement content (if stabilized)	20	10	40	0.16	1.15	6.57
Crusher-run base						
Relative density	6	15	60	6.49	20.24	47.24
Grading						
Percentage < 19 mm	6	10	40	19.10	42.36	71.61
Percentage < 2.36 mm	6	10	40	19.10	42.36	71.61
Percentage < 0.425 mm	6	10	40	19.10	42.36	71.61
Percentage < 0.075 mm	6	10	40	19.10	42.36	71.61
Asphalt surfacing						
Relative density	6	15	60	6.49	20.24	47.24
Asphalt content	8	10	40	10.32	28.02	57.21
Grading						
Percentage < 13.2 mm	6	10	40	19.10	42.36	71.61
Percentage < 2.36 mm	6	10	40	19.10	42.36	71.61
Percentage < 0.300 mm	6	10	40	19.10	42.36	71.61
Percentage < 0.075 mm	6	10	40	19.10	42.36	71.61

specification limits, the value of ϕ_a, which is used in deriving the acceptance limit, is limited to 50 percent of the total percentage allowed to be outside the double specification limits.

If the test results are to represent the true population of the material property as accurately as possible, the samples must be randomly obtained (i.e., every position or portion of the material should have an equal chance of being selected for sampling). This leads to a more balanced assessment of the material by eliminating the subjective element that would otherwise be involved.

The distinction between acceptance control and process control procedures should be noted: Acceptance control indicates the inherent quality of the finished population from which the sample was drawn, and process control, which is based on selected sampling during the actual production process, indicates adjustments required of the producer to maintain the process within the prescribed limits.

FEATURES OF ACCEPTANCE CONTROL PLAN

Contractor's Risk

Implicit in the acceptance control plan of the South Africa Department of Transport (1) is the fact that both the contractor and the client at all times run the risk that the wrong conclusion will be drawn from the test results. These are the commonly known α and β risks. The relation between the α and β risks is as follows:

$$(k_\alpha + k_\beta) = \sqrt{n}(k_{\phi_a} - k_{\phi_u}) \quad (1)$$

where

- k_α = normal distribution constant, related to the contractor's risk of α percent;
- k_β = normal distribution constant, related to the client's risk of β percent;
- k_{ϕ_a} = normal distribution constant, related to the percentage ϕ_a; and
- k_{ϕ_u} = normal distribution constant, related to the percentage ϕ_u.

Both ϕ_a and ϕ_u are, of necessity, dependent variables because they must conform to specified engineering requirements in the definition of acceptable and unacceptable products. n may be an independent variable, but for the immediate future at least it should conform as far as possible to currently used sample sizes. At this stage, therefore, n is a dependent variable. In the case of the remaining parameters, α and β, either one could be an independent variable but the other must be a dependent variable. For the control of an unacceptable product, β is fixed to limit the client's risk; for the control of an acceptable product, α is fixed to limit the contractor's risk.

In South African practice, it is currently deemed advisable to control the acceptable product—in other words, to fix α—because α is the only unknown parameter whereas, in the control of the unacceptable product, decisions would have to be taken on both ϕ_u and β. It is also felt that the fixing of α at a reasonably low level will provide the essential assurance to the contractor—through the introduction of rational quality assurance—of fair judgment under all circumstances.

The contractor's risk at the acceptance limit (α_a) was set at a reasonable level of 5 percent. A second limit with an even lower risk, known as the rejection limit, was introduced. Initially, the contractor's risk at the rejection limit (α_r) was set as low as 0.1 percent. However, as the contractor's risk of having an acceptable product wrongly assessed decreases, the client's risk of accepting an unacceptable product increases (Figure 2).

Table 4 shows the β risks of the client for some of the properties that have already been incorporated into the system for the proposed sample sizes; α risks equal to 5.0, 1.0, and 0.1 percent; and ϕ_u equal to four times ϕ_a. Data given in Table 4 clearly indicate that the client assumes a much greater risk than the contractor does. For this reason, the contractor's risk was set at 1 percent at the rejection limit, which leads to a substantial decrease in the client's risk.

Effect of Using Sample Standard Deviation

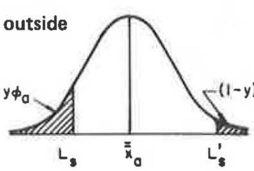
Although the assumption has been made that the true standard deviation (σ) of the population as a whole is equal to the sample standard deviation (S), this is not quite correct. However, S has a normal distribution with a mean $[\sqrt{(2n-3)/(2n-2)} \cdot \sigma]$ and a standard deviation $(\sigma/\sqrt{2n-2})$. A product is judged by comparing the mean test result ($\bar{x}_n$) with the acceptance limit (L_a or L_a') or the rejection limit (L_r or L_r'). By taking the properties of S into account, it is possible to calculate the true α risk (the contractor's risk of being wrongly

Table 5. True α risks of the contractor.

Property	Sample Size	Contractor's Risk		
		α_{at}	α_{rt}	ϕ_a
Relative density of fill	4	6.88	1.24	5
Relative density of selected subgrade	4	5.31	0.97	10
Subbase				
Relative density	6	5.98	1.09	10
Lime or cement content (if stabilized)	20	7.84	1.90	10
Crusher-run base				
Relative density	6	5.19	0.98	15
Grading				
Percentage < 19 mm	6	5.98	1.09	10[a]
Percentage < 2.36 mm	6	5.98	1.09	10[a]
Percentage < 0.425 mm	6	5.98	1.09	10[a]
Percentage < 0.075 mm	6	5.98	1.09	10[a]
Asphalt surfacing				
Relative density	6	5.19	0.98	15
Asphalt content	8	6.46	1.25	10[a]
Grading				
Percentage < 13.2 mm	6	5.98	1.09	10[a]
Percentage < 2.36 mm	6	5.98	1.09	10[a]
Percentage < 0.300 mm	6	5.98	1.09	10[a]
Percentage < 0.075 mm	6	5.98	1.09	10[a]

[a]In the case of properties with double specification limits, ϕ_a is equal to 0.5 ϕ_a.

Figure 3. Parts of the population outside the specification limits.



assessed) in comparison with the theoretical α risks of 5 percent (α_a) and 1 percent α_r.

The risk at the acceptance limit is the probability that $\bar{x}_n$ will be smaller than L_a and can be shown to be equal to the following for a lower specification limit:

$$\alpha_a = F\left\{[K_{\phi_a} - K_a\sqrt{(2n-3)/(2n-2)}] \div \sqrt{(1/n) + [K_a^2/(2n-2)]}\right\} \tag{2}$$

where $k_a = k_{\phi_a} - (k\alpha_a/\sqrt{n}) = F\,(K_{\alpha_{rt}})$ where α_{at} is equal to the true α_a risk. Likewise, the risk at the rejection limit can be shown to be equal to the following:

$$\alpha_r = F\left\{[K_{\phi_a} - K_r\sqrt{(2n-3)/(2n-2)}] \div \sqrt{(1/n) + [K_r^2/(2n-2)]}\right\} \tag{3}$$

where $K_r = K_{\phi_a} - (K\alpha_r/\sqrt{n}) = F(K_{\alpha_{rt}})$ where α_{rt} is equal to the true α_r risk. There are similar relations for upper specification limits.

It is quite clear from these terms that true α risks are independent of L_s or S but are dependent on K_a or K_r, K_{ϕ_a}, and n.

Table 5 gives some of the properties that have been incorporated into the system, together with the recommended sample size, as well as the true α risks at the theoretical acceptance limit (α_a = 5 percent) and theoretical rejection limit (α_r = 1 percent). Table 5 clearly indicates that the assumption that the true standard deviation (σ) of the material as a whole is equal to the sample standard deviation (S) does lead to slightly higher α risks for the contractor. But Table 4 clearly indicates that the client's risk of accepting an unacceptable product with ϕ_u equal to four times ϕ_a is far greater than the contractor's risk of being wrongly assessed. For this reason, it seems quite equitable that the true standard deviation of the material as a whole should be accepted as equal to the sample standard deviation (σ = S).

ϕ_a for Double Specification Limits

In the case of properties with double specification limits, there may be a certain amount of material beyond both the upper and lower specification limits, as shown in Figure 3. The total allowable percentage of material outside the specification limits is expressed as follows:

$$\phi_a = y\phi_a + (1-y)\phi_a \tag{4}$$

The initial problem was to decide what value the fraction (y) should have. By using different values for y and drawing operating characteristic curves for these different values, the influence the value of y has on the assessment of a set of test results can be determined.

It was found that the closer y, or 1 - y, was to unity, the greater would be the probability of accepting an unacceptable product. For this reason it was decided to fix the value of y at 0.5. In other words, the maximum amount of material allowed to be outside either the upper or lower specification limit is limited to 50 percent of the total amount of material allowed outside the specification limits, based on past experience.

CONTRACT ADMINISTRATION

The contract was administered by the resident engineer, and the specified statistical method of control was used for the acceptance of certain portions of the work. Each section of the pavement layer under review was subjected to acceptance control; generally, seven randomly selected samples were taken from a lot (or a day's work). The lot would be accepted if the mean of the sample results was greater than L_s (or, for some parameters, e.g., percentage passing the 0.075-mm sieve, less than L_s') plus (or minus) the range (the difference between the highest and lowest test results) multiplied by a prespecified factor. (In future contracts the standard deviation will be used instead of its simplified approximation, i.e., the range.)

In the subbase layers, density, lime content, and percentage passing the 0.075-mm sieve were subjected to statistical acceptance control. The asphaltic concrete base was controlled by an assessment of aggregate gradings, filler and asphalt content, and density. Payment penalties were imposed on the bitumen layers for material that failed to meet the requirements of the specification; the maximum allowable penalty was a 30 percent reduction in payment. If the parameters tested resulted in a payment reduction of more than 30 percent, the lot would be rejected and would have to be removed.

The subbase layers on the contract consisted of three 150-mm-thick lime-stabilized layers; the bottom layers were natural shale material excavated from the road prism, and the upper layer was an imported tillite crusher-run with a specified plasticity index of 4 to 12. All the layers were mixed on site by means of graders and a mechanical mixer-leveler unit.

Visual inspection of the processed subbase showed the mixture to be consistent and homogeneous. Control testing of the material confirmed the visual inspection, and few sections were rejected by the statistical assessment of the results. On this contract, the contractor made a great effort to maintain a high standard in the processing of the subbase, and the processed material showed good results.

The results of the tests done on samples of the asphaltic concrete base material indicated a wide variability in the product that was mainly caused by the variability of the fine-sand fraction of the aggregates. Penalties were invoked, and the contractor suffered

Table 6. Payment deductions for apshaltic concrete under new specifications.

	Grading										
	Coarse Aggregate		Fine Aggregate		Filler		Asphalt				
Lot (Mg)	Payment Deduction (%)	Number of Test Failures	Payment Deduction (%)	Number of Test Failures	Payment Deduction (%)	Number of Test Failures	Payment Deduction (%)	Number of Test Failures	Total Penalty (cumulative percentage)	Scheduled Payment[a] (R)	Penalty Deduction[b] (R)
486.06	5	7 out of 7	5	7 out of 7	—	—	—	—	10	4 392.83	439.28
611.87	5	4 out of 7	—	—	10	5 out of 7	—	—	15	5 529.80	829.47
603.70	5	5 out of 7	5	5 out of 7	10	5 out of 7	10	2 out of 7	30	5 405.11	1 621.53
523.38	—	—	—	—	10	2 out of 7	10	1 out of 7	20	4 729.98	946.00
535.44	—	—	—	—	10	4 out of 7	10	3 out of 7	20	4 838.97	967.79
82.14	5	1 out of 2	5	1 out of 2	—	—	10	1 out of 2	20	742.38	148.48
600.79	—	1 out of 7	5	1 out of 7	10	4 out of 7	10	3 out of 7	25	5 405.35	1 351.34
241.16	—	—	—	—	—	—	10	2 out of 3	10	2 139.01	213.90
609.03	5	2 out of 7	—	—	10	5 out of 7	10	4 out of 7	25	5 453.15	1 363.29
223.33	5	3 out of 4	5	5	10	4 out of 7	—	1 out of 4	20	2 018.23	403.64
Total										40 654.81	8 284.72

Note: 1 R = $1.15.

[a]Value of asphaltic concrete without penalty deduction.

[b]Monetary value deducted from day's work.

Table 7. Payment deductions for asphaltic concrete under original specifications.

	Number of Test Failures[a]						
	Grading						
Lot (Mg)	Coarse Aggregate[b]	Fine Aggregate[b]	Filler[b]	Asphalt	Total Penalty (%)	Scheduled Payment[c] (R)	Penalty Deduction[d] (R)
486.06	4 out of 4	4 out of 4	—	—	100	4 392.83	4 392.28
611.87	4 out of 6	—	4 out of 6	—	71	5 529.80	3 949.86
603.70	5 out of 6	6 out of 6	6 out of 6	1 out of 6	100	5 405.11	5 405.11
523.38	—	—	3 out of 5	—	29	4 729.98	1 351.42
535.44	—	—	—	3 out of 5	57	4 838.97	2 765.13
82.14	—	—	—	1 out of 2	50	742.38	371.19
600.79	1 out of 5	1 out of 5	3 out of 5	3 out of 5	57	5 405.35	3 088.77
241.16	—	—	—	2 out of 3	67	2 139.01	1 426.01
609.03	2 out of 7	—	5 out of 7	4 out of 7	71	5 453.15	3 895.11
223.33	3 out of 4	—	4 out of 4	1 out of 4	100	2 018.23	2 018.23
Total						40 654.81	28 663.11

Note: 1 R = $1.15.

[a]Tests per 100 Mg not conforming to specification.

[b]Based on one test per 100 Mg production.

[c]Value of asphaltic concrete without penalty deduction.

[d]Monetary value of deductions for asphaltic concrete judged under original specification.

financially. However, the penalties imposed were less onerous when judged on a statistical basis than when judged by the older type of specification. Tables 6 and 7 provide a comparison between new and original specifications in the assessments of some of the test results for the asphaltic concrete base material that was judged defective [data were taken from lots (a day's work) from December 1974 through April 1975]. Similar information is available for all materials and properties subjected to statistical acceptance control on this contract.

A portion of the asphaltic concrete base was rejected completely because it failed to comply with any of the specified parameters, especially that for asphalt content. The contractor was required to remove it from the work site. This material constituted 0.06 percent of the total lot of material laid down in the contract.

DISCUSSION OF RESULTS

Lime-Stabilized Layers

The results of tests on material taken from the lime-stabilized layers showed that the contractor had decided to play it safe and not take advantage of a uniform processing operation whereby the amount of stabilizing agent could be reduced.

The standard deviation of the test results was fairly consistent at a figure of 0.65 and indicated a reasonable amount of control over the processing operation, which was the client's objective.

Asphaltic Concrete Base Course

Asphalt Content

The specification in operation on the contract clearly defined the upper and lower acceptance limits. When the mean of seven test results fell outside the acceptance limits, a penalty of 10 percent payment reduction was imposed for work done. The acceptance limits were directly connected to the range of the test results. A large range produces a small difference between upper and lower acceptance limits. Such a range indicates poor production control and a poor product. Because the product was not consistent, the trend charts compiled for the test results did not show any particular trend.

Filler Content

The variation of the filler content (the -0.075-mm fraction) in the base mix was largely responsible for the penalty the contractor suffered.

A comparison between actual and specified range and standard deviation clearly showed the variability of the filler content. If it had been judged on the standard deviation specification, a large percentage of the asphaltic concrete would have been unconditionally rejected. Instead, by using the range as an indication of variability, the financial penalty was imposed as required by the specification.

Comparison of Assessments

Tables 6 and 7 give a comparison, based on test results selected at random, between material judged on the statistical acceptance scheme and on the old, or original, type of specification. It is apparent from the tables that marginally acceptable material is not penalized as heavily under the new statistical acceptance control scheme. Under this specification scheme, the client's risk appears to be higher than it was under the original specification scheme, especially when a contractor is not capable of producing a consistent product. However, because the quality of the work was generally well above the minimum standard required, a fully conclusive assessment of the financial implications of using the acceptance control scheme was not possible.

COMPARISON OF STATISTICAL ACCEPTANCE CONTROL WITH ENGINEERING JUDGMENT PROCEDURE

Before statistical principles were applied to quality control, engineering judgment based on an analysis of the test results was applied in a rational manner to the acceptance control of work. Because different resident engineers might have interpreted the specifications with varying degrees of harshness, it was decided to compare all acceptance control test results for this contract with the assessments of five experienced road construction engineers.

When the asphaltic concrete test results were assessed according to the statistical method, and this assessment was compared with that of all the engineers, the results were as follows:

1. The engineers' assessments of the individual properties agreed with the statistical assessments, on average, 77 percent of the time. Agreement varied from 81 to 73 percent for the different properties.
2. On average, the engineers agreed among themselves 95 percent of the time in their assessments of individual properties. This agreement varied between 98 and 92 percent for the different properties.
3. In an overall assessment of the lots submitted for assessment, the engineers agreed among themselves 82 percent.
4. The mean payment factor according to the engineers' assessments (using the payment system used on the contract) was 0.89. Among the engineers this factor ranged from 0.91 to 0.87.
5. The engineers' judgment was that, if the old specification had been rigidly applied, the mean payment factor for all the work would have been 0.54. Among the engineers this factor ranged from 0.64 to 0.44. (Rigid application means no payment for a rejected lot.) The mean payment factor for all the work, according to the statistical method, was 0.79.

Because the quality of the asphaltic concrete on this contract was extremely variable, test results from another asphaltic concrete project—constructed by the same paving contractor with the same plant but with a better quality asphaltic concrete—were assessed in exactly the same way as were the results on this contract. To avoid any bias in the engineers' assessment, this set of results, called section 2, was separated from the first set of results, called section 1. The following findings resulted from the assessment of section 2:

1. The engineers' assessments of the individual properties agreed with the statistical assessments, on average, 92 percent of the time.
2. On average, the engineers agreed among themselves 99 percent of the time in their assessment of individual properties.
3. The mean payment factor according to the engineers' assessments (using the payment system used on the contract) was 0.99.
4. The engineers' judgment was that, if the old specification had been rigidly applied, the mean payment factor would have been 0.97.
5. The mean payment factor for section 2, according to the statistical method, was 0.96 (when the engineers used the system used on the contract).

When the subbase test results for the contract were assessed according to the statistical method and this assessment was compared with that of the five engineers, the results were as follows:

1. The engineers' assessments of the individual properties agreed with the statistical assessments, on average, 95 percent of the time.
2. On average, the engineers agreed among themselves 98 percent of the time in their assessment of individual properties.
3. In the overall assessment of the lots, the engineers agreed among themselves 91 percent of the time.

A comparison of the assessments of the asphaltic concrete of sections 1 and 2 reveals that, as the quality of the material decreases, the correlation in the assessments of the material decreases. This is clearly a result of the subjective element involved when work is assessed purely on the basis of engineering judgment. In addition, greater assessment problems are involved in arriving at a balanced decision about borderline material (this is borne out by the subbase results). It can be concluded from these results that the use of the statistical method leads to more consistent interpretations of results than does the judgmental approach to acceptance control.

Some of the engineers involved in the assessment were asked to make another assessment of the project after a substantial time period had elapsed without referring to their original assessments. The reassessments of individual properties of the subbase agreed with the original assessments, on average, 97 percent of the time. For the overall assessments of the subbase, the correlation was 95 percent.

The reassessments of the individual properties of section 1 asphaltic concrete agreed with the initial assessments, on average, 94 percent of the time. However, in the overall reassessment of the asphaltic concrete, the correlation dropped to about 80 percent. The reassessments of the individual properties of section 2 asphaltic concrete agreed with the initial assessments, on average, 99 percent of the time. In the overall reassessment of the asphaltic concrete, this correlation dropped to about 95 percent.

It is clear from these results that, although engineers were able to reassess individual test results with a fair degree of repeatability, the overall reassessments of the lots, which combined more than one acceptance parameter, were not as accurate, which clearly points to the subjective element involved when engineering judgment is used. If the statistical approach had been used, the correlation in all cases would have been 100 percent because the material would have been judged according to certain criteria whose influence on the assessment remained stable irrespective of the subjective approach of the engineer involved.

CONCLUSIONS

Although the project described in this paper was the first application of this scheme to a road project in South Africa, approximately $200 million of work incorporating the use of this or similar acceptance control schemes has since been let to contract. Unfortunately, however, it is not possible to gauge accurately the feelings of contractors about the scheme. Certain contractors—generally the more technologically advanced organizations—appear to welcome the approach, but others have expressed their doubts about it. The more dubious contractors generally do not appear to understand the principles involved nor to be able to explain their misgivings clearly.

A great deal of education in statistical principles thus appears to be required. This education, the collecting and processing of more and reliable information about the variability associated with construction control testing, and the relative variabilities contributed by the sampling and testing processes are regarded as the most important phases of this work to be done in the immediate future. It is hoped that the introduction of a quality assurance subsystem into the computerized data management system currently being implemented by the South Africa Department of Transport will help to some extent in obtaining more information about variability in materials and processes.

A study of the results of tests on asphaltic concrete showed clearly that, when a contractor makes a definite effort to produce a homogeneous or consistent product, there is no difficulty in fulfilling the requirements of the specification. The use of inconsistent material in the production of asphaltic concrete can only lead to trouble. Use of the statistical acceptance plan provides a client with an adequate means of judging the product.

The statistical acceptance control scheme should not be seen as another "big stick" with which the engineer may beat the contractor but as a scientific assessment of the contractor's capability to produce a uniform product. Ad hoc or biased judgments of the product are eliminated, and on-site arguments between the contractor and the resident engineer are reduced to a minimum. The contractor is encouraged to produce a uniform product, which is what the client desires, and the benefits that accrue to both contractor and client must eventually accrue to the construction industry as a whole.

The continued use of statistical acceptance control on road-work projects is therefore recommended. The ultimate aim of the major clients connected with the road construction industry in South Africa is to develop a standard statistical acceptance control specification based on the several specifications that are now being implemented throughout the country.

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Quality Criteria for Maintenance and Reinforcement of Pavements in the Netherlands

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In the next 10 years, population and traffic density will reach a maximum in the Netherlands and other industrial countries of Western Europe. The need for an adequate policy for spending the available (and decreasing) funds for roads necessitates the development and application of objective criteria for maintenance and reconstruction of the existing road network. These criteria deal mainly with road-surface characteristics (safety and riding comfort), bearing capacity (strength) and durability of the pavement, and traffic safety. Quality criteria relating to road-surface characteristics and the condition of the pavement as a bearing layer are examined. In the Netherlands, such criteria are gradually being more systematically applied in judging priorities and making decisions on maintenance and strengthening of pavements. These criteria have been developed on the basis of measurements and research on the national road system carried out by the State Road Laboratory, especially during the past 25 years. An explanation is given of the methods of measurement, the interpretation of the results of testing and visual inspection of roads, and the way the data are used in developing and applying a system of rational pavement management practice.

In a number of Western European countries, and particularly in the densely populated Netherlands, the planned road networks required to provide an effective infrastructure are nearing completion. Figure 1 shows the 1968 state highway plan for the Netherlands; after some necessary correction, the greater part of this plan will be completed in the next 10 years. In the past 15 to 20 years, a number of new main roads, primarily motorways, have been built. These roads linked large industrial and residential areas but also frequently bypassed those centers of population. As a result, a degree of saturation will be reached in the next 5 to 10 years. This trend is being accentuated by increasing resistance on the part of certain groups to further extension of the road network, the resulting loss of areas of open country, and the continued damage to the environment caused by the increasing volume of road traffic. The result is, of course, that road builders and highway authorities are turning their attention more and more to methods of maintaining, strengthening, and reconstructing existing roads instead of building new ones.

MANAGEMENT SYSTEM FOR ROADS

For several years, the aim in the Netherlands has been to achieve greater rationalization in the determination of objective criteria for pavement maintenance, reinforcement, and reconstruction with the object of making the best use of financial resources. Since 1973, the results of measurements and the criteria developed have been more systematically applied to objective decisions on the maintenance and reinforcement of existing primary roads. To decide whether maintenance and reinforcement measures are needed, the following objective criteria are applied:

1. The surface characteristics of the pavement, i.e., skidding resistance, evenness, rutting, and surface texture;
2. The condition of the pavement as a bearing layer, i.e., cracking, disintegration of materials, and deformation; and
3. Factors of traffic safety, i.e., location of the road, roadway width, and crossfall for drainage.

A final judgment on the condition and the quality of the pavement can only be made after data on the age and the structure of the pavement and the traffic load are considered and a visual inspection is made.

In the Netherlands, the State Road Laboratory performs such measurements and inspections. The results and the judgment made based on the results and on pavement data, traffic volumes, and road situations are sent to the authorities concerned. The executive authorities can then determine a draft working scheme and draft specifications. At this stage, the State Road Laboratory may be asked to give general advice.

CRITERIA FOR DETERMINING MAINTENANCE AND REINFORCEMENT NEEDS

Skid Resistance

The main characteristics that determine skid resistance, other than traffic density, are the texture of the road surface and the resistance of the aggregates to polishing. A considerable amount of research has been done on this subject in the Netherlands in the past few years.

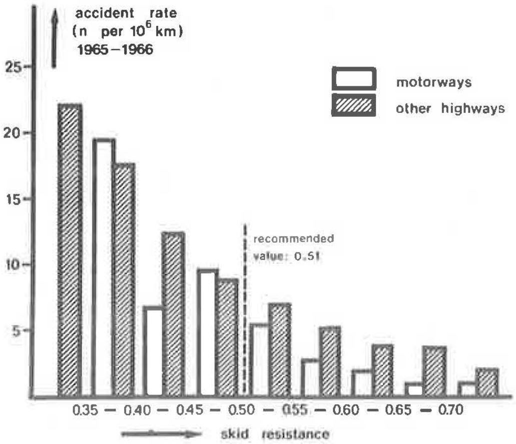
The complete network of existing primary roads (national highways) in the Netherlands totals about 4000 km. It has been fully checked for skid resistance every year since 1953 by using 100-m test sections that are representative of road sections several kilometers in length and that have the same pavement type and traffic history. A standard test vehicle with a retarded wheel (86 percent slip) is used (Figure 2) on a wet road surface at a speed of 50 km/h, usually in the right-hand lane. In some cases, especially if the depth of the pavement texture is low, measurements are carried out at a speed of 70 km/h because of the possibility of a greater than usual decrease in skid resistance at increasing speeds.

The annual testing of national highways takes place

Figure 1. 1968 state highway plan for the Netherlands.

Figure 2. Skid-resistance tester.

Figure 3. Relation between skid resistance and accident rate (1965-1966).



from August to November. During this period, skid resistance may reach minimum values because of summer temperatures and polishing (in winter, skid resistance is improved by erosion). Because this seasonal influence largely determines the condition of the road surface, it must be taken into account in the interpretation of test results.

This systematic examination involves only random measurements. The measurements made in connection with working plans are much more extensive, particularly at junctions, slopes, overtaking lanes, and "black spots."

To establish the relation between skid resistance and accident risk on a wet road surface, data were collected on all accidents that occurred on national highways in the Netherlands in 1965 and 1966 (Figure 3). These data can be used not only for the determination of maintenance priorities or for comparisons with a model but also for budgetary purposes.

In the classification adopted by the Netherlands State Road Laboratory, a coefficient of friction (f) of 0.51 measured at a speed of 50 km/h is regarded as a minimum value for national highways. Values lower than 0.51 are considered insufficient and indicate that measures should be taken to improve skid resistance. However, the State Road Laboratory does not always recommend immediate measures for improvement or traffic restrictions. This depends strongly on factors such as the length of the road section in question, the date the measurements were taken, the situation of the road, the cause of the low skid resistance, the traffic volume, the condition of the pavement, and any strengthening operations already planned.

If skid-resistance values below 0.46 are found, immediate action—such as the placing of SLIPPERY ROAD signs—is recommended regardless of the length or the location of the road section. If necessary, other measures such as restrictions on speed are also taken. At the same time, measures to improve skid resistance are prepared.

In summary, it can be stated that the following directives regarding the friction coefficient (f) are applied in practice: If $f < 0.46$, the pavement must always be repaired; if $f = 0.46$ to 0.51, the need for repair depends mainly on road traffic conditions; and if $f \geq 0.51$, pavement repair is generally not undertaken.

Figure 4 shows the frequency distribution of skid-resistance values for various lengths of national roads measured in 1970 by means of systematic investigations. (Because of the effort required to determine this distribution, it has not been done since 1970, but evidence indicates that the distribution has not greatly changed since that time.)

Figure 5 shows the percentage length of the national highway system that had a friction coefficient lower than the recommended minimum value as measured during the period from 1954 to 1974. Figure 6 shows the frequency distribution of 1975 skid-resistance values determined in relation to 1976 work plans for maintenance and reinforcement of highways.

Figure 4. 1970 frequency distribution of skid-resistance values.

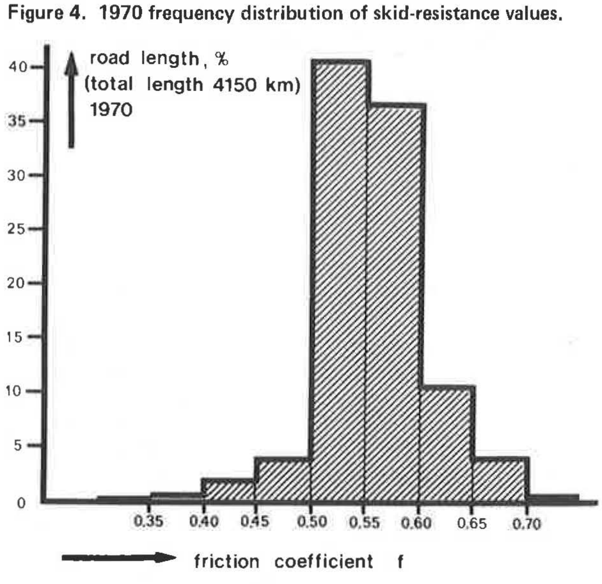


Figure 5. Percentage length of roads from 1954 to 1974 with a friction coefficient below 0.51.

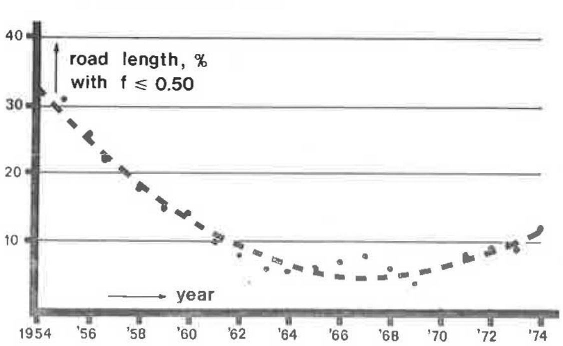


Figure 6. 1975 frequency distribution of skid-resistance values measured in relation to 1976 work plans for maintenance and reinforcement.

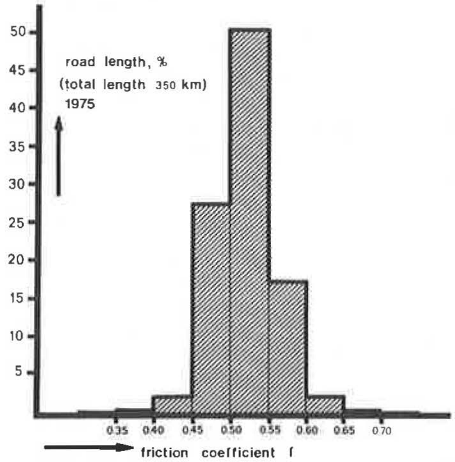


Figure 7. Roughometer.

Roughness

The present serviceability index criterion (PSI), developed in the United States in connection with the AASHO Road Test, is used in many countries. It consists primarily of results obtained from measurements of longitudinal roughness, rut depth, cracking, and repaired surface. But in the Netherlands this combination is frequently not very meaningful. Regarding PSI as an exclusive figure indicating the residual value of the pavement ignores two other important parameters for pavement quality—skid resistance and bearing capacity. If PSI is regarded as a figure for assessing roughness or riding comfort, the cracking index has little significance. For this reason, separate criteria are used in the Netherlands for longitudinal and transverse roughness.

Longitudinal

To obtain an idea of riding comfort on roads in the Netherlands since 1953, the roughness of the road surface was determined by using the so-called roughometer designed by the U.S. Bureau of Public Roads and manufactured by the U.K. Transport and Road Research Laboratory (Figure 7). The relative displacement of the wheel axle in a single direction with reference to the frame of a one-wheel, mass-spring system (trailer) was measured and totaled over 500- to 1000-m stretches of road. Measurements were made at a test speed of 50 km/h.

The following rating has been developed and adopted as a criterion for riding comfort (expressed as the roughometer value): <1.4 = good, 2.4 to 3.4 = moderate, 1.4 to 2.3 = adequate, and ≥3.5 = poor. This rating agrees fairly well with standards adopted in other countries for the interpretation of roughometer results.

In some countries, roughometer results are used as an absolute standard for maintenance. In the Netherlands, however, roughometer values are used for classifying stretches of road in order of evenness. The results, including those from comparative measurements, have indicated a good correlation between roughometer readings and PSI. A roughometer value (R) between 2.5 and 3.5 is approximately equivalent to a PSI between 2.5 and 1.5 (Figure 8); that is, R = 5 - PSI.

The roughness of one-third of the length of all highways is measured every year, and thus the whole national road system is checked for roughness every 3 years. The condition of a given section of road can therefore be compared with the overall condition of the state road network. Figure 9 shows the frequency distribution of roughometer values of national roads measured in 1975 in relation to work plans for maintenance and reinforcement of highways to be carried out in 1976. This distribution does not differ essentially from the overall frequency distribution of these values for all roads except in the percentage of roads classified as poor, which is much greater.

As a rule, improvement of evenness will definitely be necessary if the roughometer rating is poor. If the rating is moderate, factors such as the condition, rut depth, crossfall, and skid resistance of the road will generally play a part in the decision on what measures should be taken.

Transverse (Rut Depth)

Deformation of the pavement in the wheel tracks caused by traffic may result in rutting. The main causes are insufficient stability of the asphalt and high levels of traffic density. Research carried out on test sections of roads in the Netherlands shows that serious rut formation is usually caused by only a few heavy vehicles with high tire pressures on no more than a few very hot days a year.

Although deep ruts impair the steerability and roadholding capability of vehicles, a more serious drawback is that the ruts can fill with water, even at low rainfall intensities. Thick layers of water on roads can greatly decrease skid resistance at higher vehicle speeds. In extreme conditions, hydroplaning may occur. In addition, vehicles moving on water-covered road surfaces will produce a strong water spray that is both annoying and dangerous to other road users.

A normal straightedge can be used in rut measurement. In 1974, however, the State Road Laboratory developed a measuring device known as the rut meter (Figures 10 and 11), which continually registers maximum rut depth. The result is expressed as the mean rut depth per 100 m of road length. The measuring speed is 50 km/h.

The thickness of the water layer in a rut depends on rut depth, transverse and longitudinal slope of the roadway, traffic volume, and rainfall intensity. Investigations are being made into the degree of rutting that is just admissible; the incidence and the prevention of accidents are important factors in this research. Although the investigations have not yet been completed, a tentative mean rut depth of 20 mm per 100 m of road length has been adopted in the Netherlands as a limit beyond which corrective measures are needed. The depth of the right-hand rut is considered the determining factor. The decision on what improvement measures should be recommended depends on the age and the vertical structure of the pavement as well as crossfall, skidding resistance, longitudinal roughness, and possibly other factors.

Figure 12 shows the frequency distribution of rut depths measured in 1975 in relation to work plans for maintenance and reinforcement of highways to be carried out in 1976.

Deflection

Maximum pavement deflections (vertical displacements of the pavement surface) are measured by using the Lacroix deflectometer at intervals of about 4 m, at a measuring speed of about 3 km/h, and under a load of 9 Mg (Figure 13). About 25 measurements are made per 100 m of road length between the left-hand and right-hand dual wheels of a heavy truck. The deflection values, which are not corrected for temperature variations, are classified, and a frequency distribution is recorded for each 100 m of road length. Influence curves (deflection as a function of the distance to the wheels) are determined. It is possible by means of the Lacroix deflectometer to determine the deflection values of rather long stretches of road in a fairly short time. The deflection-related homogeneity of a pavement structure can thus be determined. By using the frequency distribution per 100 m of road length and theory about expected deflection values, any weak spots in the structure can often be clearly discerned. Existing design methods are mainly based on Benkelman beam measurements; the State Road Laboratory (and others) have established the correlation between the two measuring methods.

Recommendations concerning the need for pavement strengthening are based on several deflection-related factors, which are discussed below.

Seasonal Influence

Deflection values are influenced by temperature and humidity. Measurements must be made under comparable

Figure 8. Relation between present serviceability index (P) and bump integrator value (R).

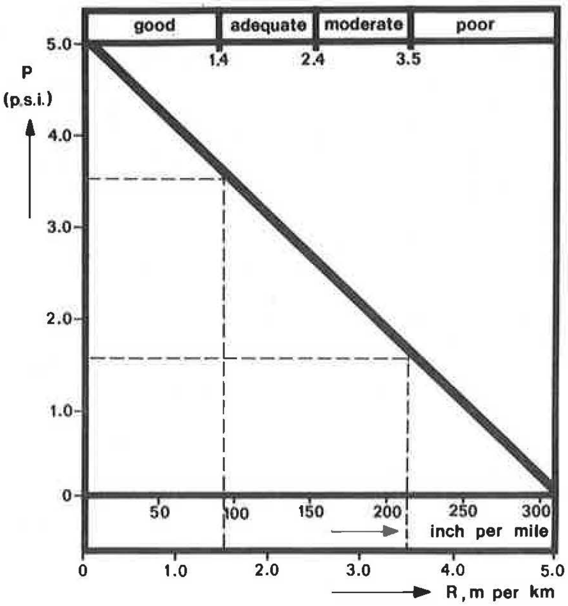


Figure 9. 1975 frequency distribution of roughometer values measured in relation to 1976 work plans for maintenance and reinforcement.

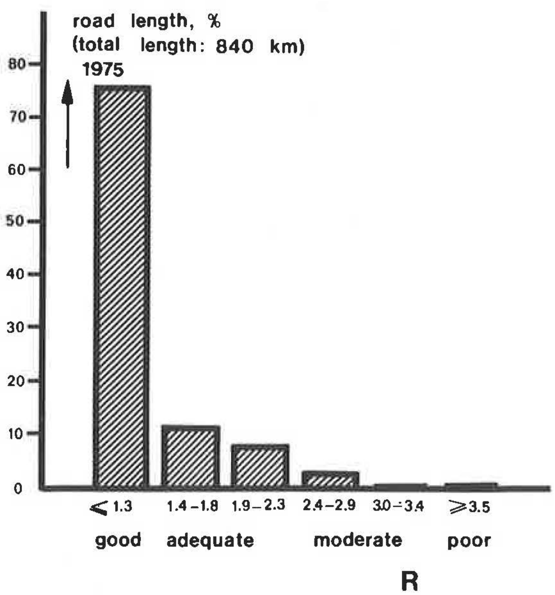
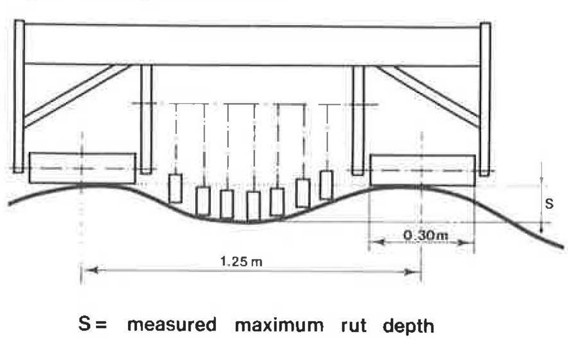


Figure 10. Diagram of rut meter.



conditions to reflect this dependence. The measurements are therefore carried out only in spring and autumn, those seasons in which variations are the smallest and thus reliable corrections can be made.

Figure 11. Rut meter.

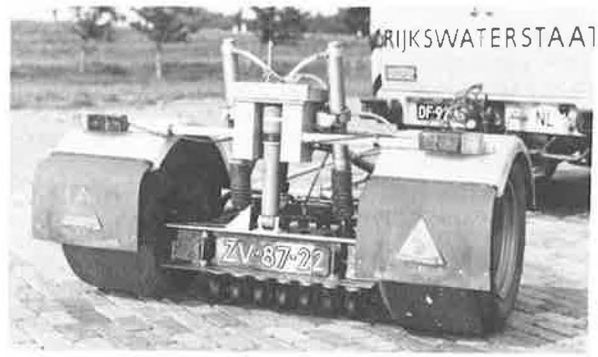


Figure 12. 1975 frequency distribution of rut depth measured in relation to 1976 work plans for maintenance and reinforcement.

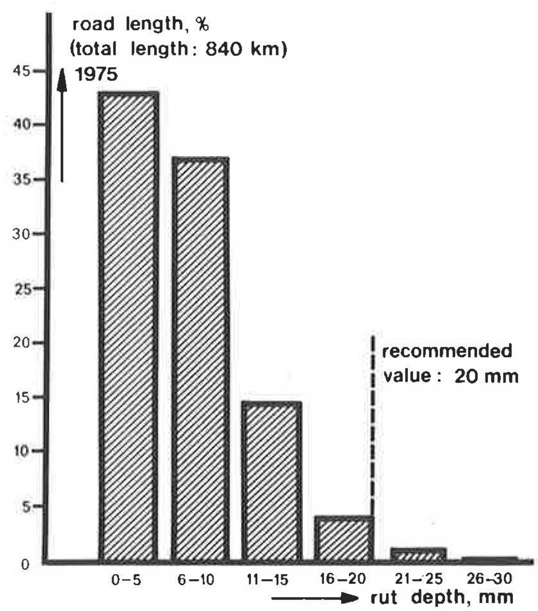


Figure 13. Lacroix deflectometer.

Cracking and Disintegration

Deflection measurements aid in the estimation of the theoretical service life of a pavement. Visual examination of the road surface and reporting of deformation and raveling and the degree and type of cracking make it possible to estimate the point at which strengthening will have to be recommended.

Structural Data

Data about the structure of the pavement are important because the theoretical service life and therefore the deflection values to be expected strongly depend on the materials and layer thicknesses of the structure. Moreover, differences in the behavior of the various materials must be considered.

Age of Pavement Structure

The age of the various parts of the structure is especially important in connection with the total traffic load that has acted on the pavement; such data make it possible to determine that part of the fatigue life of the structure that has been "consumed."

Truck Traffic Volume and Axle-Load Distribution

It appears from the AASHO Road Test that the decline in serviceability of a pavement depends on traffic volume and axle loads. Therefore, truck traffic volume is important in the calculation of the total traffic load. This calculation is based on data for axle-load distribution and on the assumption of a rate of growth in truck traffic of about 4 percent a year.

Additional Data From Test Cores

If data about the structure as a whole or about its parts are unknown or unreliable, more data may be obtained by boring cores. Cores should be taken especially from the right-hand wheel track. If it is believed that the thickness or the material of the structure varies across the road section, this should be verified by taking cores from other parts of the pavement cross section. The number of cores should be adequate to enable the structure of the pavement to be accurately determined. The number of cores taken also depends on the road manager's knowledge of a particular pavement section. In this respect, data on the behavior of the structure and the type of cracking are important.

MEASUREMENT METHODS

The type of measuring equipment used to define the quality of existing pavements is not of critical importance provided that what is measured is sufficiently closely related to the principal road characteristics—safety, riding comfort, and durability. Quality control by means of measurements at the site, the results of which are related to objective criteria, is one of the most significant procedures for maintaining the desired quality level. To ensure that the quality criteria used are reliable and effective, it is most important to use test methods that are quick, reproducible, and, if possible, continuous.

Final and effective decisions on the maintenance and reinforcement measures to be taken in a given road and traffic situation can only be made by combining the results of measurements, by visual inspection at the site, and by thorough deliberation involving an expert team of specialists and the site engineer. This method of road management can only be complete and adequate, however, when pavement maintenance and strengthening activities form part of a total, rational system of management in which other relevant factors (e.g., maintenance of verges and traffic control measures), especially those that have economic consequences, are also weighed.

CONCLUSIONS

The completion of the national road network in the Netherlands calls for much stronger attention to adequate methods of maintenance, strengthening, and reconstruction of existing pavements. Objective standards of quality and maintenance criteria for pavements are essential elements in a total system of pavement management and are necessary for fixing priorities in connection with the appropriation of funds. In the Netherlands, especially in the last 10 years, such objective criteria have been developed, mainly for skid resistance and longitudinal and transverse roughness. The criteria are based on a vast number of results of systematic measurements on main roads (most of them made during the past 20 years) and on long experience in road inspection. Longitudinal friction coefficient (f), roughometer value (R), and maximum rut depth value measured by a rut meter are used in obtaining the results. The results of deflection measurements made by using a Lacroix deflectometer are recorded by an expert inspection team of pavement specialists and are related to pavement and traffic history and data on cracking and disintegration of the surface. Deflection measurements are an essential factor in determining priorities and in making decisions on the maintenance and reinforcement of pavements.

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Quality Control of Pavements in the Netherlands

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A brief historical review of the main elements of the development of quality control in road construction in the Netherlands is given. In this development, the continuing deliberation on technical specifications and methods of construction between the State Road Laboratory and committees of experts from contractors' organizations plays an essential part. The general principles of the nonstatistical quality control systems applied since 1968 as well as the statistical system developed during the past few years are explained based on 10 years' experience in the quality control of more than 250 road works. In the Netherlands, the separation between the responsibility for daily production control exercised by the contractor and the acceptance control exercised by the engineer after completion of the job is of great importance. Acceptance control concerns layer thickness, strength of soil cement, and density and bitumen content of asphalt mixtures on the one hand and skid resistance and roughness of the road surface on the other. Finally, issues of costs and benefits for jobs of inferior quality and the payment deductions required in such cases are discussed.

In the Netherlands, during the period from 1960 to 1965, a start was made in several major road contracts in using a general method of pavement quality control. This method ultimately led to an adequate system accepted by both the national highway authority and the contractors' organization. On the basis of the great quantities of data accumulated during these years, contract conditions connected with penalty clauses were introduced for the first time in 1966 as an experiment on some jobs. The conditions and the specifications were completed during the following 2 years and were drawn up in a definite version in 1968.

Since that time, these developments have been stimulated on the one hand by a lack of sufficiently skilled supervisory staff on the part of the highway authority and on the other hand mainly by ever-increasing mechanization and automation and the increase in magnitude of most of the jobs. The highway authority and the contractors became convinced that the method of unilateral quality control by the engineer, usually based only on testing of a relatively limited number of selectively taken samples, was no longer justified and needed to be changed. Although these conditions have not fundamentally changed since 1968, an alternative system was proposed in 1975 based on statistical principles and on the sampling and testing experience gained over a 10-year period.

CONSULTATION BETWEEN HIGHWAY AUTHORITY AND CONTRACTORS

Before the system of quality control incorporating reduced-payment clauses was introduced in 1968, thorough deliberation took place between the State Road Laboratory (SRL), representing the government, and a committee of contractors' representatives. In the Netherlands, the State Road Laboratory is a central institute of the national authority that is responsible for (a) formulation of the quality specifications, (b) structural design of the constructions and the composition of the mixtures, and (c) directions on quality control and on the execution of that control as far as it concerns the national (primary) roads. This centralization has resulted in a high level of uniformity in road construction specifications and effective consultation with the representative committee of the contractors' organization.

The system of quality control used since 1968 and now revised has been accepted in general terms by the contractors' organization and has been recognized as being reasonable, fair, and effective in reaching the desired quality level.

PRINCIPLES OF NONSTATISTICAL QUALITY CONTROL SYSTEM

The nonstatistical system of quality control is based on a consistent separation between daily production control by the contractor and limited acceptance control by the highway authority after completion of the work. To ensure a correct mix design, the contract prescribes that the contractor is to carry out a preliminary investigation of the strength (and, therefore, the needed cement content) of the sand cement base as well as the composition and the Marshall stability of the asphalt mixtures (the job mix design). The results of the preliminary tests are compared with the results of similar tests conducted by the State Road Laboratory. In this way, clear agreement is reached between the authority and the contractor on the design and the characteristics of the mixtures before the work starts.

Production Control by the Contractor

The contract prescribes that the contractor must maintain thorough, daily production control of the composition and the properties of the mixtures (the specifications are the same as they are for acceptance control testing). Therefore, a well-equipped on-site laboratory with skilled personnel must be available to the contractor.

The contractors have decided over time to make more use of statistical methods in managing the quality of production, especially in the execution of big jobs. For that reason, they use the so-called control charts, which aid in adjusting mixing, compacting, and finishing methods.

Acceptance Control by the Authority

Quality control, as practiced on road projects in the Netherlands, involves the following properties: (a) thickness of the pavement layers; (b) bitumen content of the different types of asphaltic concrete; (c) compaction (percentage of air voids) of asphalt; and (d) compressive strength of sand cement. In the past 10 years, cement concrete has not been used in the construction of motorways in the Netherlands. Because it may be used again in the near future, a system of quality control for cement concrete roads similar to that now used for flexible roads must be developed.

According to the quality control system used since 1968, one test sample is taken from every 2000 m^2 of pavement; each sample consists of two cores 10 cm in diameter drilled out of the finished construction. When testing of the four properties mentioned above leads to the conclusion that the quality is insufficient, penalties are imposed. These penalties are strictly limited to the specific sample and thus to the specific 2000 m^2 of pavement layer from which the sample was taken. The

Table 1. Results of quality control testing of road works in the Netherlands since 1968.

Property	Material	Overall Mean Value	Overall Standard Deviation	Specification or Penalty Limit
Compressive strength, MPa	Sand cement	6.0	2.3	2.0
Relative density (Marshall test), %	Sand asphalt	98.0	2.0	94.5
Air voids, %	Bitumen bound gravel	5.9	1.8	9.5
	Open-textured asphaltic concrete	4.7	1.9	8.5
	Dense asphaltic concrete	3.7	1.65	7.0
Bitumen content,[a] %	Bitumen bound gravel	5.0	0.32	5.0 ± 0.75
	Open-textured asphaltic concrete	5.5	0.31	5.5 ± 0.75
	Dense asphaltic concrete	6.5	0.29	6.5 + 0.75, 6.5 - 0.65
Layer thickness,[b] mm	Sand cement	150	17	120
		400	38	330
	Sand asphalt	120 (130)	18	100
	Open-textured asphaltic concrete	40 (43)	8	30
	Dense asphaltic concrete	40 (43)	6	33
	Total asphaltic concrete (120-mm bitumen bound gravel; 40-mm, open-textured asphaltic concrete; 40-mm dense asphaltic concrete)	200 (216)	20	180

[a]Nominal values.
[b]The layer thicknesses are nominal values, specified as minimum thicknesses. For asphalt mixtures, the prescribed quantities to be worked up are 20 kg/m² and 25 kg/m² per 10-mm nominal thickness for sand asphalt and all types of asphaltic concrete respectively. These quantities are controlled by a weighbridge and are settled. Because the normal mean densities of sand asphalt and asphaltic concrete are about 1850 kg/m³ and 2300 kg/m³ respectively, an extra safety margin of about 8 percent is calculated for each asphalt layer to ensure that minimum (nominal) thicknesses are present everywhere. Mean effective thicknesses are given in parentheses.

Figure 1. Cumulative percentage of road works for which penalties of various magnitudes have been demanded.

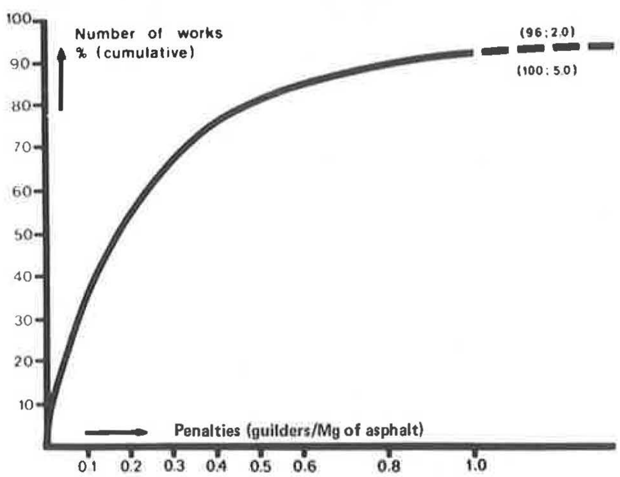
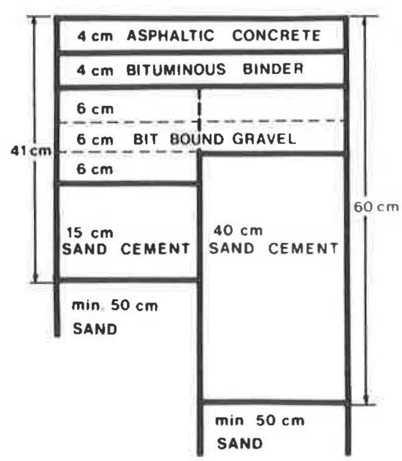


Figure 2. Standard construction for motorways in the Netherlands.



penalty per sample varies roughly in practice between 1000 and 10 000 guilders (1 guilder = $0.40) per 2000 m² and per property (e.g., the compressive strength of the sand cement subbase or the density of the asphaltic concrete top layer) of a layer. This system is essentially not a real statistical method because it is based on judging the quality of individual samples and implementing the test results on penalties. In practice, however, the number of samples is normally so great (i.e., 50 from a total controlled surface of 100 000 m²) that random testing may be justified.

The test results indicate that in most cases the number of samples of an insufficient level of quality (expressed in percentages of the total number of samples) is approximately equal to the number that transgress the specification limits, as calculated theoretically from the mean value and the standard deviation. Moreover, the system allows that 2 percent of the number of samples may give results below the specification limits without incurring a penalty. Higher percentages lead to reduced payments.

Results

The most important results of the quality control of more than 250 road construction jobs since 1968 are summarized in Table 1 and Figure 1 and in the table below (on the average, 1 Mg of asphalt in the Netherlands costs 50 guilders):

Highest Penalty (guilders per megagram of asphalt)	Percentage of Controlled Projects (cumulative)	Highest Penalty (guilders per megagram of asphalt)	Percentage of Controlled Projects (cumulative)
0	9	1.0	91
0.1	35	2.0	96
0.2	54	3.0	98
0.3	67	4.0	99
0.4	77	5.0	100
0.5	80		

These projects covered areas of at least 50 000 m²; most of them measured 100 000 to 200 000 m² and some still more. The majority of the works thus had lengths of 10 to 20 km with a mean lane width of about 10 m.

The decision on whether the quality of work on a project is good or poor does not depend on any one property because every work is judged according to the results of testing for at least three or four properties: layer thickness, strength of soil cement (if present), and density and bitumen content of asphalt mixtures. Moreover, this testing deals with at least three or four different layers: sand cement (15 to 40 cm) or sand asphalt (10 to 12 cm), bitumen-bound gravel (12 to 24 cm), open-textured asphaltic concrete (4 to 8 cm), and dense asphaltic concrete (4 cm) (Figure 2).

In summary, the whole system is generally a combination of about 10 different quality control subsystems; the risks are thus spread over all parts of the construction. Therefore, if a penalty of 1 percent has to be imposed for insufficient strength of the sand cement and if no penalties are imposed for the other properties and layers, the overall reduced payment is limited to about 0.2 percent of the total value of the construction. Thus, when the total reduced-payment penalty for a job is very high, one can conclude that the quality of the work as a whole is poor.

PRINCIPLES OF THE NEW STATISTICAL QUALITY CONTROL SYSTEM

The nonstatistical method of quality control applied in the Netherlands since 1968 has some drawbacks. According to modern theory, these drawbacks can be overcome by using statistics. In a statistical testing system, the interpretation of test results on the basis of mean values and standard deviations takes the place of interpretation based on testing of individual samples.

In recent years, such a completely statistical system of quality control has been developed on the basis of the results of tests conducted on the 250 road projects controlled for quality in the past 10 years (Table 1). This system must be such that, for projects of an acceptable quality level, only small, incidental penalties can occur. To achieve this, a contractor's risk of 0.05 is generally chosen, which means for the contractor a 95 percent chance of approval (or acceptance) of the work or a 95 percent chance of incurring no penalties.

The system is now applied in the following way: From n samples taken at random, the physical properties (layer thickness, bitumen content, compaction, density, and compressive strength) are determined. From the n results, mean values ($\bar{x}$) and standard deviation (s) are calculated according to the following well-known formulas:

$$\bar{x} = \Sigma x_i/n \qquad (1)$$

and

$$s = \sqrt{\Sigma(\bar{x} - x_i)^2/(n - 1)} \qquad (2)$$

On the basis of these values, $(R_{max} - \bar{x})/s$ or $|\bar{x} - R_{min}|/s$ is calculated, where R is the quality (or penalty) limit, either a maximum or a minimum. If this critical value is equal to or greater than the quality number (Q), the amount of the penalty imposed depends on the level of the critical value. Q values can be calculated with the help of statistical formulas that are not dealt with in this paper. Fixed values for Q for the various properties given in Table 1 are given below:

Property	Quality Number
Compressive strength	1.40
Relative density	1.40
Air voids	1.60
Bitumen content	1.60
Layer thickness	
Sand cement	1.40
Sand asphalt	1.40
Open-textured asphaltic concrete	1.40
Dense asphaltic concrete	1.40
Total asphaltic concrete	1.40

Random Samples

In the new system, 40 samples are drilled per job—always at random—over a maximum lot size of 200 000 m^2. When the lot is bigger than 200 000 m^2, it is divided into two equal lots and 20 test samples are taken from each lot. Because of the effort involved in the testing method, determination of bitumen content is limited to only 20 samples.

Statistical quality control is generally too expensive for use on lots smaller than 20 000 m^2. In this case, more frequent and thorough supervision and inspection by the engineer during production can ensure sufficient quality control.

Quality Criteria

As mentioned above, test results for the 250 jobs controlled nonstatistically since 1968 have been worked up statistically; i.e., the mean value ($\bar{x}$) and the standard deviation (s) have been calculated for every controlled property. Overall mean $\bar{\bar{x}}$ and $\bar{s}$ values for all the jobs have been determined from these data and are used to deduce the criteria for R and Q. Thus, the deduction is fully based on the test results acquired in practice on all 250 large jobs (both the good and the bad). The mean values $\bar{\bar{x}}$ ($\sim \mu$) and s ($\sim \sigma$) are thus defined as being representative of a standard job (Table 1). The penalty system is only applied for three layer thicknesses: (a) the total thickness of all asphaltic concrete layers together; (b) the thickness of the base of sand cement or sand asphalt, which is central to the structural design of the pavement; and (c) the thickness of the wearing course, which is highly important for the durability of the surfacing.

Penalties

Based mainly on experience with the nonstatistical quality control system since 1968, the following relations have been determined between the magnitude of penalties (K) and the number of test results (B) [B = percentage defective of the total percentage of samples (n)] that is below the fixed penalty limit (R) as calculated from the mean value ($\bar{x}$) and standard deviations: $K = 0.3B - 1.0$ for bitumen content and percentage of air voids for asphaltic concrete, and $K = 0.3B - 2.0$ for layer thicknesses, relative density of sand asphalt, and compressive strength of sand cement.

TRIAL USE OF THE STATISTICAL SYSTEM

It was agreed in deliberations between the State Road Laboratory and the committee representing the road contractors' organization that, during 1976 and 1977, between 10 and 20 road construction jobs should be judged according to both the traditional nonstatistical system and the new statistical system of quality control and that the comparative results should be thoroughly discussed. This might possibly lead to adjustments of the statistical system. (A method for taking extreme values into account had already been proposed.)

CONTROL OF SURFACE CHARACTERISTICS

Surface properties, roughness, and skid resistance are an important part of acceptance control. Testing for roughness and skid resistance is not done according to fully statistical procedures because it is not easy to express traffic safety in terms of statistical figures. However, to limit the number of measurements, at first only 30 percent of the road length is controlled for roughness and skid resistance. The sections to be measured are chosen at random. When the results of these limited measurements and the appearance of the road surface indicate insufficient quality, especially insufficient skid resistance, more measurements are carried out and, if necessary, the whole surface of the finished work is controlled.

Roughness

Until 1975, roughness was always controlled by using a rolling straightedge 3 m in length. If five or more deviations of >3 mm from the even profile were found per 100-m measuring section, penalties were imposed. When more than a few incidental deviations of >5 mm were found, surface roughness had to be corrected by the contractor by "shaving."

Since 1976, the viagraphe (Figure 3) has been used instead of the rolling straightedge for controlling road-surface roughness. When the so-called percentage of deviation (C_5) is greater than 2, a penalty is required. Deviations greater than 5 mm call for shaving by the responsible contractor. In special cases, when roughness is so bad that correction by shaving will not result in a sufficiently even surface, the contractor is forced by special contract conditions to adjust the surface by constructing an extra 40-mm top layer on the finished surface.

Skid Resistance

Since 1967, it has been required in the Netherlands that, before highways are opened to traffic, new finished pavements must be quality-controlled for skid resistance. This is done by using a standard measuring vehicle with a retarded wheel (86 percent slip) on a wet surface and at a measuring speed of 50 km/h (see Figure 2 in the paper by Elsenaar and van de Fliert elsewhere in this Record). When the measured friction coefficient is lower than 0.56, penalties are imposed. When it is lower than 0.51, the contractor must also correct the surface of the asphaltic concrete by treating it with white spirit and crushing sand. Such a treatment effectively removes the excess bitumen from the surface until a friction coefficient of at least 0.56 is reached. Because this treatment is rather costly, the prospect that it will be required works as a good preventive measure. Slippery sections on new roads are now rare, and normally the finished surface amply meets the specifications on skid resistance.

Figure 3. Viagraphe.

One prescription plays an important role in the acceptance control of skid resistance. For about 10 years, it has been prescribed in Dutch road contracts that asphaltic concrete top layers must be spread with about 2 kg/m^2 of fine chippings 2 to 5 mm in size before the surface is finished by rolling. This treatment effectively prevents the occurrence of the "initial slipperiness" of new asphaltic concrete top layers, which is caused by an excess of bitumen in the surface.

COSTS AND BENEFITS OF QUALITY CONTROL

Neither production control nor acceptance control alone can give 100 percent assurance of the total average quality of the work or prevent exceptional cases of poor quality. Research is, and will continue to be, based on random checks. An exact and ideal match cannot be achieved between optimum quality and the biggest possible profit for the contractor nor between the highest possible level of quality and the lowest possible cost for the governmental authority.

For this reason, the primary object of penalty clauses cannot be to secure adequate compensation for inferior quality. Any theoretical rules drawn up with this end in view would be affected by so many factors (e.g., subsoil, traffic trends, maintenance methods, and weather conditions) that an absolutely exact approach would be impossible. The introduction of an apparently watertight system based on cost-benefit investigation would therefore lead to unfairness to the contractors executing the jobs.

Brouwers of the State Road Laboratory has devoted a special study to the problem of costs and benefits. The conclusion of that study is that the penalties incurred for quality deficiencies under the new quality control system will be less by approximately a factor of 3 than the amount that would theoretically be involved if the effects of these quality deficiencies on the life of the pavement were expressed in loss of value. The following summary of the observations made in Brouwers' research includes findings relating to layer thickness and voids and bitumen content of an asphalt structure. The reduction in the service life of a pavement is calculated on the basis of specific deficiencies in these characteristics.

Layer Thickness

The loss resulting from pavement layers that are too thin can be quantified on the basis of data that relate layer thickness to traffic parameters. Use was made of the thickness design formula applied in the AASHO Road Test, which gives the relation between the number of load repetitions (n)—up to the moment when a specified serviceability index (p) is reached—and thickness expressed as thickness index (D) for a specified wheel load (P) in megagrams. It is possible, by using a thickness design formula, to calculate how many equivalent 10-Mg axle-load repetitions can take place before the point is reached where P equals, for instance, 2.5.

Pavement life can be calculated by assuming a standard traffic density in the first year for the type of road in question and allowing for a constant annual increase. Then the life of a similar pavement of less thickness can be calculated in the same way. It is also possible to calculate, on the basis of a normal distribution, what percentage of the total pavement has a layer thickness greater or smaller than the specified values, if either the average thickness (μ) or the standard deviation (σ) is varied. The reduction in pavement life can thus be calculated in each case and, finally, the loss resulting from having to undertake premature reconstruction and maintenance can be calculated on the basis of comparison with the normal structure. The details of these calculations are not treated here.

Costs determined in this way to compensate for deviations in layer thickness can then be compared with the penalty calculated on the basis of the penalty clauses; the penalty is determined for the same varying average values and standard deviations. Use of this method has resulted in a fairly constant average ratio between costs and benefits of 1:3.

Voids

Calculations of the reduction in pavement service life resulting from excessive voids caused by insufficient compaction can be done on the basis of fatigue tests on asphalt carried out by researchers in various countries. It can be concluded from a study of the published data that a reduction in pavement life of 25 percent for each percentage increase in voids is a cautious, relatively low estimate. Theoretical calculations have now shown that the cost to the highway authority as a result of the reduction in pavement life is on the average five to seven times as large as are the profits that result from penalties imposed because of excessive void percentages. It is also possible to calculate the reduction of the rigidity of asphalt layers because of excessive voids; the resulting effect on the increase in ground pressure (and thereby on the reduction of pavement life if ground pressure is also a decisive factor in the life of the pavement) can then also be calculated.

Bitumen Content

The effects of deviations in bitumen content can be calculated in the same way as can the effects of voids. A study of various published data on the relation between the binder content of asphalt mixtures and pavement service life leads to the conclusion that, for every reduction of 0.3 percent in bitumen content, service life is reduced on average by a factor of 0.75, provided that the bitumen content does not fall to very low values. From these calculations it can be concluded that the costs due to shorter life are on average three to four times as great as the benefits derived from the penalties. The following table gives data on the percentage cost attributable to the reduced service life of pavement and the percentage profit resulting from penalties or allowances imposed on the contractor:

Deficient Property	Cost (%)	Profit (%)	Deficient Property	Cost (%)	Profit (%)
Thickness	100	30 to 40	Bitumen content	100	25 to 35
Density	100	15 to 20	Mean	100	30

The overall cost-benefit conclusion drawn from this study was that costs caused by quality deficiencies are still several times greater than the benefits derived from penalties. Costs and benefits related to the quality control of road construction are therefore far from being in balance.

SUMMARY AND CONCLUSIONS

Since 1968, a uniform, adequate, and fair system of quality control of pavements has been applied in the Netherlands. The system, which has been developed in deliberations between the government and road construction contractors, is characterized by complete daily production control by the contractor and limited, random acceptance control by the highway authority after completion of the work.

According to the current nonstatistical quality control system, one sample per each 2000 m^2 of pavement is tested for layer thickness, strength of soil cement, and density and bitumen content of asphalt. When the quality does not meet specifications, penalties are applied in proportion to the extent of the deviation.

On the basis of experience gained on more than 250 major road works, the system has now been developed into a fully statistical control method. According to this statistical system, from 20 to 40 samples are tested on lots at most 200 000 m^2 in area, and penalties are imposed according to the mathematical formulas discussed in this paper.

Surface characteristics—skid resistance and roughness—are measured at random on 30 percent of the pavement surface. Penalties are applied if test results do not meet the requirements. When the results are below specific safety limits, the surface must be restored at the contractor's expense.

A theoretical study on the decrease in the service life of pavements caused by deficiencies in the thickness, density, and bitumen content of asphalt layers leads to the conclusion that the penalties applied under the quality control system are, on the average, about three times lower than the costs that would theoretically be necessary to compensate for the loss in pavement service life.

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Judgment of Concrete Quality in Transportation Structures

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This paper deals with the present method of judgment of the quality of fresh and hardened concrete used in transportation structures in Czechoslovakia. Standard methods and criteria are presented for estimating properties of fresh concrete mixes and hardened concretes for different types of structures. In the case of hardened concretes, destructive and nondestructive methods of testing concrete properties are analyzed and evaluation techniques are given. The problems of quality control of cements and aggregates are studied. The judgment of the acceptability of fresh concretes in relation to their composition and workability is analyzed. Requirements are presented for such properties as concrete strength and properties related to short- and long-term deformation. Sclerometric and acoustic methods of nondestructive testing are discussed. Various methods for the judgment of acceptability of concrete are analyzed by using large and small samples and standard Czechoslovak specifications. Statistical evaluation is emphasized. Acceptability criteria for safety, homogeneity, and economy are presented.

The intent of this paper is to show the complexity of the judgment procedure used in the quality control of concrete on the basis of current Czechoslovak standards. Judgment of concrete quality involves the choice of the physicomechanical characteristics to be tested and the testing procedures and evaluation methods to be used. This paper addresses all of these problems. The possibility of using a refined procedure of statistical quality control for concrete produced over longer periods is also discussed.

CONTROL OF QUALITY PARAMETERS

Because transportation construction deals with different kinds of structures and these structures involve different requirements, the control procedure used must also be different. The most pronounced difference in estimating quality parameters occurs between concretes used for road and airport pavements and concretes used in other transportation-related structures.

Quality parameters are estimated by using three categories of tests that correspond with the different time periods of a construction project. Some of these tests are prescribed by standards; the rest are optional or recommended. The test categories are as follows:

1. Agreement tests carried out before the start of the construction work;
2. Control tests, which include the production control tests carried out by the producer of the structure and the official control tests carried out by government agencies; and
3. Acceptance tests, which are carried out by the consumer of the structure.

TESTS FOR ROAD AND AIRPORT PAVEMENT CONCRETES

According to Czechoslovak standards, the following tests are carried out for road and airport pavement concretes.

Agreement Tests

The following agreement tests are prescribed:

1. For concrete mix and concrete—workability of the concrete mix, air content when air-entrainment admixtures are used, bulk density of fresh compacted concrete, and flexural strength of concrete;
2. For cement—estimation of the normal consistency of cement paste, initial set and time of setting, fineness, volume stability, and strength;
3. For aggregate—grading, bulk density of loose and compacted aggregate, specific gravity, contents of sulfur compounds, water absorption, contents of clay, sand equivalent, humus content, and humidity;
4. For water—testing only in cases of doubt; and
5. For admixtures—in addition to the prescribed tests, proof of the influence of admixtures on the concrete mix and concrete.

The following agreement tests are recommended:

1. For concrete mix and concrete—bulk density of concrete, compressive strength of concrete, splitting tensile strength, volumetric changes, Young's modulus, deformation properties, water absorption, water permeability, and frost resistance;
2. For cement—resistence to cracking, specific gravity, and volumetric changes of cement mortar; and
3. For aggregate—particle shape, mica particles content, influence of the aggregate on the volume changes of concrete, and abrasion of coarse aggregate by the Los Angeles machine.

Agreement tests are essential for gaining permission to start the production.

Control Tests

Control tests are carried out continuously over the entire period of construction. The sampling of concrete mixes is carried out as often as necessary to ensure the presence of the required properties according to prescribed and recommended tests; these tests involve a control of components, concrete mixes, and concretes similar to that applied in the agreement tests.

The extent of production control tests is prescribed by standards, as follows: Workability and air content are to be examined at least once during a shift; flexural strength must be assessed on three specimens 15 by 15 by 70 cm in size for each 1000 m^2 of pavement; the quality parameters of cement and aggregate must be evaluated at least once for each 3000 m^2 of pavement; and the test results should be summarized in a test record that is kept at the site for inspection purposes.

Official control tests are carried out according to a decision made by the pertinent supervising authority. This authority decides which concrete properties or components are to be tested.

Acceptance Tests

Acceptance tests are only used to control finished pavements. The number of samples is determined according to the pavement area. For up to 2000 m^2, two samples are tested. For every additional 2000 m^2, another specimen is tested. Nondestructive methods can be used to estimate pavement quality provided an experimental relation between the destructive and the nondestructive test values has been determined. Combining several nondestructive methods is advisable to increase the exactness of values determined by such methods. The flexural strength of concrete is determined according to the results of production control tests. In acceptance of the pavement, it is essential to assess whether (a) the design strength was achieved and (b) the coefficient of variation computed from the production control tests shows lower than prescribed values. The homogeneity of the cast concrete is tested by nondestructive methods (e.g., the pulse velocity method). If the splitting tensile test or nondestructive testing is done on cores, the relation between these values and the splitting tensile strength tests carried out with normally cast specimens must be determined.

The control procedure for transportation-related structures other than pavements also involves the determination of properties of concrete mix components; properties of concrete mixes in the processes of mixing, transport, and casting; curing of hardened concrete during casting; 28-d cube strength; and cube strength at the time of formwork removal and structure loading. Other properties prescribed by the design—e.g., flexural strength, water permeability, frost resistance, and volume changes—are also tested. The control procedure is divided into the same test categories as those used for pavements—i.e., agreement, control, and acceptance tests.

Control tests are carried out for each 500 Mg of the material. In prestressed structures, the number of tests is prescribed by the design. The workability and the air content of the mix are examined at least once a day. If doubts arise, the concrete composition should be checked. In hardened concrete, the cube strength is controlled; other properties are examined only when such examination is prescribed by the design. Samples are taken from each 200 m^3 of concrete. In mixing plants that use semiautomatic or fully automatic mixing machines with a capacity greater than 300 m^3/d, samples are taken from each 500 m^3. In prestressed structures, one sample is taken from each 25 m^3 of concrete in such a way that a sample is taken when the prestressing is applied and another sample is taken to determine the 28-d strength. The strength at the time when prestressing is applied can be found by using a nondestructive test method; the strength values are derived from a calibration diagram.

METHODS OF DETERMINING PROPERTIES OF CONCRETE MIXES AND CONCRETES

Testing Concrete Mix Properties

The workability of plastic concrete mixes is assessed by means of slump tests. For no-slump concretes, the VeBe method is prescribed. Czechoslovak standards allow the use of two methods: the Skramtayev test and the technical viscosimeter test.

The air content in the concrete mix is determined by the volumetric and pressure method. The analysis of the concrete mix can be performed either by sieve analysis or by drying.

Testing Hardened Concrete Properties

The compressive strength of concrete is determined by testing at least three specimens and using the mean value of the three tests as the test result. Compressive strength is estimated by testing cube strength, cylinder strength, and prism strength.

The basic cube has a side length of 20 cm. Standards allow the use of cubes with side lengths of 10, 15, 30, and 40 cm, but the strength results determined on these cubes are converted to the strength obtained on 20-cm cubes. The basic cylinder has a height of 30 cm and a diameter of 15 cm. Cylinders of different sizes can also be used if the results are converted to conform to the basic cylinder size. The basic specimen for determining prism strength measures 15 by 15 by 60 cm. When an appropriate strength coefficient is applied, 10 by 10 by 40-cm and 20 by 20 by 80-cm prisms can also be used. The direct tension test is carried out on 15 by 30-cm cylinders and 10 by 10 by 30-cm prisms by using clamps glued on the ends of the specimens. The basic specimen used in determining flexural strength is a 15 by 15 by 60-cm beam; beams 10 by 10 by 40 cm and 20 by 20 by 80 cm can also be used. The beams are tested by using third-point loading. Splitting tensile strength is determined on cubes loaded parallel with their diagonal, and the basic test specimen has a side length of 20 cm. Splitting tensile strength can also be determined on cylinders. The size of the basic specimen in this case is 15 by 30 cm, but specimens of different sizes can be used.

In the determination of Young's modulus of elasticity, the specimens used are of the same size as those used for the testing of prism and cylinder strength. The initial Young's modulus is determined at a stress equal to 20 percent of the crushing strength. Shrinkage, swelling, and creep of concrete are investigated on specimens used for the prism strength tests.

Water permeability is tested on at least three specimens. These specimens are plates measuring 30 by 30 by 15 cm or cylinders measuring 15 by 30 or 15 by 15 cm. The frost resistance of concrete is estimated for at least three beams that have the same dimensions as those used for testing flexural strength. The freezing of specimens takes place in the air, and the thawing is done in water. One freeze-thaw cycle consists of 4 h of freezing at a temperature of -20°C and 2 h of thawing at a temperature of +20°C. Frost resistance can be investigated by using destructive or nondestructive methods.

Czechoslovak standards distinguish the following methods for nondestructive testing of concrete properties: the pulse velocity method; the resonance frequency method; and four types of surface-hardness tests—(a) the Waitzman method, (b) the Bauman-Steinrück-Franck spring hammer, (c) the Schmidt test hammer, and (d) the mechanical pick hardness tester.

The Waitzman method uses a steel ball projected by hand simultaneously onto a concrete surface and a control steel bar of known mechanical properties. The compressive strength is determined by comparing and evaluating the diameter of both indentations.

The Bauman-Steinrück-Franck hammer consists of a spring-controlled mechanism housed in a tubular frame. The tip of the hammer is fitted with a ball, and the impact is effected by placing the hammer up against the concrete surface and triggering the spring mechanism. The diameter of the indentation is measured, and this in turn is correlated with the compressive strength of the concrete.

The Schmidt test hammer testing procedure consists of releasing a plunger from its locked position by pressing it against the concrete surface. Then the spring-loaded weight is released from its locked position, which produces an impact. While the hammer is still in its testing position, the sliding index is read to the nearest whole number. This reading is designated as the hammer rebound number. Each hammer is furnished with a calibration chart supplied by the manufacturer.

When the mechanical pick hardness tester is used, the number of impacts necessary to excavate an indentation of prescribed depth is counted and the concrete strength is evaluated from the number of impacts.

These nondestructive methods may have the character of either approximative or refined tests. In approximative tests, the strength of concrete is estimated with the help of a general calibration relation. In refined tests, relations are experimentally established.

The grindability of concrete is tested by using a machine with a circular test track. The resulting loss of weight is the measure of grindability. Concrete is also tested by means of other methods not included in Czechoslovak standards. These methods are not dealt with in this paper.

METHODS OF ESTIMATING CONCRETE QUALITY ACCORDING TO TEST RESULTS

Czechoslovak standards distinguish among seven strength classes, including concretes with a cube strength in the 6- to 55-MPa range. In each class the following characteristics are given: an upper limit for the average strengths, the value of the so-called control strength (R_{bk}) estimated with a probability of 0.90; and a further strength characteristic estimated with a probability of 0.95, the design strength that serves for the design of concrete mixes. The result of a test is understood to be the average of strengths obtained on three 20-cm cubes. The judgment of acceptability of concrete can be conventional or statistical. In both cases the average strength of all cube-strength tests must be equal to or higher than the class value of the concrete but lower than the upper limit of average strength. The samples for each control test must be taken from a different batch, and none of the obtained results can be disregarded.

Conventional Judgment

When a judgment is made according to 1 or 2 tests, none of the results must fall below 1.2 R_{bk}. When 3 to 9 tests are performed, none of the results must fall below R_{bk} or the class value. When 10 or more tests are used, a maximum of 10 percent of the results may fall below the value of 0.8 R_{bk}.

Quantitative and Qualitative Statistical Judgment

In any basic set of specimens, a maximum of 5 percent of the values may fall below a value (R_{bcu}). According to Czechoslovak standards, the results are judged according to quantitative and qualitative characteristics.

Quantitative Approach

Essentially, two main criteria and a supplementary criterion are applied in the quantitative approach. The main criteria are those of safety and economy, and the supplementary criterion is that of homogeneity. A statistical judgment made according to these criteria is based on a random sample (n) taken from a set of specimens. The average ($\bar{x}$), standard deviation (s_x), variation coefficient (v_x), and (in the case of large samples) skewness are computed for the random sample. A small sample is $16 \leq n < 100$; a large one is $n > 100$. In the case of a small sample, the theoretical model of the normal distribution is used because skewness cannot be determined with sufficient exactness for small samples. The Pearson distribution is used for large samples.

By applying the safety criterion, it can be determined whether the occurrence of a characteristic in a certain set is lower than (first-case characteristics) or higher than (second-case characteristics) a certain value (x_{cu}), but the difference must not be larger than the value that could occur with a probability (p). The decision can only be made from the random sample by using a certain predetermined reliability (q). Usually p = 0.05 and q = 0.80. In the instance of a first-case characteristic,

$$t = (\bar{x} - x_{cu})/s_x \tag{1}$$

and in the instance of a second-case characteristic,

$$t = (x_{cu} - \bar{x})/s_x \tag{2}$$

When a sample is small, the t values are compared with values (t_{min}) and t_{max}), which were derived from the "noncentral" distribution (t) and are a function of probability (p) and reliability (q). If $t > t_{max}$, the concrete is satisfactory from the safety point of view, and if $t < t_{min}$, the concrete is unsatisfactory from this point of view. If $t_{min} \leq t \leq t_{max}$, no decision can be made and the random sample must be enlarged until a decision can be made.

By applying the economy criterion, it can be determined if the computed average ($\bar{x}$) is within the prescribed limits. The concrete is satisfactory if $\bar{x}_{min} \leq \bar{x} \leq \bar{x}_{max}$. By applying the homogeneity criterion, it can be determined if the computed coefficient of variation (v_x) is lower than the highest allowable coefficient of variation (v_{xmax}). The concrete is satisfactory if $v_x \leq v_{xmax}$. The homogeneity of concrete can also be determined by the pulse velocity method. If $n \geq 16$ and the coefficient of variation of the pulse velocity is ≤ 0.05, the homogeneity of concrete can be considered to be satisfactory.

The quantitative approach is usually used to determine strength in compression and tension as well as bulk density. Czechoslovak standards allow the use of the quantitative judgment approach in other cases too, but these should be either agreed on or ordered by an authority.

Qualitative Approach

Qualitative judgment is based on the model of the binomial distribution. The aim is to find out whether the number of faulty elements coming from concrete production or production of concrete elements is larger than agreed on or ordered. The portion of faulty specimens (Z) in a random sample is determined and compared with the value (Z_{crit}). A decision concerning the whole set can be made from the random sample only by using a certain predetermined reliability. In this way the range within which Z_{min} and Z_{max} occur can be determined. In the case of a small sample, the concrete is satisfactory if $Z < Z_{min}$ and is unsatisfactory if $Z > Z_{max}$. No decision can be made if $Z_{min} \leq Z \leq Z_{max}$. In the case of a large sample, Z_{min} and Z_{max} merge with Z_{crit}.

PROPOSED METHOD FOR REFINED STATISTICAL QUALITY CONTROL OF CONCRETE

The production of concrete is usually statistically tested over a time period (T). The question arises, Can the concrete production over a long period be characterized by a standard population (S) of size (N) of parameter value (X) obtained by the experiment (E_o) typical for the concrete production during the whole period (T), i.e., by a standard population with an approximate normal distribution $N(\mu, \sigma^2)$? In other words, are the following two requirements on which the Czechoslovak standards are based fulfilled?

1. Over the entire period in which quality control is carried out, are the conditions influencing the tested characteristic held constant?
2. Are the test conditions (i.e., test procedure, specimen size, age of concrete) held constant?

Let the period (T) be divided into $k \geq 2$ relatively short periods ($C_1, \ldots, C_k$), which do not overlap each other. Assume that the following conditions are fulfilled:

1. Concrete production is carried out during the periods ($C_1, \ldots, C_k$) under conditions that do not influence each other.
2. Concrete production in periods ($C_1, \ldots, C_k$) can be characterized by standard populations ($S_1, \ldots, S_k$) of the same size (M) of characteristic values ($X_1, \ldots, X_k$) obtained by experiments ($E_{o,1}, \ldots, E_{o,k}$) typical for the production in $C_1, \ldots, C_k$.
3. The standard populations ($S_1, \ldots, S_k$) have an approximately normal distribution [$N(\mu_1, \sigma_1^2), \ldots, N(\mu_k, \sigma_k^2)$], where $\mu_1, \ldots, \mu_k$ are the mean values and $\sigma_1^2, \ldots, \sigma_k^2$ are the variances of the standard populations ($S_1, \ldots, S_k$).

It is evident that the first and second conditions are fulfilled only in the case in which the following two conditions are also satisfied:

$$\sigma_1^2 = \ldots = \sigma_k^2 = \sigma^2 \qquad (3)$$

$$\mu_1 = \ldots = \mu_k = \mu \qquad (4)$$

If only one of these conditions is fulfilled, it is certain that the first requirement in the list given above is not fulfilled, and usually neither is the second requirement.
Two problems arise:

1. Is the first condition (Equation 3) fulfilled or not?
2. Is the second condition (Equation 4) fulfilled or not?

Methods of solving these problems have been proposed. It should be noted, however, that the method of solving problem 2 can be applied only when the answer to problem 1 is positive.

In solving the two problems, assume that the time period (T) was divided into $k \geq 5$ periods ($C_1, \ldots, C_k$) and that random samples ($V_1, \ldots, V_k$) of equal size ($m \geq 5$) taken from standard populations ($S_1, \ldots, S_k$), characterizing concrete production in periods ($C_1, \ldots, C_k$) are available.

Problem 1 (Equation 3) is solved as follows. On the significance level ($\alpha = 0.05$), the hypothesis (H)—$\sigma_1^2 = \ldots \sigma_k^2$—is tested against alternative ($\overline{H}$) by asserting that at least two of the variances ($\sigma_1^2, \ldots, \sigma_k^2$) are different. The test is carried out by using the Bartlett formula (1):

$$B = 2.302\,59/\{1 + [(k+1)/3k(m-1)]\} \left[\log S^2 - (1/k)\sum_{i=1}^{k} \log S_j^2\right] \qquad (5)$$

where $S_1^2, \ldots, S_k^2$ are variances of random samples ($V_1, \ldots, V_k$),

$$S^2 = (1/k)\sum_{j=1}^{k} S_j^2 \qquad (6)$$

Log S_j^2 and S^2 are Briggs' logarithms of S_j^2 and S^2. The value (B) is compared with the critical value [$X^2_{0.05}$ ($f = k - 1$)] of the X^2 distribution with ($f = k - 1$) degrees of freedom. One of the following decisions is made:

1. If $B > X^2_{0.05}$ ($f = k - 1$), the hypothesis (H) is rejected in favor of alternative ($\overline{H}$).
2. If $B \leq X^2_{0.05}$ ($f = k - 1$), the hypothesis (H) is not rejected in favor of alternative ($\overline{H}$). S^2 is then assumed as an impartial estimator of variance (σ^2) i.e., $\sigma_1^2 = \ldots = \sigma_k^2$, i.e., an estimator with $f = k(m-1)$ degrees of freedom.

Problem 2 (Equation 4) is solved as follows. On the significance level ($\alpha = 0.05$), the hypothesis (H)—$\mu_1 = \ldots = \mu_k$—is tested against alternative ($\overline{H}$) by asserting that at least two of the mean values ($\mu_1, \ldots, \mu_k$) are different. The test is carried out by using the following formula:

$$F = \left\{[1/(k-1)]\sum_{i=1}^{k}(\bar{x}_j - \bar{x})^2\right\} \Big/ [S^2/km(m-1)] \qquad (7)$$

where $\bar{x}_1, \ldots, \bar{x}_k$ are mean values of random samples ($V_1, \ldots, V_k$),

$$\bar{x} = (1/k)\sum_{j=1}^{k} \bar{x}_j \qquad (8)$$

and S^2 is the impartial estimator of variance ($\sigma^2 = \sigma_1^2 = \ldots = \sigma_k^2$) computed by using Equation 6. F is compared with the critical value {$F_{0.05}$ [$f_1 = k - 1$; $f_2 = k(m-1)$]} of the distribution (F) with $f_1 = k - 1$ and $f_2 = k(m-1)$ degrees of freedom.

One of the following decisions is made:

1. If $F > F_{0.05}$ [$f_1 = k - 1$; $f_2 = k(m-1)$], the hypothesis (H) is rejected in favor of alternative ($\overline{H}$).
2. If $F \leq F_{0.05}$ [$f_1 = k - 1$; $f_2 = k(m-1)$], the hypothesis (H) is not rejected in favor of alternative ($\overline{H}$). The value ($\bar{x}$) is then assumed as an impartial estimator of the average ($\mu = \mu_1 \ldots \mu_k$).

Procedure in Case of Rejection of Equality of Mean Values

In solving problem 2, it was necessary at a significance level of $\alpha = 0.05$ to reject the hypothesis (H)—$\mu_1 = \ldots = \mu_k$—in favor of alternative ($\overline{H}$)—i.e., at least two of the mean values $\mu_1, \ldots, \mu_k$ are different. The question arises whether among mean values ($\mu_1, \ldots, \mu_k$) there are not two or more groups of equal mean value.

For the solution, the following procedure is proposed. The mean values ($\bar{x}_1, \ldots, \bar{x}_k$) of the random samples ($V_1, \ldots, V_k$) are arranged according to their magnitude:

$$\bar{x}_1' \geq \bar{x}_2' \geq \ldots \geq \bar{x}_{k-1}' \geq \bar{x}_k' \qquad (9)$$

Denote the mean values ($\mu_1', \ldots, \mu_k'$), the estimators of which are the mean values ($\bar{x}_1', \ldots, \bar{x}_k'$). The question is whether the hypothesis (H)—$\mu_A = \mu_1' = \ldots = \mu_i' = \mu_B = \mu_{i+1} = \ldots = \mu_k$—for certain $i = 1, \ldots, k - 1$ should not be rejected in favor of alternative ($\overline{H}$)—$\mu_A \neq \mu_B$. Schaffe (2) has proved the following concerning $\hat{L}_i^2$ values:

$$\hat{L}_i^2 = \{[(\bar{x}_1' + \ldots + \bar{x}_i')/i - (\bar{x}_{i+1}' + \ldots + \bar{x}_k')/(k-1)] \div [\sqrt{(k-1)/m}\, S\sqrt{(1/i)+(1/k)}]\}^2 = \{[k(\bar{x}_1' + \ldots + \bar{x}_i') - i(\bar{x}_1' + \ldots + \bar{x}_k')] \div [\sqrt{(k-1)/m}\, S\sqrt{ik(k-i)}]\}^2 \quad (10)$$

where S^2 is the impartial estimator of the variance ($\sigma^2 = \sigma_1^2 = \ldots = \sigma_k^2$), computed according to Equation 6. For each i = 1, ... k - 1, the values have a distribution (F) with $f_1 = k - 1$ and $f_2 = k(m - 1)$ degrees of freedom. This can be used in the following way. All K_i values are computed by using the following formula:

$$K_i = [k(\bar{x}_1' + \ldots + \bar{x}_i') - 2(\bar{x}_1' + \ldots + \bar{x}_k')]/\sqrt{ik(k-i)} \quad (11)$$

i = 1, ..., k - 1, and the maximum value is chosen from them. This maximum K_i value obviously corresponds to the maximum $\hat{L}_i^2$ value, and so does the maximum F value. This leads to the following conclusion: For i, for which the $\hat{L}_i^2$ value is maximal on the lowest possible significance level (α), it is necessary to reject the hypothesis $H - \mu_A = \mu_B$—in favor of alternative $(\bar{H}) - \mu_A \neq \mu_B$.

By means of this procedure the concrete production in the T time period is divided into two approximately equal normal productions, one of which is characterized by standard population (S_A) of size (N_A = iM) with an approximately normal distribution [$N(\mu_A, \sigma^2)$] and the other by the standard population (S_B) of size [N_B = (k - i)M] with an approximately normal distribution [$N(\mu_B, \sigma^2)$].

From S_A comes the random sample (V_A) of size (N_A = im), and from S_B comes the random sample (V_B) of size [n_B = (k - i)m]. Experience shows that it is usually sufficient to divide the concrete production obtained in period (T) into two approximately normal productions.

Proof may be obtained at a significance level (α = 0.05) by using the test described previously and substituting the first time into the numerator of Equation 7

$$(1/i)\sum_{j=1}^{i} (\bar{x}_j' - \bar{x}_A')^2 \quad (12)$$

where $\bar{x}_A = (1/i) \sum_{j=1}^{i} \bar{x}_j'$, and the second time

$$[1/(k-i)] \sum_{j=i+1}^{k} (\bar{x}_j' - \bar{x}_B)^2 \quad (13)$$

where $\bar{x}_B = [1/(k-i)] \sum_{j=i+1}^{k} \bar{x}_j'$. $\bar{x}_A$ and $\bar{x}_B$ are impartial estimators of mean values (μ_A) and μ_B.

Evaluation of Production Quality

In evaluating the quality of both approximately normal lots into which the production obtained in the period (T) was divided, the quantitative method described previously in this paper can be used.

If, in the evaluation, the case ($t_{min} \leq t \leq t_{max}$) occurred, it is proposed that the original value (P_o = 0.05 to P_o' = 0.10) should be increased and it should be verified whether hypothesis (H)—P = P_o'—should be rejected in favor of alternative ($\bar{H}$)—P < P_o'. If, according to the verification, H should be rejected in favor of $\bar{H}$, one lot of concrete or the other should be considered satisfactory. When H should not be rejected in favor of $\bar{H}$, one lot or the other should be considered unsatisfactory.

STANDARDIZATION OF METHODS

The mode of choice of the physicomechanical concrete properties to be tested and the testing methods and evaluation procedures have not as yet been unified on an international basis. For this reason, any discussion dealing with this problem is welcome because it may accelerate the process of international standardization.

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