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33 Construction
40 Maintenance, General

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FOREWORD

The subjects covered in the 5 papers in this RECORD include techniques in bituminous concrete construction, quality assurance programs used in bituminous concrete production and construction, and quality assurance per se. The papers should be of interest to materials and construction engineers and to persons having the responsibility for writing specifications, particularly to the latter because of the increased emphasis being placed on statistically oriented specifications.

Corlew and Dickson present a new concept demonstrating the effectiveness of preheating an existing asphalt pavement, or a base course, prior to cold-weather overlaying with hot-mixed seal coats or thin wearing surfaces. From the use of computer programs verified by laboratory tests, they found that preheating the base increased the allowable time for compaction. The preheat times required appear to be consistent with the production rates for thin lifts.

Santoro, Affterton, and Walz give an evaluation of compaction equipment used on bituminous base courses. The use of 2 vibratory rollers and a tandem roller was compared to the use of standard 3-wheel breakdown and tandem finish rollers. Comparisons were made of both multiple- and thick-lift paving methods. The conclusions deal with roller and thick-lift compaction effectiveness and with pavement riding quality.

Gorman discusses the first application in Illinois of an end-result type of specification in which the contractor had complete responsibility for the design, control, and placement of the mix. Characteristics discussed are asphalt content and gradation from uncompacted mix samples and the density and thickness of the compacted pavement. In an ancillary experiment, comparisons were made between results from uncompacted mix samples and those from pavement cores. A pertinent discussion is given by Tunnicliff.

Hughes describes a statistical quality assurance and acceptance specification for asphaltic concrete under which nearly 3 million tons of plant mix have been produced. A workable specification is included.

Kühn, Walker, and Savage discuss a rational system for the application of statistical quality control procedures to highway construction. Included are representative coefficients of variation, desired frequency of sampling, and suggested lot sizes. An interesting discussion is given by Davis.

—C. S. Hughes

COLD-WEATHER PAVING OF THIN LIFTS OF HOT-MIXED ASPHALT ON PREHEATED ASPHALT BASE

J. S. Corlew, Atlantic Richfield Company; and
Philip F. Dickson, Colorado School of Mines

Base preheat is the application of thermal energy to the base prior to the placement and compaction of hot-mixed asphalt pavements. The greatest potential use of base preheat is in the placement of hot-mixed seal coats or thin wearing surfaces on existing asphalt pavements or asphalt bases in the early spring or in the fall. Bench scale laboratory tests were conducted in which test specimens having 4-in. diameter asphalt bases were preheated with a direct-fired propane heater. Initial base temperatures ranged from 20 to 50 F. The computer program that was developed and tested experimentally for base preheat was combined with a computer program for cooling of hot-mixed asphalt pavements after palcement; thus, it was possible to simulate cold-weather paving operations involving the placement of thin mats on preheated asphalt bases. The preheat times required are shown to be a function of heater release rate and initial base temperature in addition to the variables governing the cooling of the mat. Required preheat times appear to be in consonance with the logistics of the placement of thin lifts of hot-mixed asphalt surfacing.

•BASE preheat may be defined as the application of thermal energy to the base (in-place material on which hot-mixed asphalt concrete is placed) prior to the placement and compaction of hot-mixed asphalt pavements. Probably the greatest potential use of base preheat is in the placement of hot-mixed seal coats or thin wearing surfaces on existing asphalt pavements or asphalt bases in the early spring or in the fall. Mat thicknesses of less than 2 in. are very seldom placed on untreated granular bases because of structural design considerations. On the other hand, thin lifts on asphalt bases or existing pavements are very common, and quite frequently it is advantageous from the standpoint of construction schedules to perform this work in the early spring or in the fall.

Hot-mixed asphalt must be compacted at temperatures that will permit the attainment of desired density and void content. The available time for compaction of thin lifts during cold weather is much less than the time required by the logistics of paving operations, and thus it is necessary to change the normal paving process if such work is to be performed satisfactorily.

A mathematical model for computing the temperature distribution in hot-mixed asphalt pavement after placement has been described by Corlew and Dickson (1). Computations based on this model have been used by Foster (2) in a study of cessation requirements for constructing hot-mixed asphalt pavements. According to Foster (2) "reasonable times to apply breakdown rolling" are 15 min for thicker lifts and 8 min for thinner lifts, but no specific delineation is needed because the 8-min time could be used for any thickness of lift if rollers were available. Using a minimum average mix temperature of 175 F for breakdown compaction, Foster shows that a 1¼-in. mat can be adequately compacted within 8 min if placed at a base temperature of 30 F or higher and that a 1-in. mat can be adequately compacted within the same length of time if placed at a base temperature of about 75 F. Frenzel, Dickson, and Corlew (3) describe a com-

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puter analysis of modifying base environmental conditions to permit cold-weather paving and conclude that base preheat is economically feasible from the standpoint of fuel cost and that preheating the base has a pronounced effect on the time for the mix to cool to a specified temperature.

EXPERIMENTATION

Bench scale laboratory tests were conducted in which test specimens of asphalt base, 4 in. in diameter, were preheated with a direct-fired propane heater. Some of the test specimens were laboratory-prepared, and others were field cores from asphalt base construction projects. The test specimen used in obtaining the experimental results reported in this paper consisted of a core sample of asphalt base from a project located north of Kaycee, Wyoming. Thermocouple junctions were located in the test specimen at a radius of 1 in. and at vertical distances of $\frac{1}{4}$, $\frac{1}{2}$, $\frac{3}{4}$, 1, $1\frac{1}{2}$, and 2 in. from the upper surface. After thermocouples were installed, the test specimen was insulated radially with a $1\frac{1}{2}$-in. thickness of 85 percent magnesia block insulation.

The direct-fired propane heater consisted of a partially premixed propane jet burner mounted in a refractory lined and insulated heater shell. Capacity of the heater was about 90,000 Btu/hr/sq ft of heated area.

Base temperature distributions were measured before, during, and after preheat; and continuous records of temperatures at various locations in the base were obtained by means of 2-pen strip chart recorders. Experimental preheat runs included initial base temperatures of 20, 40, and 50 F with preheat times of 5, 10, and 15 sec. Figure 1 shows the experimental time-temperature curves during and after preheat at a point $\frac{1}{4}$ in. from the upper surface of the base for preheat times of 5, 10, and 15 sec. The heater release rate was 75,000 Btu/hr/sq ft of heated area, and the calculated heat loss from the external surfaces of the heater was 9,100 Btu/hr/sq ft of heated area. A maximum temperature is reached within 1 min after the start of preheat, and longer preheat times give higher maximum temperatures. After preheat, the asphalt base test specimen cooled at room temperature.

COMPARISON OF EXPERIMENTAL AND COMPUTED RESULTS

Figure 2 shows a comparison of experimental results and computed results for a typical base preheat application. The asphalt base test specimen was cooled to a temperature of 21 F before preheating. Preheat time amounted to 10 sec with a heater release rate of 75,000 Btu/hr/sq ft of heated area and with a calculated heater loss of 9,100 Btu/hr/sq ft of heated area. Thermal efficiency (heat absorbed by the asphalt base divided by the heat released by the heater) averaged 60 percent for the test period. After preheat, the test specimen was allowed to cool at room temperature of 77 F.

Experimental and computed results are considered to be in good agreement. At a distance of $\frac{1}{4}$ in. from the upper surface, the temperature reached a maximum value of about 120 F at a total elapsed time of about $\frac{3}{4}$ min after the start of preheat. At greater distances from the surface, the magnitude of the maximum temperature reached was less and the time at which the maximum temperature occured was greater.

The mathematical model used for the computations differs from the one described by Frenzel, Dickson, and Corlew (3) and is considered to be more realistic from the standpoint of physical equipment considerations in that it is based on a constant heater release rate instead of a constant temperature energy source. Assumed values used for the thermal properties of the asphalt base were as follows: thermal conductivity, 0.7 Btu/hr/ft, F; specific heat, 0.22 Btu/lb, F; and density, 140.0 lb/ft^3.

SIMULATED COLD-WEATHER PAVING

The computer program that was developed and experimentally tested for base preheat was combined with the computer program for cooling of hot-mixed asphalt concrete, and thus it was possible to simulate cold-weather paving operations involving the placement of thin mats on preheated asphalt bases. The ultimate goal was to determine, if possible, how much preheat was required to give adequate time for compaction of thin lifts. The simulations were based on an initial mix temperature of 300 F, wind velocity of 10 knots, and solar radiation of 40 Btu/hr/sq ft. Maximum theoretical heater release rate, calculated from data presented by Spalding (4) relative to flame strength and flame speeds, amounted to about 440,000 Btu/hr/sq ft of heated area. It was felt that to attain the maximum heater release rate might be neither practical nor necessary; therefore, heater release rates of 110,000, 220,000, and 330,000 Btu/hr/sq ft of heated area were considered. The thermal energy loss from the external surfaces of the heater was assumed to be 7.5 percent of the heater release rate.

Two different modes of operation were considered: (a) the combined heater-paver operation in which there is no elapsed time and, consequently, no heat loss between base preheat and placement of the mix and (b) the separate heater-paver operation in which there is an elapsed time of 1 min during which the preheated base cools in the existing environment prior to the placement of the hot-mixed asphalt concrete.

Combined Heater-Paver Operation

Figure 3 shows the times for the average temperature of a 1-in. mat to cool to 175 F for initial base temperatures of 20, 30, 40, and 50 F and for base preheat of various times with a heater release rate of 110,000, 220,000, and 330,000 Btu/hr/sq ft of heated area and with no heat loss between base preheat and placement of the mix. A compaction time of 8 min and the information shown in Figure 3 were used to construct the graphs shown in Figure 4. For the previously mentioned heater release rates, Figure 4 shows the base preheat time required for compaction of a 1-in. mat thickness at an average temperature of at least 175 F. Thus, for an initial base temperature of 50 F, a preheat time of 2 sec would be required when a heater is used that has a capacity of 220,000 Btu/hr/sq ft of heated area; and for an initial base temperature of 40 F, a preheat time of about 5.4 sec would be required when a heater is used that has a release rate of 110,000 Btu/hr/sq ft of heated area.

Figure 4 also shows the preheat times required for compaction of a ½-in. mat when heaters are used that have capacities of 220,000 and 330,000 Btu/hr/sq ft of heated area respectively. The preheat times required for the ½-in. mat are 3 to 6 times greater than those required for the 1-in. mat. When a heater is used that has a release rate of 110,000 Btu/hr/sq ft, preheat times for a ½-in. mat are all more than 20 sec.

Figure 5 shows temperature profiles for 1 min and 8 min after placement of a 1-in. mat thickness with and without base preheat of 6 sec and with a heater release rate of 110,000 Btu/hr/sq ft on a base with an initial temperature of 40 F. The temperature of the surface of the base is substantially greater when base preheat is used; that should result in improved bonding of the mat to the base.

As mentioned previously, it is assumed that no heat loss occurs between the application of base preheat and the placement of the mix; such an operation is equivalent to one employing a combined heater-paver. Thus, if it is assumed that the preheater of the preceeding example has a length of 6 ft, the paver speed would be 60 ft/min, which according to Foster (5) is equivalent to a production rate of about 260 tons/hour of hot-mixed asphalt concrete when a 1-in. mat thickness is placed.

Separate Heater-Paver Operation

The required preheat times for separate heater-paver operations are greater than those for combined heater-paver operations because of the thermal energy loss from the preheated base during the elapsed time between base preheat and placement of the mix. Computations for the separate heater-paver operation are based on an elapsed time of 1 min during which thermal energy is transferred from the upper surface of the preheated base to the atmosphere by means of radiation and convection and downward into the base by means of conduction.

The base preheat times required for the compaction of a 1-in. mat thickness at an average mat temperature of at least 175 F for the separate heater-paver operation are shown in Figure 6. As in the case of the combined heater-paver, compaction is assumed to be accomplished within 8 min after placement of the mix. Figure 6 shows that a preheat time of about 2.9 sec is required for an initial base temperature of 50 F

Figure 1. Temperature at a point ¼ in. from upper surface of base during and after preheat.

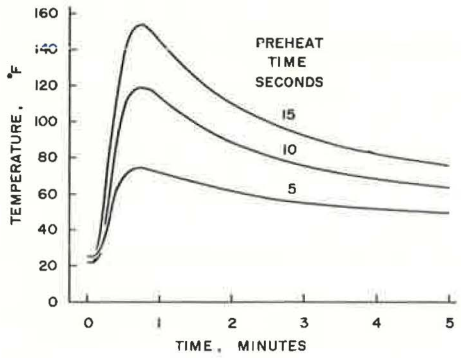


Figure 2. Experimental and computed temperatures at ¼, ½, ¾, 1, 1½, and 2 in. from upper surface of base.

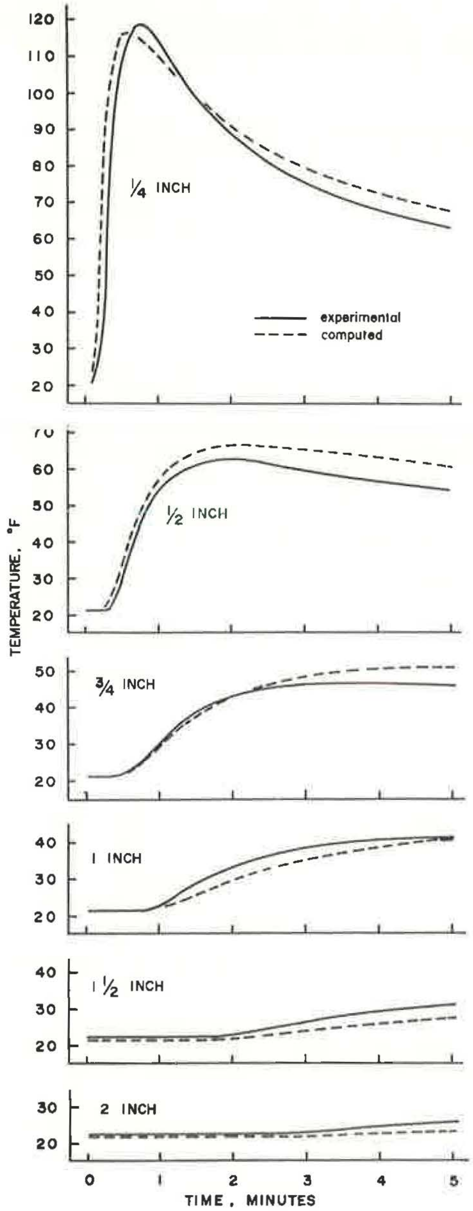


Figure 3. Cooling times for 1-in. mat in combined heater-paver operation.

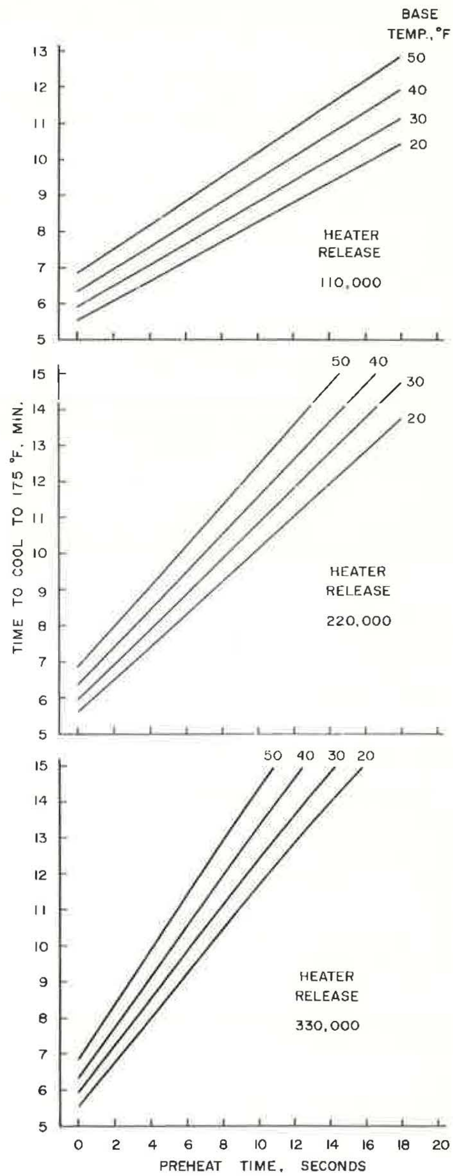


Figure 4. Required preheat times for 1- and ½-in. mats in combined heater-paver operation.

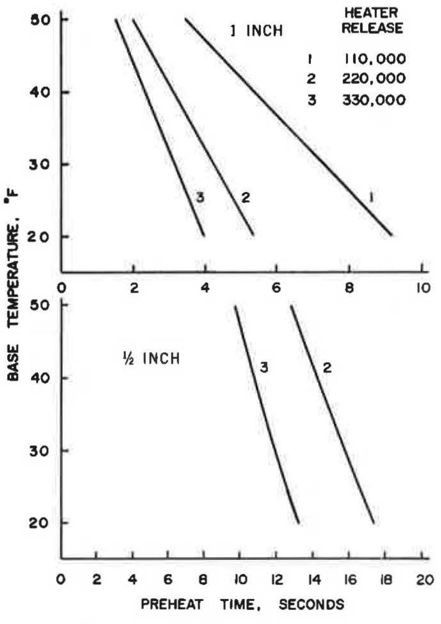


Figure 5. Mix temperature profiles for 1-in. mat in combined heater-paver operation.

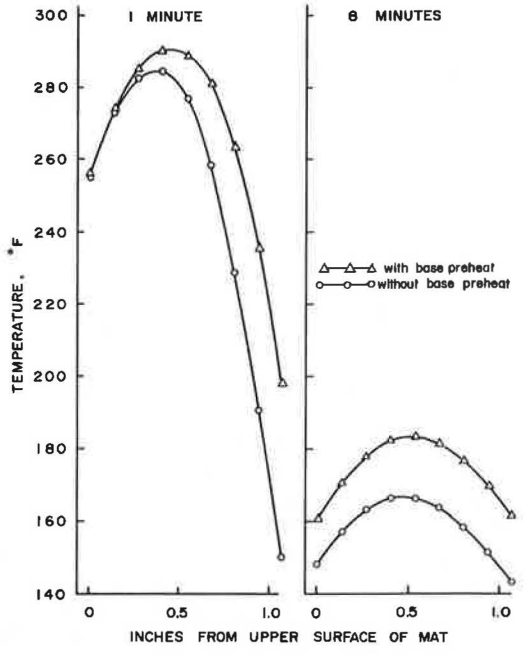
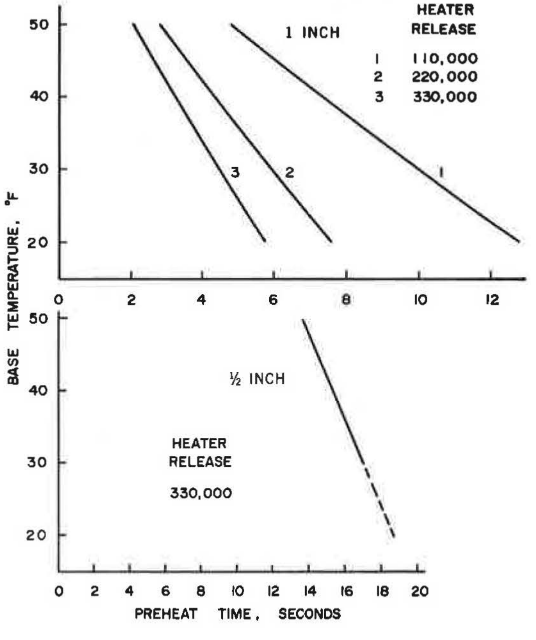


Figure 6. Required preheat times for 1- and ½-in. mats in separate heater-paver operation.



when a heater is used that has a release rate of 220,000 Btu/hr/sq ft of heated area. The preheat time for the separate heater-paver is about 45 percent greater than that required for the combined heater-paver for the foregoing conditions. For an initial base temperature of 40 F and a heater release rate of 110,000 Btu/hr/sq ft, a preheat time of about 7.3 sec is required; that time is approximately 38 percent greater than the time for the combined heater-paver operation.

Required preheat times for a ½-in. mat thickness (compacted within 8 min at an average temperature of at least 175 F) are also shown in Figure 6. For heater release rates of 110,000 and 220,000 Btu/hr/sq ft, the required preheat times are more than 18 sec. For a heater release rate of 330,000 Btu/hr/sq ft, the required preheat times are about 40 percent greater for the separate heater-paver operation than for the comparable combined heater-paver operation.

SUMMARY AND CONCLUSIONS

A mathematical model was developed for simulating the radiative and convective transfer of thermal energy from a direct-fired propane heater to the upper surface of the base. The validity of the model was confirmed with bench scale laboratory tests in which asphalt-base test specimens were preheated with an insulated and refractory lined direct-fired propane heater. Experimentation included nominal initial base temperatures of 20, 40, and 50 F.

The computer program for base preheat combined with the computer program for cooling of hot-mixed asphalt concrete enables the simulation of cold-weather paving operations involving the placement of thin mats on a preheated base. The results indicate that base preheat is effective in increasing the allowable time for compaction and in maintaining the temperature of the surface of the base at increased levels prior to and at the time of compaction. Improved bonding between the mat and the base as well as improved compaction should be the result.

The preheat time required is shown to be a function of the heater release rate and the initial temperature of the base in addition to the variables governing the cooling of the mat. Required preheat times appear to be in consonance with the logistics of the placement of thin lifts; that is, for a 1-in. mat thickness placed on a 40 F base, it appears that paver speeds of 60 ft/min are realistic corresponding to an asphalt hot-mixed production rate of 260 tons/hour. Longer preheat times required for mats thinner than 1 in. would naturally decrease production rates with the same physical equipment.

ACKNOWLEDGMENT

The authors wish to express their sincere thanks for the support of the National Science Foundation whose project made this research possible.

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STANHOPE STUDY OF COMPACTION METHODS FOR BITUMINOUS STABILIZED BASE

R. R. Santoro, K. C. Afferton, and J. A. Walz,
New Jersey Department of Transportation

In September 1970 the Equipment Committee of the New Jersey Department of Transportation conducted an evaluation test of compaction equipment for bituminous stabilized base course. The objective of the test was to evaluate the compaction capabilities of 2 vibratory rollers and a tandem roller and to compare them with the capabilities of the department's standard compaction system (3-wheeled roller with tandem finish). Comparisons were made by using both multiple- and thick-lift paving methods. The findings indicated that all rollers evaluated were capable of achieving acceptable densification levels in the stone mix, bituminous stabilized base course used in the test construction. In multiple-lift construction the vibratory compactors were found to attain essentially the same base density as that produced by the department's standard system. However, the vibratory units required approximately 30 percent more compaction time. In thick-lift construction the department's system was again found to be the optimum of the roller systems considered. The vibratory rollers were not observed, within the range of applications evaluated, to cause decompaction or density drop off of the base material. Pavement riding quality was not adversely affected by either one of the vibratory compactors studied.

•ON SEPTEMBER 30, 1970, the Equipment Committee of the New Jersey Department of Transportation conducted its third major evaluation test of compaction equipment for bituminous concrete. A test section consisting of a 4-in. thick, plant-mixed bituminous stabilized base course (stone mix) was constructed at Stanhope, New Jersey, on the southbound lanes of the NJ-206 connector for Interstate 80, section 1M. The basic objective of the test was to compare the breakdown compaction capabilities of 2 vibratory rollers and a tandem roller with those of a standard 3-wheeled roller. The capabilities to be studied encompassed the important factors of densification, compaction efficiency, and pavement smoothness.

Current specifications of the department require that all breakdown compaction of bituminous paving materials be accomplished with a 3-wheeled roller having a total weight of not less than 10 tons and having not less than 330 lb/in. of width on the rear wheels. A minimum of one breakdown pass with a 3-wheeled roller is specified.

The vibratory roller has been successfully used in bituminous pavement construction in Europe for several years. However, in the United States and particularly in New Jersey, the extension of vibratory compaction from soil aggregates to bituminous paving materials is still in its infancy. The New Jersey Department of Transportation first used a vibratory compactor on bituminous concrete experimentally in 1967 on a small portion of Interstate 80, section 3K. Unfortunately, the experiment proved inconclusive because of the extremely variable and uncontrollable operational characteristics of the roller. Two years later, the department also participated in the monitoring of an impressive demonstration of a dual-drum vibratory roller on bituminous construction for the New Jersey Turnpike. The decision to conduct the vibratory roller tests at Stanhope resulted primarily from the successful nature of the turnpike demonstration.

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The 2 vibratory rollers used in this evaluation were model CA-25A supplied by Vibro-Plus Products, Inc., and Rustler 404 supplied by RayGo, Inc. Both units were self-propelled, 2-axle, single vibratory drum compactors with rubber tires on the drive axle. Both rollers had the ability to change dynamic compactive force by varying their frequency of vibration. The Vibro-Plus unit also had the capability of operating at 2 different amplitude levels; only the high amplitude mode of operation was employed in the test work.

The inclusion of tandem breakdown rolling in the Stanhope test was prompted by findings in the committee's study of bituminous pavement riding quality. Investigations suggested that, through the use of tandem rather than 3-wheeled rollers for initial mat compaction, several states may be achieving markedly better riding pavements than those achieved in New Jersey. It was expected that, under the controlled conditions of a test section, the beneficial effects, if any, of tandem breakdown rolling on pavement smoothness could be quantified.

The planning, construction, control testing, and data evaluation for the Stanhope test section were shared by the various member divisions of the Equipment Committee. Guidance in the use of the vibratory rollers was provided by representatives of the 2 suppliers.

METHOD OF STUDY

The Stanhope test section was divided into 8 subsections. In subsections 1 through 4, the compactors were evaluated in conjunction with the multiple-lift mode of stabilized base construction (4-in. base constructed in two 2-in. thick-lifts). The same rollers were then used with single, or so-called, thick-lift construction in subsections 5 through 8 (4-in. base constructed in one 4-in. thick lift). Current department specifications require that the multiple-lift method be used in all bituminous base paving. However, recent successful trials of single, thick-lift paving suggest that this may soon be an acceptable alternate on department projects.

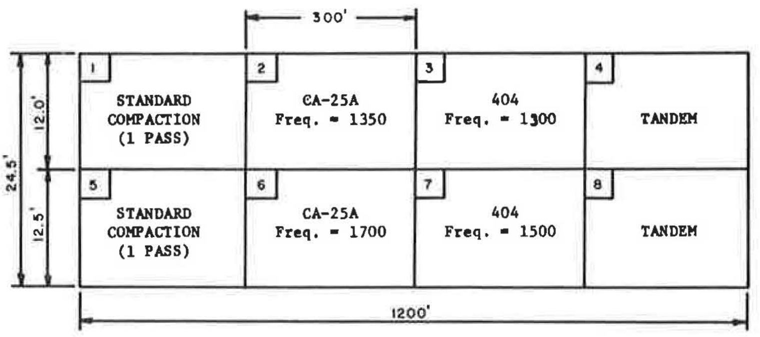
The general layout developed for the test area and the construction requirements for each subsection are shown in Figure 1. A complete description of the 4 compactors used in the study is given in Table 1.

In each subsection, the prescribed compaction sequence produced strips or zones having different numbers of roller coverages. It was expected that the density growth characteristics of each compactor could best be determined by evaluation of the coverage zones. As used in this paper, 1 coverage is defined as 1 pass over a point on the base of 1 rear wheel of the 3-wheeled roller, the rear drum of the tandem, or the vibrating drum of the vibratory compactors.

In subsections 1 and 5, where the department's standard method of stablized base compaction, used was the 3-wheeled roller applied the specification minimum of 1 breakdown pass. A pass in this instance means that the roller progressed from edge to edge uniformly lapping (one-half width of rear wheel) each preceding truck until the entire mat was rolled by the rear wheels. The roller overlaps associated with this pass produced 2 and 3 coverage zones in subsection 1. The same rolling procedure resulted in 2 and 4 coverage zones in subsection 5. The development of density growth data for the vibratory rollers was facilitated by providing 2, 3, and 5 coverage zones in the vibratory subsections. The manufacturers of the vibratory equipment estimated that 2 to 3 coverages at the frequencies they recommended would provide sufficient densification in subsections 2, 3, 6, and 7 (Fig. 1).

In subsections 4 and 8, which were to receive tandem breakdown rolling, only a tentative compaction sequence was established to produce 3, 4, and 6 coverage zones. The lack of experience with tandem rollers used in the breakdown position required that the Equipment Committee give construction control personnel the option of increasing roller coverages if the planned compaction proved inadequate. Nuclear density measurements (2 locations) were taken in the tandem breakdown subsections immediately after completion of compaction to determine the level of densification achieved. Additional coverages of either the tandem or 3-wheeled roller were to be applied if the nuclear measurements suggested air voids levels above that permitted in the department's standard specifications.

Figure 1. Stanhope test section.



STANDARD CONSTRUCTION
(PLACE 2 LIFTS, 2" THICK)

SUBSECTION NO. 1

1. Breakdown Rolling - 3 wheel
2. Finish Rolling - Tandem (passes as necessary)
3. Evaluate 2 & 3 coverage zones

SUBSECTION NO. 2

1. Breakdown Rolling - Vib. CA-25A
2. Frequency = 1350±50 V.P.M.
3. Finish Rolling if Needed - Tandem (passes as necessary)
4. Evaluate 2,3&5 coverage zones

SUBSECTION NO. 3

1. Breakdown Rolling - Vib. 404
2. Frequency = 1300±50 V.P.M.
3. Finish Rolling if Needed - Tandem (passes as necessary)
4. Evaluate 2,3&5 coverage zones

SUBSECTION NO. 4

1. Breakdown and Finish Rolling if Needed - Tandem
2. Evaluate - 3,4&6 coverage zones (If nuclear densities are inadequate, additional passes will be made and recorded.)

THICK LIFT CONSTRUCTION
(PLACE 1 LIFT, 4" THICK)

SUBSECTION NO. 5

1. Breakdown Rolling - 3 wheel
2. Finish Rolling - Tandem (passes as necessary)
3. Evaluate 2 & 4 coverage zones

SUBSECTION NO. 6

1. Breakdown Rolling - Vib. CA-25A
2. Frequency = 1700±50 V.P.M.
3. Finish Rolling if Needed - Tandem (passes as necessary)
4. Evaluate 2,3&5 coverage zones

SUBSECTION NO. 7

1. Breakdown Rolling - Vib. 404
2. Frequency = 1500±50.V.P.M.
3. Finish Rolling if Needed - Tandem (passes as necessary)
4. Evaluate 2,3&5 coverage zones

SUBSECTION NO. 8

1. Breakdown and Finish Rolling if Needed - Tandem
2. Evaluate - 3,4&6 coverage zones (If nuclear densities are inadequate, additional passes will be made and recorded.)

Table 1. Compaction equipment.

Equipment	Size	Equipment	Size
3-wheeled roller		RayGo	
Weight, tons	10 to 12	Shipping weight, lb	18,500
Rolling width, in.	84	Drum diameter, in.	59
Width of rear wheels, in.	24	Drum length, in.	84
		Variable frequency, vpm	1,150 to 1,500
Vibro-Plus		Reed vibration tachometer	
Overall net weight, lb	20,300	Static drum force[a], lb	12,000
Drum diameter, in.	60	Dynamic force[a], lb	27,000
Drum length, in.	84		
Variable frequency, vpm	To 2,400	Tandem roller	
Static drum force, lb	10,500	Weight, tons	10 to 12
Centrifugal force (high amplitude setting)[a], lb at 1,700 vpm	18,500	Width of rear roll, in.	54

[a]From equipment brochures.

Table 2. Mix design.

Property	Quantity	Property	Quantity
Sieve, percent passing		Sieve, percent passing	
2 in.	100	No. 200	6.1
1½ in.	100	Asphalt cement, percent	4.3
¾ in.	79	Air voids, percent	3.98 (6.1)[a]
No. 4	48	Average stability, lb	2,650 (2,050)[a]
No. 8	38	Average flow, in.	0.11 (0.11)[a]
No. 50	15	Weight, lb/ft³	150 (149)[a]

[a]Average of 2 sets (6 plugs) of Marshall specimens molded at plant on day of test section construction. The maximum specific gravity of Marshall specimens was determined by the New Jersey Department of Transportation's solvent immersion test method.

Tandem-finish rolling was used on any subsection where the mat surface was irregular after breakdown rolling was completed. It was expected that tandem-finish rolling would not be necessary in the tandem breakdown subsections and, also, possibly not needed in the vibratory subsections.

The bituminous stabilized base used in construction of the test section was in accordance with the design and control requirements of mix 1 of the 1968 Addenda A Revisions to the department's standard specifications. The specific design characteristics of this material are given in Table 2. The entire test section was constructed over a 6-in. layer of dry-bound macadam base underlaid by 14 in. of granular subbase. Department personnel monitored the material production at the asphalt plant and the overall construction of the test area.

PLANT INSPECTION

The major objective of the plant inspection was to control the uniformity of material being supplied to the test pavement. This was necessary because a significant variability in material would prevent the making of valid statistical comparisons both within and between subsections.

The adequacy of the composition uniformity was determined by the analysis of 6 random samples of the plant's production. Extraction results indicated that the base material was well controlled and in good conformity with the job mix formula. Control of mixing temperature was also quite adequate for mixture temperatures ranging from 280 to 300 F.

Table 2 gives the average Marshall results for 2 sets (6 plugs) of specimens molded at the plant on the day of the test pavement construction. The Marshall test data are given for comparison with the job mix design values.

CONSTRUCTION OBSERVATIONS

The bituminous stabilized base test pavement was 1,200 ft long and 24.5 ft wide. Each of the 8 subsections was 300 ft long and approximately 12 ft wide. A 100-ft dead-zone area was provided at the interface of each subsection to facilitate construction equipment movements. The dead-zone areas were not included in the roller evaluation.

Paving operations began by placing the bottom lift in subsections 1 through 4 and continued by placing the single thick lift in subsections 5 through 8. Paving of the test section was then completed by placing the top lift in subsections 1 through 4. The breakdown compaction of each subsection was not started until the paver had completed laydown in that subsection.

Each compactor began breakdown rolling at the low edge of the uncompacted mat. Lateral displacement at the edge of the mat was not considered excessive during compaction of either the multiple-or the thick-lift sections. No initial or final static passes were applied by either one of the vibratory compactors.

All maneuvering (lateral shifts) by vibratory rollers required to complete their breakdown compaction sequence was performed on previously compacted material (100-ft dead-zone areas, static drum). This procedure was recommended by representatives of the vibratory roller equipment to avert any possibility of marring or rupturing the uncompacted mat. Both the tandem and the 3-wheeled rollers were capable of performing the maneuvering necessary to complete their breakdown compaction sequences on either the compacted or uncompacted mat without detrimental effects.

Slight ridges or depressions; which were similar in nature to those made by the 3-wheeled roller, were observed in the mat after the first passage of the vibratory and tandem rollers. However, these ridges or depressions were sufficiently eliminated during the remainder of the breakdown compaction sequence. Tandem-finish rolling was therefore not used on any of the subsections where vibratory or tandem breakdown compaction was performed. Finish rolling (2 coverages) with the tandem unit was applied to the 3-wheeled roller subsections.

The rubber tires of the vibratory rollers were not preheated, although both units utilized an additive to prevent tire pickup (buildup of fines from the mix). No significant tire pickup was noted on this particular mix by either of the vibratory rollers tested.

The RayGo compactor was observed to bounce off the mat severely for a short time during the compaction of subsection 3 (top lift); the vibrating drum was then brought back under control by the operator (manufacturer's representative). It appeared that this was accomplished by increasing the roller speed.

Generally, the construction of the test section was in conformance with the planned procedures. Additional compactive effort was applied to subsection 4, second lift (1 pass with 3-wheeled roller), and subsection 8, thick lift (2 additional passes with the tandem roller), as a result of nuclear density measurements taken in the 3 coverage zones at the completion of the prescribed rolling. The air voids level suggested by the average of 2 nuclear density measurements in subsection 4 were sufficiently high to indicate that the 3-wheeled rather than the tandem roller be used to achieve the necessary densification.

In addition to the overall supervision of the test project, several specific phases of the test construction were monitored and recorded by Department personnel and include the following:

1. Paver and roller times for each subsection;
2. Periodic checks of frequency of vibration with a reed type of hand vibrometer to establish vibratory roller compliance with recommended frequency levels;
3. Setting of pavers vibrating screed-different intensity settings for each mode of construction (multiple and thick lift);
4. Temperature measurements recorded by thermocouples installed either underneath or approximately at the mid-depth of mat (dead-zone areas) and by probe thermometers; and
5. Documentation of air temperature during the day of the test (temperatures ranged from a low of 42 F in the morning to a high of 58 F in the afternoon).

PAVEMENT TESTS

Pavement tests consisted primarily of random nuclear densities taken during and after construction, the measurement of density of 4-in. cores cut from the pavement, and the measurement of pavement riding quality.

Final test section densification was initially to be evaluated on the basis of cores, which is the department's normal method of determining pavement density. However, it was subsequently considered impractical to cut the number of cores required to amass significant data. It was, therefore, decided to obtain the majority of the density observations by means of nuclear density devices. Nuclear density measurements were to be utilized in predicting core density values through correlation equations. The nuclear devices were also to be employed in determining paver laydown densities in all subsections and density buildup during compaction in the vibratory and tandem subsections. Density growth data obtained in this latter fashion were to supplement the primary density growth information (final coverage zone densities).

The density data required for analysis of the test section (total of 22 coverage zones) were obtained at 154 random locations (7 per each coverage zone). A nuclear density gauge was used to obtain paver laydown densities at 2 of these locations in each subsection. In the vibratory and tandem subsections, the nuclear device was further used for density measurements between roller coverages (2 locations monitored per subsection). Determinations of final density with the nuclear gauge were made at all 154 random locations. For use in the development of a predictor equation for core density, cores were cut at 2 of the 7 locations in each coverage zone resulting in a total of 44 cores. A typical density measurement pattern for a subsection is shown in Figure 2.

Because this was the first instance in which the department was to place primary reliance on nuclear devices to obtain pavement density measurements, there was strong concern as to the particular method to follow in using a nuclear gauge. It was not initially evident which of the currently used methods would provide the best marriage between core density correlation and simplicity of use. For this reason, a nuclear density measurement was repeated 3 times wherever possible, and a different method was used each time. A measurement was made with the gauge in the backscatter position without surface preparation, then with surface preparation (standard 20-30 Ottawa sand), and finally with the air-gap method (including surface preparation).

The 44 cores taken from the test area were analyzed in the department's central laboratory to determine bulk and maximum specific gravities. Bulk specific gravities were obtained by AASHO Method T166; maximum specific gravities were determined by the department's solvent immersion test method.

A stated previously, one of the important aims of the study was to evaluate the riding quality or pavement smoothness produced by each of the compactors tested. That was accomplished by measuring the smoothness of each subsection with 2 devices: a 10-ft rolling straightedge and a BPR roughometer. The rolling straightedge indicates the span length and magnitude of surface deviations in the range of $\frac{1}{8}$ to $\frac{1}{2}$ in. in $\frac{1}{8}$-in. increments. The BPR roughometer, consisting of a fifth wheel towed over the pavement surface at 20 mph, yields an aoutput referred to as the roughness index (RI). The RI is equivalent to the accumulated deviations in the pavement surface, in in./mile. A high RI is thus indicative of a rough pavement surface.

Measurements were obtained by the 2 devices in both wheelpaths of each 300-ft subsection (dead zones were excluded). Because of the short lengths measured (200 ft), the roughometer made 3 repeat runs in each wheelpath in an attempt to obtain the best estimate of the pavement smoothness or RI.

DISCUSSION OF FINDINGS

Temperature Measurements

The procedure established for the monitoring of pavement temperatures required the installation of 1 thermocouple in each lift of each subsection. Temperature measurements were recorded by a potentiometer attached to the thermocouples.

It was not originally planned to take probe thermometer measurements, except from trucks. However, during construction, several problems developed with the thermocouple equipment and the installation procedures employed. It was, therefore, necessary to take probe measurements, although it was not possible to fully supplement the voluminous number of temperature observations planned for the thermocouples.

Based on the combined data from the thermocouples and probe thermometers, the laydown temperature for all subsections ranged from 270 to 280 F. For the thick-lift constructed subsections, all planned breakdown compaction was accomplished within the approximate temperature range of 245 (start) to 215 F (finish). It is estimated that the 2 additional passes found necessary in tandem subsection 8 were completed above 180 F.

The thermocouple and probe thermometer measurements in the multiple-lift subsections indicated a breakdown temperature range from 240 (start) to 190 F (finish). By extrapolating some of the temperature data, we estimate that the added pass of the 3-wheeled roller in subsection 4 was accomplished between 145 (start) and 125 F (finish).

Density Determination

A basic assumption in the Stanhope test was that roller compaction capabilities could be evaluated by comparison of density levels achieved both within and between subsections. This assumption is essentially valid if subgrade support conditions and bituminous base composition were uniform throughout the test area. Observations prior to construction indicated that the macadam base had been adequately densified to afford a consistent, stable subgrade for the bituminous base. Also, statistical analysis of laboratory extraction test results revealed that sufficient uniformity of mixture composition was maintained during the test pavement construction.

As previously stated, both nuclear and core density determinations were utilized in this study, and the nuclear densities were the primary measure of pavement densification. To analyze the density data in one standard form required that the nuclear density measurements be subsequently converted to predicted core density values by use of linear correlation equations. Core densities were predicted by using only a few of the many correlation relations established from the density measurements. Most

Figure 2. Typical density measurement pattern for subsection.

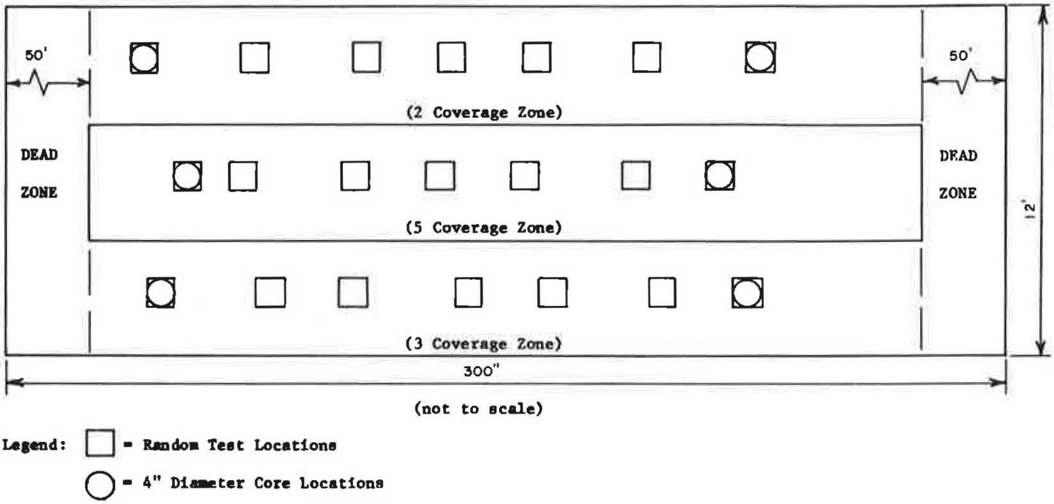


Table 3. Mean densities.

Compactor	Multiple-Lift Construction Sub-section	Coverages	Mean Density[a] (lb/ft^3)	Single-Lift Construction Sub-section	Coverages	Mean Density[a] (lb/ft^3)
3-wheeled roller	1	2	146.1	5	2	147.4
		3	146.6		4	147.4
Vibro-Plus	2	2	143.5	6	2	143.7
		3	144.8		3	145.0
		5	146.8		5	146.8
RayGo	3	2	143.3	7	2	144.9
		3	144.7		3	146.1
		5	146.1		5	146.3
Tandem roller	4	3	144.8	8	5	146.2
		4	144.1		7	147.4
		6	145.3		10	148.0

[a]Average of 7 measurements: two 4-in. diameter cores and five predicted core density values based on nuclear density measurements (air-gap method).

Table 4. Mean densities as a percentage of Marshall density.

Compactor	Multiple-Lift Construction Sub-section	Coverages	Percent of Marshall Density[a]	Single-Lift Construction Sub-section	Coverages	Percent of Marshall Density[a]
3-wheeled roller	1	2	98.1	5	2	99.0
		3	98.5		4	99.0
Vibro-Plus	2	2	96.4	6	2	96.5
		3	97.2		3	97.4
		5	98.6		5	98.6
RayGo	3	2	96.2	7	2	97.3
		3	97.2		3	98.1
		5	98.1		5	98.3
Tandem roller	4	3	97.2	8	5	98.2
		4	96.8		7	99.0
		6	97.6		10	99.4

[a]Marshall density = 100 x (mean density/Marshall density). Mean density is average of 7 measurements: two 4-in. diameter cores and five predicted core density values based on nuclear density measurements (air-gap method). Marshall density is average of 2 sets of Marshall specimens (6 plugs molded at the plant on the day the test pavement was constructed).

of the density conversions were actually accomplished with the following equation, developed from paired core and air-gap measurements and found to yield the most accurate predictions:

$$Y = 82.6 + 0.467\ X$$

where

Y = predicted core density value, lb/ft^3; and
X = nuclear density measurement, air-gap method, lb/ft^3.

The correlation coefficient is 0.75, and the standard error of estimate is 1.27 lb/ft^3.

From the actual and predicted core densities (154 random locations), mean densities were determined for each of the 22 coverage zones in the 8 subsections of the test pavement. A summary of these mean density values is given in Table 3. Comparing the average densities of the different coverage zones within each subsection makes it possible to establish density growth patterns for the rollers under study. The density growth measurements collected during the actual compaction operations were in good agreement with the data given in Table 3.

An important observation to be made from the density growth information is that the vibratory rollers, within the range of coverages considered (2 to 5), continued to increase density with each added coverage. The higher number of coverages did not cause loosening of the material or the density reduction that often occurs with increased vibratory roller applications on cohesionless soil materials.

The density data for the various rollers cannot be evaluated further without first considering the level of densification actually needed in a stabilized base course. After a good deal of investigation, the department formulated in 1968 a pavement specification that essentially requires a contractor to achieve an average air voids level of not more than 6 percent in a bituminous base. Unfortunately, this requirement has little meaning for most highway engineers for other states normally evaluate compaction in terms of relative density rather than air voids. On a relative-density basis, the 6 percent air voids limit corresponds to approximately 98 percent of the Marshall density when the department's design criteria are taken into account. This means that on most stabilized bases a roller or roller system must be capable (on the average) of achieving at least 98 percent of the laboratory Marshall density to satisfy voids requirements.

So that a comparison could be made, the test pavement density data given in Table 3 were refashioned in terms of percentage of Marshall density. The resulting values, given in Table 4, can be correctly evaluated by distinguishing between real or significant differences and those resulting simply from normal variation in measurements. For this reason, a statistical, 1-tail t-test was used in the study to analyze density differences within and between subsections.

The information given in Table 4 reveals that the department's standard compaction system is equal to or better than the critical 98 percent Marshall density level in both multiple- and thick-lift construction. Furthermore, statistical analysis indicates that the additional 3-wheeled roller breakdown coverages investigated in the study did not effect any significant density increase in either paving mode.

The vibratory rollers behaved much differently from the standard compaction system. In both its multiple- and single-lift subsections the Vibro-Plus roller significantly increased mat densification with increased coverages. The same situation occurred with the RayGo vibratory roller. However, in deep-lift construction (subsection 7), the RayGo compactor was unable to cause a significant density increase with its final 2 applications. In this particular instance, however, the RayGo unit did surpass the important 98 percent Marshall density level with only 3 roller coverages; in all other vibratory subsections (multiple- and single-lift construction), 5 coverages were needed.

Before the performance of the tandem roller is considered, it is necessary to comment on the decision made during construction that modified the tandem roller's planned compaction sequence. The special monitoring used with the tandem roller resulted in additional compactive effort being applied to its subsections. In retrospect, it is questionable whether the added compaction was completely justified inasmuch as the decision was made on the basis of air voids, and not percentage of Marshall density. After the test section was constructed, it was discovered that all subsections generally had a higher than expected air voids level. Further investigation of the plant's mix design uncovered an error that had caused the base mixture to exhibit slightly less than normally desired compaction characteristics (somewhat elevated Marshall air voids). Based on the implications of this finding, the added compaction in the tandem subsections was applied more as a result of a mix design deficiency than of a poor compactive effort.

The additional compactive effort used in tandem roller subsection 4 was 1 pass of a 3-wheeled roller on its top lift. Unfortunately, this change in roller type makes it impossible to establish the tandem roller's compaction capabilities in multiple-lift construction from the final density measurements. In thick-lift construction, the extra compactive effort was applied with 2 additional passes of the tandem unit; the added applications are reflected in the data given in Table 4. Analysis of final densities in this instance is, therefore, valid.

In both tandem subsections the added compaction was accomplished at relatively low temperatures (180 F for thick, and 140 F for multiple). There is, therefore, strong doubt especially in multiple-lift construction that the supplemental roller passes increased densification. This doubt is supported by the fact that measurements before and after the extra rolling at the monitoring locations failed to detect a density increase.

A review of the mean densities for subsection 8 reveals that the tandem roller reached the 98 percent Marshall density level with 5 coverages. Also, at 7 coverages the tandem roller reached what might be considered its optimum densification level for the test—99 percent of the Marshall value. The statistical test indicated that no significant increase in density occurred between 7 and 10 tandem coverages.

Comparing the performance of the various rollers on multiple-lift construction, it appears that the department's standard compaction system was the optimum densification system tested. The 98 percent Marshall density level was attained with only two 3-wheeled breakdown coverages (plus 2 tandem finish passes); the vibratory rollers required 5 coverages to achieve essentially the same density condition. In thick-lift construction the standard compaction system again seems to have been the optimum method of compaction. It produced higher densities, with less breakdown coverages, than those achieved by any one of the other rollers. However, in thick-lift paving the RayGo compactor did reach the 98 percent Marshall level after only 3 applications.

A comparison of the density levels attained in the thick-lift and multiple-lift construction reveals additional performance differences. As the Equipment Committee found in past studies, the single-lift paving method generally resulted in higher degrees of bituminous base densification. Analysis of the individual density measurements further disclosed that, in terms of variance, σ^2, the thick-lift base had less than half the longitudinal variability of the multiple-lift base. It is believed that the higher mat temperatures intrinsic to thick-lift construction provided for the improved pavement densification.

Roller Efficiency

Although it is valuable to compare the various rollers in terms of density levels achieved, an equally important factor is the compaction time employed. To account for both the densification and time characteristics, we used a parameter termed "roller efficiency." Roller efficiency was taken to be the change in density effected by a roller divided by its expended compaction time. Table 5 gives the roller efficiency value for each compactor and coverage level evaluated.

The efficiency values have been calculated by assuming that the pertinent roller coverages were placed over the entire width of a 12-ft wide mat. The compaction times used in these calculations were all actual, as-measured times for the 3-wheeled and tandem rollers. For the vibratory compactors, the compaction times were esti-

Table 5. Rolling efficiency.

Compactor	Multiple-Lift Construction: Sub-sec-tion	Cover-ages	Com-paction Time (min)	Density vs. Com-paction Time[a] (lb/ft³/min)	Thick vs. Multi-ple Lift[b] (per-cent)	Thick-Lift Construction: Sub-sec-tion	Cover-ages	Com-paction Time (min)	Density vs. Com-paction Time[a] (lb/ft³/min)	Thick vs. Multi-ple Lift[b] (per-cent)
3-wheeled roller	1	2	46[c]	0.398	—	5	2	22[c]	0.850	214
Vibro-Plus	2	5	60	0.317	—	6	5	30	0.603	190
RayGo	3	5	60	0.305	—	7	3	12	1.450	309
							5	20	0.880	289
Tandem roller						8	5	26	0.673	—

[a]Final average laydown density/total compaction time, where average laydown density = 128.0 lb/ft³.
[b](Thick lift/multiple lift)/100.
[c]Includes tandem-finish rolling and all maneuvering time.

Table 6. Riding-quality measurements.

Compactor	Multiple-Lift Construction: Sub-Sec-tion	Rough-ometer[a]	Straight-edge[b]	Thick-Lift Construction: Sub-Sec-tion	Rough-ometer[a]	Straight-edge[b]
3-wheeled roller	1	141	4	5	168	14
Vibro-Plus	2	146	4	6	179	12
RayGo	3	140	2	7	143	9
Tandem roller	4	171	5	8	191	12

[a]Average equivalent roughness index, in./mile. Values and average of 6 roughometer measurements per subsection, 3 repeat measurements in each of 2 wheelpaths. Based on variability of repeat measurements, the 95 percent confidence limits, or tolerance, for each average value is +25 in./mile.
[b]Number of deviations in wheelpaths of individual subsections.

Table 7. Centrifugal forces of Vibro-Plus compactor.

vpm	Dynapac Compaction Effort During the Test—High Amplitude	Dynapac Compaction Effort Since the Test: High Amplitude	Low Amplitude
2,400	Not available for test[a]	36,000	18,000
1,700	18,060	18,060	9,030
1,350	11,390	Not rec-ommended	Not rec-ommended

[a]The CA-25 S/N 251A was tested at 1,700 vpm because a 2,400-vpm motor was not available in time. The machine has a considerably higher compaction effort and effect at 2,400 vpm than at 1,700 vpm.

mated by using the recorded, average time per coverage multiplied by the number of coverages being considered. In all instances the average densities achieved in the subsections were used to determine related density changes.

To evaluate the roller efficiency data, one must keep in mind that a bituminous base must normally be densified to 98 percent of the Marshall density (average level) to ensure compliance with department air voids criterion. The efficiency that rollers exhibit in reaching this density level is therefore of major importance in considering their use on department porjects. A review of data given in Table 5 shows that in multiple-lift construction the department's specified roller system was more efficient than the vibratory compactors in reaching the critical 98 percent Marshall density plateau. This system had the highest efficiency value and, accordingly, had the lowest compaction time of all the compaction units evaluated.

It is more difficult to compare roller efficiencies on the thick-lift and the multiple-lift paving. This is due to the fact that the standard 3-wheeled, tandem system produced 99 percent level, the 3-wheeled, tandem system would definitely be the most efficient. This system required less time to reach 99 percent density than the other rollers did to achieve lower densities. The only comment that can be offered concerning comparisons at the 98 percent Marshall density level is that the RayGo compactor was more efficient than both the Vibro-Plus and the tandem compactors.

As would be expected, the data given in Table 5 show that all rollers increased their efficiency and reduced their compaction times significantly in going from multiple-lift to thick-lift construction.

Riding Quality

The riding-quality measurements obtained in the Stanhope evaluation test are given in Table 6. The data were obtained on relatively short (200 ft) pavement lengths. A great deal of judgment would have to be exercised in extrapolating the riding qualities achieved in the short subsections of this test to those achievable over the entire length of a full-sized construction project.

The BPR roughometer data are given in the form of average roughness index values for each subsection. These values, which are each an average of 6 measurements, range from 140 to 191 in./per mile. On a typical, full sized, paving project in New Jersey, the top surface of a completed stabilized base would be expected to have a roughness index somewhere between 120 and 180 in./mile.

On the multiple-lift subsections, the 2 vibratory rollers and the department's standard compaction system produced about the same level of pavement smoothness. In contrast, tandem breakdown rolling resulted in a surprisingly rougher base surface. These same findings also apply to the thick-lift construction with the exception of the RayGo vibratory roller. In thick-lift compaction the RayGo roller seems to have produced a smoother riding surface (lower roughness index) than that attained by any one of the other compactors.

A comparison of roughometer data for the thick-lift subsections and for the multiple-lift subsections reveals an additional interesting factor. The thick-lift subsections are all, to varying degrees, rougher than corresponding multiple-lift subsections. Apparently, when a manually controlled paver is used, it was not possible to overcome as much subgrade roughness with 1 lift of base as with 2 lifts.

The riding quality measurements made with the rolling straightedge are also given in Table 6. For each subsection, the number of surface deviations in the wheelpaths is more than ⅛ in. Good riding pavements normally have few deviations, and rough pavements have many deviations. It is apparent, therefore, that the straightedge observations substantiate, in a general way, the findings from the roughometer measurements. This is particularly true in regard to the indicated advantage in riding quality of the multiple-lift over the thick-lift mode of construction.

The preceding comments must be conditioned on 2 additional considerations. First, the repeat readings with the roughometer on any one subsection were more variable than had been anticipated. Although 6 measurements were averaged in each subsection, their variability was such as to cause the resulting average to be of rather low precision. The 95 percent confidence limits for each average was approximately -25 in./mile. This basically means that the difference in roughness index between any 2 subsections must be in the order of at least 25 in./mile before it could be considered real and significant. Second, poor riding quality was exhibited by the tandem breakdown subsections, unlike the rest of the test pavement, these subsections were on the beginning of a slight horizontal curve. The related superelevation changes may have had some detrimental effect on the surface smoothness achieved in these areas.

CONCLUSIONS

The results of this bituminous base compaction study are summarized by the following conclusions:

1. All roller systems evaluated were able to achieve the critical 98 percent Marshall density required to satisfy department air voids specifications. However, because of a change in the planned roller sequence during construction, it was not possible to evaluate the compaction capabilities of the tandem roller in the multiple-lift paving mode.
2. The vibratory rollers did not produce base densities equivalent to those achieved by the department's standard compaction system within the coverage range (2 to 3) recommended by the manufacturers of the units.
3. In multiple-lift construction, 5 coverages of the vibratory compactors were necessary to produce essentially the same pavement density as that attained by the department's standard system. The associated densification approximated the normally needed 98 percent level of Marshall density. However, the vibratory rollers required nearly 30 percent more time than the standard system to achieve this density level.
4. In thick-lift construction, the department's standard compaction system was the most efficient of the rollers studied in achieving 99 percent Marshall density. The only conclusion that can be made pertaining to roller-compaction characteristics at the important 98 percent Marshall density level is that the RayGo unit was more efficient than both the Vibro-Plus and the tandem rollers. Unfortunately, no comparison can be made with the department's standard system for it achieved higher base densification (above 98 percent Marshal) with the minimum number of coverages evaluated.
5. Decompaction or density drop off of the bituminous base mixture was not found to occur within the range of vibratory roller coverages (2 to 5) evaluated.
6. Pavement riding quality was not adversely affected by either one of the vibratory compactors tested. In addition, no measurable improvement in riding quality was discernible when the tandem rather than the 3-wheeled roller was used to perform breakdown compaction.
7. The pavement surface produced by the vibratory and tandem rollers on the test mixture, after breakdown compaction, was such that finish rolling was not necessary. This suggests that, in instances where compaction time is not critical, certain economies could be realized by using the vibratory or tandem rollers instead of the department's standard system. The ability to achieve a smooth base of adequate density with 1 roller and 1 operator, rather than 2 rollers and 2 operators, could effect a reduction in construction costs.

DISCUSSION

M. Geller, Vibro-Plus Products, Inc.

The test results obtained from the use of the Vibro-Plus CA-25 S/N 251A at Stanhope in October 1970 should not be used as an indication of the current performance of the CA-25A. For the Stanhope test, the CA-25 S/N 251A utilized for the first time a dual-amplitude device that was intended to function at 2,400 vpm. During the test, the frequency was 1,700 vpm for the 4-in. lift and 1,350 vpm for the 2-in. lift.

Table 7 gives the centrifugal forces that CA-25 S/N 251A developed for the test and the forces that are now available at 2,400 vpm.

STATISTICAL ACCEPTANCE OF DENSE-GRADED BITUMINOUS MIXES BASED ON THE EXTRACTION OF PAVEMENT CORES

Robert F. Gorman, Division of Highways, Illinois Department of Transportation

In 1970 the Illinois Division of Highways initiated its first practical application of an "end result" type of specification on a bituminous resurfacing project. Nearly all responsibility for the design, control, and placement of the mix was placed on the contractor. Payment for the completed pavement was based on data obtained from the extraction of the uncompacted mix and from the density and thickness of the compacted pavement. Findings indicate that there is a mean shift of the core data, especially for the top size of aggregate. Adjusting target values (mix formula) by these differences indicates a very close correlation between the mix and the core extractions. In a few cases where there was considerable difference between the mix and the core extractions, there was usually as much difference between the testing of identical mix samples. The standard deviation for the core extractions, in most cases, was about the same as or less than that for the mix extractions. Based on the findings, core extraction tests could be used for the acceptance of bituminous mixtures provided judgment is used in adjusting mix formula values to coincide with the mean data shift due primarily to the cutting of larger sized aggregate during the coring operation.

•THE ILLINOIS Division of Highways has for years used modified-extraction, Marshall, and density tests for the design and control of bituminous pavement construction. Based on our experiences and knowledge as to the reliability of these tests, a contract for an "end-result" project was awarded during 1970 (1). Nearly all responsibility for the design, control, and placement of the mixture was placed on the contractors, and payment was based on the results of tests of the completed pavement taken by the state. Payment could entail a bonus or a penalty. The contractors were required to submit mix designs to the division's central laboratory for approval prior to mix placement. These designs were based on our present design criteria within mix specifications. Some mixes had to be redesigned and resubmitted before final approval was granted. The contractor's payment was based on his ability to meet the job mix tolerances, density, and thickness. Each of these items affected his unit bid price by one-third. Payment was established daily based on a predefined lot.

The intent of the end-result specification was to explore the feasibility of eliminating as many state job-control personnel as possible and still obtain a quality pavement. With this specification, state personnel were not required at either the plant or the paver. This is not to imply that the contractor was not required to do quality work. The resident engineer still had the responsibility of enforcing good construction practices. One drawback immediately encountered was that the specifications required the extraction test to be based on 5 random samples per lot of uncompacted mix taken from behind the paver. This required state personnel to be at a precise location at an exact time to obtain the necessary samples and, thereby, somewhat limited the value of the specification.

Sponsored by Committee on Quality Assurance and Acceptance Procedures.

We decided, therefore, to explore the possibility of basing mix acceptance of future jobs on extraction tests of cores required for density and thickness purposes. We reasoned that, by possibly shifting target values or job tolerances on future jobs or both, acceptance samples could be taken the day after mix placement and compaction.

SAMPLING PROCEDURES

As stated earlier 5 extraction samples of uncompacted mix were taken per lot at random locations from behind the paver. A lot was defined as "one day's production per paver, but in no case shall it exceed one mile of 2-lane pavement or its equivalent." Because a lot was in most cases a day's production, random locations, both longitudinal and transverse, were established by the use of tables of random numbers based on the contractor's estimated daily production. On most days we were able to obtain the required 5 samples.

Each mix sample was taken either with a 1-ft square template pressed into the uncompacted mix or by a pan placed ahead of the paving machine. The mix (approximately 18 lb) was placed in canvas bags and transported to the field laboratory for testing. The day following each day's construction, cores having $3^7/_8$-in. diameters were taken at 5 random locations from the previous day's work for use in determining density and thickness. These cores were taken by the contractor under the supervision of state personnel and transported to the field laboratory. Two cores were taken at each location and constituted 1 sample.

TESTING PROCEDURES

The mix samples received by the field laboratory were heated, if required, and split by an aggregate splitter until approximately a 1,200-gram sample was obtained. The remaining material was prepared for shipment to our central laboratory where identical tests as described later were performed with different personnel and equipment.

Because this was our first end-result job, we had one of our most experienced employees perform all field tests in order to limit, as much as possible, the chances of testing error. Employees who conducted the comparison tests in our central laboratory had considerably less experience, yet the results were, in our opinion, equally as good.

The extraction tests for asphalt content were conducted in accordance with AASHO T164-70, Method B (2), except that the ash content of the filtrate was not computed. Previous investigations in our central laboratory relating various filter paper types and ash contents revealed that no appreciable material is lost through the type of filters we use.

Upon completion of the extraction test, a sieve analysis was made of the remaining aggregate in accordance with AASHO T30-70 (2), omitting the wash test. Tests conducted in the past have shown that the extraction test virtually "washes" the type of aggregates used in our primary mixes. Therefore, we base all design and control of our mixes on dry sieves as a matter of practice.

After the density tests were completed, the cores were shipped to the central laboratory for extraction and sieve analysis tests. The 2 cores (approximately 750 grams each) obtained at each location were extracted as a single sample. The actual sample size varied according to the thickness of the cores (1.03 to 2.37 in.). No attempt was made to remove the cut aggregate from the periphery of the cores.

TEST DATA AND ANALYSIS

According to the requirements of the specifications, basis of payment for the mixture was computed on the average of 5 samples per lot on the controlling sieves as given in Table 1. The contractors were also required to maintain the mix within certain percentages on other sieves according to mix specifications, but only the sieves given in Table 1 were used for basis of payment; for purposes of clarity, only the lot averages on the controlling sieves are given.

From a research standpoint we received an added dividend with this contract in that it was awarded as a joint venture. One contractor paved half of the job and a second

Table 1. Extraction and sieve analysis.

Contractor, Course, and Lot Number[b]	Gradation[a] (percent) <1 in. >½ in.[c] <½ in. >No. 4[d]				<No. 4 >No. 10				<No. 40 >No. 80				<No. 200				Asphalt Content[a] (percent)			
	M	F	L	C	M	F	L	C	M	F	L	C	M	F	L	C	M	F	L	C
1-B-MF	21.5				7.5				12.4				4.8				4.8			
1-B-1		25.9	25.4	—[e]		10.9	10.6	—[e]		8.4	8.4	—[e]		4.8	5.0	—[e]		4.36	4.32	—[e]
1-B-2		22.8	22.2	19.4		8.5	8.0	10.2		10.0	10.1	9.5		5.2	5.4	5.9		4.88	4.82	4.78
1-B-3		23.7	23.4	21.6		7.6	7.4	7.1		10.3	10.4	10.4		4.2	4.4	4.8		4.74	4.70	4.52
1-B-4		25.4	23.1	19.3		6.2	6.2	8.8		10.6	11.0	11.8		4.4	4.5	4.9		4.70	4.70	4.92
1-B-5		25.8	22.3	16.8		6.2	5.9	8.1		10.2	10.4	11.3		5.2	5.7	5.9		4.74	4.58	4.82
1-B-CF	23.0				7.5				12.4				4.8				4.8			
1-B-6		25.5	21.5	20.1		6.8	7.0	6.3		11.0	11.0	10.3		4.3	4.9	5.7		4.80	4.74	4.76
1-B-7		25.6	22.9	18.6		6.7	5.8	8.7		11.0	11.0	10.8		5.0	5.6	5.3		4.94	4.92	4.82
1-B-8		24.5	24.2	21.7		12.2	10.0	10.4		9.9	10.0	9.7		4.5	4.8	5.0		4.73	4.76	4.94
2-B-MF	22.5				13.9				7.8				4.8				4.8			
2-B-9		22.5	22.1	20.6		10.8	10.3	9.7		8.6	7.8	8.6		6.1	6.1	5.8		4.54	4.80	4.44
2-B-10		24.6	20.4	17.0		10.5	11.2	12.3		7.4	7.8	7.8		5.9	6.3	6.2		4.44	4.54	4.50
2-B-11		19.9	23.4	18.2		13.2	12.0	12.2		7.4	7.1	7.9		6.0	5.9	6.2		4.70	4.36	4.42
2-B-12		18.9	18.8	16.8		12.1	11.9	12.9		7.1	7.1	7.4		4.9	5.2	5.1		4.78	4.54	4.52
2-B-13		20.7	20.6	15.8		12.2	11.3	12.4		7.5	7.7	7.8		4.7	4.8	5.5		4.78	4.54	4.80
2-B-14		21.8	22.2	15.8		11.7	11.3	12.7		7.2	7.5	7.8		5.2	5.2	5.7		4.96	4.74	4.90
2-B-15		21.0	17.0	16.3		12.7	12.4	12.2		7.4	7.8	7.4		5.5	5.8	5.7		4.88	5.02	4.60
2-B-16		20.2	20.2	20.4		13.5	13.0	12.7		7.8	8.0	7.7		5.0	5.1	5.1		5.02	4.94	4.40
1-S-MF	38.7				20.1				12.1				5.0				5.3			
1-S-1		35.8	36.2	32.3		22.4	21.7	24.2		10.3	10.3	10.1		5.6	5.7	5.9		5.33	5.23	5.34
1-S-2		35.9	36.5	33.5		22.3	21.7	23.4		10.8	10.3	10.4		5.9	6.3	6.6		5.26	5.10	5.36
1-S-3		39.4	38.6	35.6		20.9	20.9	21.5		10.0	9.8	9.7		5.5	5.9	7.3		5.18	5.20	5.38
1-S-4		40.0	40.2	36.6		20.7	19.9	22.9		10.1	10.3	10.3		5.6	6.0	6.1		5.16	5.28	5.36
1-S-5		39.8	40.2	36.9		20.0	19.4	21.1		10.2	10.2	10.5		5.9	6.3	6.4		5.24	5.36	5.26
1-S-6		40.9	41.4	37.5		19.2	18.5	20.6		10.2	9.7	10.1		6.0	6.0	6.3		5.22	5.42	5.40
1-S-7		41.9	41.4	37.4		20.3	19.9	21.7		9.9	10.3	10.0		4.9	5.1	5.5		5.30	5.40	5.30
1-S-8		37.4	39.1	37.6		21.4	20.2	21.2		10.5	10.5	10.4		5.6	5.8	5.6		5.44	5.20	5.36
1-S-9		39.3	39.6	37.0		21.2	20.7	22.7		10.6	10.4	10.0		5.3	5.5	5.3		5.26	5.16	5.32
2-S-MF	33.7				24.5				8.1				5.5				5.5			
2-S-10		30.3	32.5	30.2		24.6	23.4	23.2		9.1	9.0	9.0		5.6	5.6	5.9		5.64	5.46	5.40
2-S-11		32.3	33.6	31.6		21.8	20.7	22.8		9.3	9.4	9.3		5.8	6.1	6.1		5.60	5.48	5.34
2-S-CF	33.7				24.5				10.0				5.5				5.5			
2-S-12		30.4	32.8	29.5		24.9	23.2	24.1		10.0	9.7	10.3		5.3	5.4	5.9		5.62	5.68	5.62
2-S-13		30.4	31.4	29.1		25.4	24.6	25.2		10.3	9.6	10.5		5.1	5.5	5.3		5.76	5.80	5.60
2-S-14		30.0	30.6	29.4		23.8	23.4	25.3		10.6	10.5	9.8		5.0	5.4	5.7		5.86	5.66	5.54
2-S-15		30.6	35.3	29.4		24.9	22.4	24.4		9.4	9.0	9.8		5.1	5.0	5.4		5.72	5.62	5.56
2-S-16		32.7	35.6	30.9		24.4	22.9	23.7		9.1	8.7	10.0		4.7	4.8	5.2		5.60	5.58	5.58
2-S-17		32.5	32.3	26.9		25.4	25.0	25.7		8.9	8.6	9.2		4.9	5.0	5.7		5.58	5.62	5.64

[a]M = mix; F = mix analysis in field laboratory; L = mix analysis in central laboratory; C = core analysis in central laboratory.
[b]1 and 2 in first position = contractor; B = binder course; S = surface course; MF = original mix formula; CF = changed mix formula; digits in last position = lot number.
[c]Binder course.
[d]Surface course.
[e]No tests made.

Table 2. Deviations from target values based on mix and core extractions.

Contractor, Course, and Lot Number	Gradation and Asphalt Content	F and L Mix Target	Mean	Standard Deviation	Number	Core Target	Mean	Standard Deviation	Number
1-B-1, 2, 3, 4, 5	<1 in. >½ in.	21.5	24.0	1.48	10	17.5	19.3	1.96	4
	<No. 4 >No. 10	7.5	7.8	1.80	10	8.0	8.6	1.30	4
	<No. 40 >No. 80	12.4	10.0	0.88	10	12.6	10.8	1.01	4
	<No. 200	4.8	4.9	0.50	10	5.2	5.4	0.61	4
	Asphalt	4.80	4.65	0.18	10	4.80	4.76	0.17	4
1-B-6, 7, 8	<1 In. >½ in.	23.0	24.0	1.58	6	19.0	20.1	1.55	3
	<No. 4 >No. 10	7.5	8.1	2.47	6	8.0	8.5	2.06	3
	<No. 40 >No. 80	12.4	10.6	0.53	6	12.6	10.3	0.55	3
	<No. 200	4.8	4.9	0.45	6	5.2	5.3	0.35	3
	Asphalt	4.80	4.82	0.09	6	4.80	4.84	0.09	3
2-B-9, 10, 11, 12, 13, 14, 15, 16	<1 in. >½ in.	22.5	20.9	1.87	16	18.5	17.6	1.94	8
	<No. 4 >No. 10	13.9	11.9	0.95	16	14.4	12.1	1.02	8
	<No. 40 >No. 80	7.8	7.6	0.40	16	8.0	7.8	0.37	8
	<No. 200	4.8	5.5	0.55	16	5.2	5.7	0.42	8
	Asphalt	4.80	4.72	0.21	16	4.80	4.57	0.18	8
1-S-1, 2, 3, 4, 5, 6, 7, 8, 9	<½ in. >No. 4	38.7	39.1	1.97	18	36.3	36.0	1.90	9
	<No. 4 >No. 10	20.1	20.6	1.06	18	20.9	22.1	1.21	9
	<No. 40 >No. 80	12.1	10.2	0.28	18	12.0	10.2	0.25	9
	<No. 200	5.0	5.7	0.37	18	5.3	6.1	0.62	9
	Asphalt	5.30	5.26	0.10	18	5.24	5.34	0.04	9
2-S-10, 11	<½ in. >No. 4	33.7	32.2	1.37	4	31.3	30.9	1.0	2
	<No. 4 >No. 10	24.5	22.6	1.72	4	25.3	23.0	0.28	2
	<No. 40 >No. 80	8.1	9.2	0.18	4	8.0	9.2	0.21	2
	<No. 200	5.5	5.8	0.24	4	5.8	6.0	0.14	2
	Asphalt	5.50	5.54	0.09	4	5.44	5.37	0.04	2
2-S-12, 13, 14, 15, 16, 17	<½ in. >No. 4	33.7	32.1	1.88	12	31.3	29.2	1.29	6
	<No. 4 >No. 10	24.5	24.2	1.02	12	25.3	24.7	0.78	6
	<No. 40 >No. 80	10.0	9.5	0.70	12	9.9	9.3	0.45	6
	<No. 200	5.5	5.1	0.25	12	5.8	5.5	0.27	6
	Asphalt	5.50	5.68	0.09	12	5.44	5.59	0.04	6

contractor paved the other half. There were also changes in mix formulas. Although those changes were small, each change had to be approved by central laboratory staff.

Data given in Table 1 were used to prepare graphs of lot test data for the mix plotted as deviations from the target for each controlling sieve; Figures 1 and 2 show a typical plot. The percentages (110, 100, 90, 70 percent) on either side of the target values represent the payment percentage allotted the contractor for deviations from the target. These percentages were made to coincide with the sieve tolerances of 1, 2, 3, and 4 standard deviations. The contractor's unit bid price was adjusted by the lowest percentage payment indicated on any sieve or asphalt content for each lot. This percentage was averaged with the bid price adjustments for both density and thickness. In effect, each had a one-third effect on the unit price.

Table 2 gives the target, mean, and standard deviation values for all tests. It was decided to use the mean of all the mix extraction tests for comparison with the cores because this mean represents the best estimate of the mix (3). As expected, there was a shift in the core means from the mix means. The mean shift was computed for each sieve fraction and asphalt content per mix formula by averaging the differences between the mix means and their corresponding core means.

$$\text{Mean shift} = \left[\left(\overline{X}_{C1} - \overline{X}_{F1+L1}\right) + \left(\overline{X}_{C2} - \overline{X}_{F2+L2}\right) + \left(\overline{X}_{C3} - \overline{X}_{F3+L3}\right)\right]/3$$

For the binder course mix, <1-in. sieve and >½-in. sieve,

$$\text{Mean shift} = [(19.3 - 24.0) + (20.1 - 24.0) + (17.6 - 20.9)]/3$$
$$= -3.97$$

The mean shift from the mix for each type of core is as follows:

Sieve	Binder (percent)	Surface (percent)
<1 in. >½ in.	-4.0	
<½ in. >No. 4		-2.4
<No. 4 >No. 10	+0.5	+0.8
<No. 40 >No. 80	+0.2	-0.1
<No. 200	+0.4	+0.3
Asphalt content	0.0	-0.06

In the core data given in Table 2, the original mix formula has been adjusted by the preceding values to obtain an "adjusted target" for comparison to core extractions. Material passing the 1-in. sieve and retained on the ½-in. sieve for the binder course, for both contractors, showed the greatest variation between the mix and the cores. Because payment was based on the field extractions, there appeared to be, in some cases, as much difference between field and laboratory extractions as between field and core extractions. When there is a difference in pay based on the cores, it is usually 10 percent, which in essence would affect the contractor's pay only by 3.34 percent. For the material passing the ½-in. sieve and retained on the No. 4 sieve for the surface course, the correlation is somewhat better than that for the binder course, and, as previously indicated, testing variation accounts for some of the difference. The relationship is very good for the material passing the No. 4 sieve and retained on the No. 10 sieve for both the binder and surface courses. For the material passing the No. 40 sieve and retained on the No. 80 sieve, the figures are nearly identical for corresponding sets. Material passing the No. 200 sieve appears to have a very good correlation between the mix and cores. Except for an occasional sample that exhibited considerable variation between the mix and the cores, the correlation between mix and cores for the asphalt content is also good.

This study could have proved more enlightening if 4 cores had been taken from each location, one set extracted at the field laboratory and the other extracted at the central

Figure 1. Deviation from target values based on mix extraction (contractor 1, binder course, (1-in.)½-in. sieve).

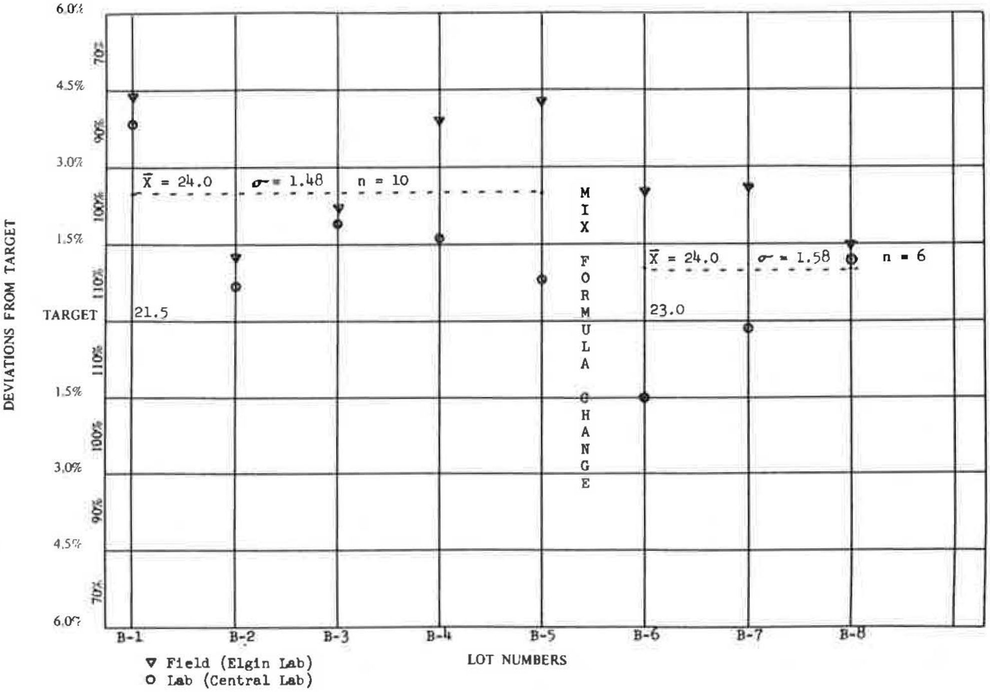
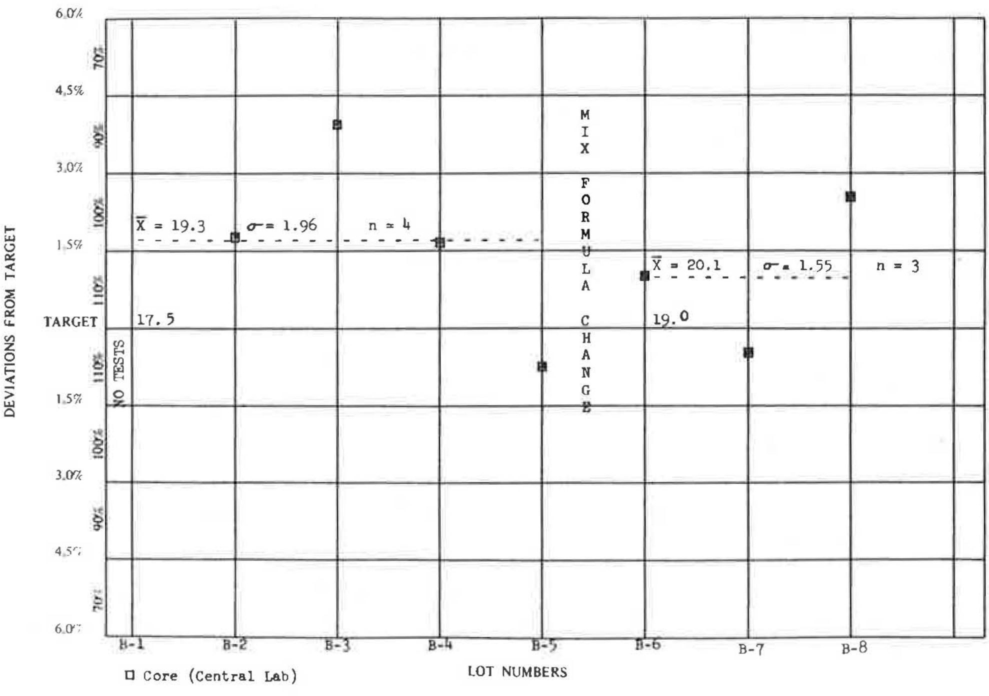


Figure 2. Deviation from target values based on core extraction (contractor 1, binder course, (1-in.)½-in. sieve).



laboratory. This would have given some indication of the core test variability as indicated with duplicate mix samples. Because the mix and the core samples were not of the same size or taken in the same location, we were very gratified with the results. In most cases the standard deviations for the cores were equal to or smaller than those for the mix, indicating very good uniformity.

CONCLUSIONS

The intent of this paper is to give engineers a practical tool for use in the analysis of existing bituminous pavements or a more convenient method of accepting bituminous mixtures whether by end-result or standard specifications. Based on the findings of this study, we offer the following conclusions:

1. Statistical methods of sampling and testing of pavement cores, based on 5 extraction samples per lot, should be as representative of the lot as the extraction of uncompacted mix provided certain adjustments are made;
2. A shift in target values, similar to those listed in this report, must be made when core extractions are equated to the mix formula;
3. The standard deviations for the extraction of the cores were about the same as or smaller than those for the extraction of the mix;
4. Deviations between the cores and the mix were, in some cases, no greater than between identical mix samples due to testing variations; and
5. One advantage of accepting the mix based on core extraction is that a recheck of the lot can be made by resampling when test values deviate excessively from target values.

ACKNOWLEDGMENTS

The author wishes to thank John J. Staab for performing the field tests and for tabulating and preparing data for this paper and also to thank W. Charles McIntyre for supervising the tests in the central laboratory and for making suggestions for this paper.

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DISCUSSION

David G. Tunnicliff, Warren Brothers Company

Gorman has presented a very useful and timely analysis of the use of core samples for purposes of acceptance of gradation and asphalt content of bituminous mixtures. Although core samples are used for this purpose, reliable data, such as Gorman's, on what to expect from cores have not been readily available. Three points deserve additional consideration.

1. The concept of a mean shift is both correct and necessary if acceptance of gradation and asphalt content is to be based on extraction of core samples. The question that remains is, What should the magnitude of the mean shift be? Mean shifts developed in this study are not questioned, but can they be used on another project? If not, how can the correct mean shift for another project be established? The correct magnitude of mean shift probably depends on a number of variables including aggregate gradation and type, asphalt content, pavement thickness, core diameter, and sampling and testing techniques.

2. A mean shift might not be necessary if different sampling were used. Gorman indicates that mean shift is necessary primarily because the larger aggregate sizes are cut during the coring operation. If larger diameter cores or larger sawed rectangles were used, then the proportion of cut particles in the sample would be reduced, and the need for mean shift might be eliminated. For example, personal experience indicates that mean shift for 6-in. diameter cores would be small, perhaps insignificant, on the surface mixture. Yet, the sample obtained by a 6-in. diameter core is much smaller than required by AASHO T168.

3. Gorman notes that the usual difference in the contractor's pay based on cores would be 3.34 percent compared to pay based on field extractions. Although 3.34 percent of the bid price may seem to be a reasonably small penalty or bonus, as the case may be, actually it is not. A 3.34 percent penalty is deducted from the contractor's profit if he is operating at a profit, and it might represent something like a third or more of his expected return on his investment in the lot. Otherwise, it means either no profit at all or a loss.

In the case at hand, the penalty would be solely the result of core sampling, rather than production of inferior materials. In order to stay in business the contractor must either bid high enough to cover this contingency or expect enough unearned bonuses, solely the result of core sampling, to balance the loss from undeserved penalties. Acceptance methods that tend to minimize both the rejection of acceptable materials and the acceptance of inferior materials are needed. It is not clear that core samples do this, but the cost of incorrect rejection or acceptance can be significant.

AUTHOR'S CLOSURE

I agree that the mean shifts used in this paper do not necessarily apply to mixes other than those used by these 2 contractors. We plan to conduct the same study with other mixes that include the variables that Tunnicliff mentions.

Six-in. cores or large samples sawed from the pavement should lessen, as he suggests, the need for a mean shift. We used 4-in. cores because samples of this size are compatible with our present testing equipment.

As stated previously, we plan to conduct an extensive study of core extractions versus mix extractions on future jobs. Before one acceptance procedure is used to replace another, we will have to assure ourselves that both methods will give the same results whether we have to change sample sizes or adjust our specification tolerance to account for degradation or do both.

VIRGINIA'S EXPERIENCE WITH A QUALITY ASSURANCE AND ACCEPTANCE SPECIFICATION FOR ASPHALTIC CONCRETE

C. S. Hughes, Virginia Highway Research Council

The Virginia Department of Highways has used a statistical quality assurance and acceptance specification for asphaltic concrete production since 1968. During this period, nearly 3 million tons of plant mix have been bought under this specification. The major benefits derived from use of the specification include a clear-cut understanding between the producer and the state as to control and acceptance responsibilities. Also, a large decrease in acceptance testing with no change in quality has occurred. Some aspects of the specification, however, could be improved by slight modifications.

•THE Virginia Department of Highways has had a quality assurance and acceptance specification for the control of the density of asphaltic concrete in effect since 1965 (1). This specification is now employed in the construction of most flexible pavements on the primary and Interstate systems. Encouraged by the success of this specification, the Department instituted in 1968 an acceptance specification for asphaltic concrete production (the Appendix contains the latest revision). During 1968, this specification was used on 3 construction projects and 1 maintenance schedule. The following year, 5 construction projects and 2 maintenance schedules were let to contract under the specification. In 1970, practically all of the asphaltic concrete used in construction and maintenance, more than 1.2 million tons, was bought under this acceptance specification. In 1971, the total exceeded 1.4 million tons. The specification is used on both state and federally financed projects; the Federal Highway Administration approves the latter on a project-by-project basis. It is used on all projects having more than 4,000 tons of one mix type. The reason for this practice is primarily administrative because the state's personnel force is small at asphalt plants producing very limited quantities.

SPECIFICATION

In a specification for the acceptance of asphaltic concrete, many items must be included to ensure a quality material; and many additional items must be included to ensure a clear understanding of the respective responsibilities of the consumer and the producer. It is imperative that the producer realize that his responsibility lies in supplying a product that will meet specifications and that the consumer realize that he has the responsibility of testing the product for acceptance.

The 5 elements discussed below are necessary in any thorough acceptance specification. Virginia's method of handling these elements is indicated in the discussion. (Elements 1 through 4 are based primarily on technical and administrative considerations and not statistical ones.)

1. The specification must identify the place of testing. The asphalt plant is designated because sampling and testing can be done quicker and more conveniently there than elsewhere. (Before this specification was written, the point of testing was not stated. Sometimes the asphaltic concrete was tested at the plant, sometimes at the district lab, and sometimes at both places. The establishment of a single place for acceptance testing is important.)

Sponsored by Committee on Quality Assurance and Acceptance Procedures.

2. The method of test must be prescribed and must not be changed. The reflux extractor is designated. (Although this method is not so rapid as the centrifuge, it is more accurate. It is important to state the method of test because the tolerances are based on it. If the test method is changed, new acceptance limits must be established.)

3. A definite lot size must be stated. Originally, the lot size was 2,000 tons, which was thought to be generally compatible with previous testing rates. This was subsequently modified as will be discussed later. (A great deal of discussion accompanied the decision to use a lot size of tons rather than one based on a time period, such as a day's production. In the end, the decision was based on administrative considerations, the primary one being the number of personnel normally assigned to a plant.)

4. The specification must state the number of tests to be obtained per lot. Four tests per lot are used to judge acceptance because this number is generally compatible with the lot size determined by previous testing rates.

5. Naturally, the elements to be tested for acceptance and the tolerances to be applied must be stated. In the Virginia specification, acceptance is determined by the application of a tolerance to the average of 4 samples for the process average of each lot. The allowable variability is based on the overall standard deviation of a particular mix for the entire project.

Ideally, the contractor should run his own control tests and not rely on the state for guidance. This suggestion is not at present very realistic because many contractors are not familiar with control testing. Therefore, strictly to aid the contractor, the state's inspection personnel plot the acceptance data in the form of a control chart for the contractor's use if he so desires.

In any acceptance specification, provision must be made for handling material that does not meet the established tolerances. If the state is not going to control the product and thus infringe on the contractor's prerogative, there is a need to apply an adjustment factor to that material not meeting the tolerances. The adjustment procedure is spelled out so that the contractor at any time knows what, if any, adjustment will be made.

BENEFITS

A natural question is, Why are quality assurance and acceptance specifications desirable? Under the specification at hand, 2 advantages are evident thus far.

First, the specification required detailed decisions concerning what the state really wanted in the way of asphaltic concrete and how this material could be specified. The discussions leading to these decisions were very enlightening technologically and administratively. Some of the facets that had to be considered were (a) changes in the tolerances to make them compatible with normal production; (b) complete confidence in the plant inspection personnel, who after all actually become purchasing agents of the material; and (c) clear realization that plant control is the contractor's responsibility.

Second, the amount of acceptance testing has been greatly reduced. As a typical example, a project completed in 1970 required 37,267 tons of asphaltic concrete. The old specification under which this project was let to contract required 121 control tests and 114 acceptance tests for a total of 235 tests. Under the present acceptance specification, 75 tests would have been required—a reduction of 38 percent in acceptance tests and 68 percent in total tests.

One might also ask whether the quality of the product is sacrificed in the acceptance procedure that requires fewer tests. It will be shown later that the material being produced under the present specification is essentially the same as that produced in the past under a combination acceptance and control procedure.

REVISIONS OR MODIFICATIONS

A new specification generally must be revised or modified as a result of the experience gained in applying it on a daily basis. For this specification, a cooperative study (2) was established with the Federal Highway Administration to analyze the data collected in 1970.

Even before the cooperative study, a need for modification was realized on large projects for which asphalt plants were producing 4,000 tons or more per day. Under the original specification this amount of material would have necessitated 8 or more tests per day, which was impossible under the manpower and equipment constraints found at the plants. To alleviate this problem, the specification was modified to increase the lot size to 4,000 tons on contracts calling for more than 50,000 tons.

Several of the conclusions from the cooperative study are discussed below.

Comparison of Asphalt Production

One of the first conclusions from the study of the 1970 data was that the asphalt produced was amazingly similar to that produced in 1967, from which the tolerances for the acceptance specification were derived. Some explanation is necessary for the data given in Tables 1 and 2, which show the closeness of the test results for the material produced in these 2 years. It should be noted that the acceptance specification was not introduced to upgrade the quality of the asphaltic mixes.

The ability of a plant to remain within the process tolerances for each sieve and the asphalt content is based on 2 production characteristics:

1. An ability to "hit" the job mix, which is determined by taking the difference between the job mix and the production average, and
2. The production variability, which is simply the production standard deviation and is numerically equivalent to 2 standard errors because the sample size per lot is 4.

When these 2 characteristics are combined, the "total" value is best described by data shown in Figure 1 for the 1970 I-2 mix, No. 4 sieve. The tolerance for this sieve is 4.5 percent measured from the job mix. The data analyzed for 39 projects indicated that the production average missed the job mix by 1.50 percent, and the measured standard deviation (or 2 standard errors) equaled 2.68 percent, for a total value of 4.18 percent.

As long as the sum of the combined values for a majority of the projects is close to the tolerance, the tolerance can be considered satisfactory; however, when the combined value consistently exceeds the tolerance, then the tolerance should be increased. Conversely, if the total variability does not consistently approach the tolerance, then the tolerance should be decreased.

Admittedly, this concept is somewhat foreign to the usual statistical approach of control limits. However, because the job mix is consistently different from the production average, as data given in Table 1 demonstrate, this approach appears rational.

Data given in Table 1 indicate that the acceptance system induced no changes in the overall asphaltic concrete production.

Method of Variability Acceptance

One of the few complaints from contractors related to the method of variability acceptance. One of the greatest concerns is that the test results be immediately available so that the contractor can know whether he should institute plant changes in order to avoid price adjustments. The lot size used in the present specification provides the needed information for the process average very well. However, the variability acceptance is not determined until the entire project is finished. Although the contractors could have determined their own variability at any time, this point was somewhat disconcerting to them.

To develop an alternative to the present variability procedure, if one were needed, we analyzed the 1970 data by determining the range on each lot as an estimate of the standard deviation because of the simplicity of this determination and because it is the commonly accepted statistical method of determining the variability in production processes.

In this analysis, the first question that had to be answered was how well the range predicted the standard deviation. The statistical formula for predicting the standard deviation, s, from the range, R, for sample groups of 4 is

$$s_r = R \times 0.5$$

Figure 1. Concept of "total" value.

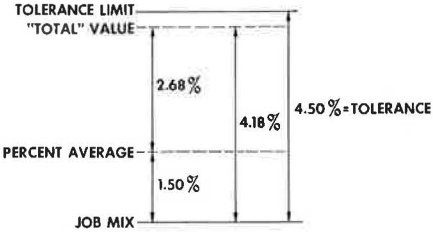


Table 1. Process tolerances.

Item	JM-$\bar{X}$	S	Total
Sieve			
3/4 in.	2.0	3.5	5.5
1/2 in.	2.0	3.5	5.5
3/8 in.	2.0	3.5	5.5
No. 4	1.5	3.0	4.5
No. 8	1.5	3.0	4.5
No. 30	1.5	3.0	4.5
No. 50	1.0	2.0	3.0
No. 200	0.5	1.0	1.5
Asphalt content	0.25	0.25	0.5

Note: JM-$\bar{X}$ = job mix less production average; S = production standard deviation; and total = sum of the 2 values.

Table 2. Summary of average standard deviations and differences between job mix and production average.

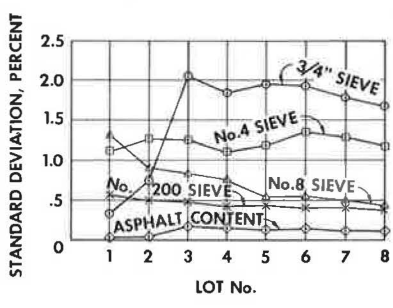
	1967[a]						1970								
	Mix I-2			Mix S-5			Mix B-3			Mix I-2			Mix S-5		
Item	JM-$\bar{X}$	S	Total	JM-$\bar{X}$	S	Total	JM-$\bar{X}$	S	Total	JM-$\bar{X}$	S	Total	JM-$\bar{X}$	S	Total
Sieve															
3/4 in.	2.63	0.79	3.42				1.39	2.75	4.14						
3/8 in.	1.09	3.10	4.19	2.95	1.54	4.49				1.82	3.30	5.12	1.57	1.37	2.94
No. 4	2.49	3.09	5.58	2.35	2.90	5.25	1.85	2.91	4.76	1.50	2.68	4.18	2.02	2.90	4.92
No. 8	1.67	2.68	4.35	1.49	2.89	4.38	1.26	2.53	3.79	1.94	2.20	4.14	1.29	2.79	4.08
No. 30				1.18	1.74	2.92							1.37	1.87	3.24
No. 50	0.75	1.29	2.04	1.60	1.39	2.99				1.10	1.15	2.25	0.82	1.31	2.13
No. 100	0.97	1.14	2.11	1.02	1.41	2.43									
No. 200				0.55	1.17	1.72	0.52	0.61	1.13	0.60	0.64	1.24	0.59	0.76	1.35
Asphalt content	0.08	0.22	0.30	0.09	0.24	0.33	0.10	0.22	0.32	0.15	0.24	0.39	0.12	0.22	0.34

Note: JM-$\bar{X}$ = job mix less production average; S = production standard deviation; and total = sum of the 2 values.
[a]No mix B-3 was tested during 1967.

Table 3. Standard deviation versus range estimate.

	Base		Intermediate		Surface	
Item	s	s_r	s	s_r	s	s_r
Sieve						
3/4 in.	2.75	2.73	—	—	—	—
3/8 in.	—	—	3.07	3.43	1.37	1.44
No. 4	2.91	2.79	2.67	2.79	2.90	3.03
No. 8	2.53	2.45	2.42	2.66	2.79	2.93
No. 30	—	—	—	—	1.83	1.93
No. 50	—	—	1.15	1.17	1.30	1.87
No. 200	0.61	0.57	0.62	0.64	0.76	0.84
Asphalt content	0.22	0.22	0.24	0.24	0.21	0.23

Figure 2. Typical association between plant variability and time.



where s_r is the standard deviation estimated from the range.

Table 3 gives the average calculated standard deviation and the average standard deviations estimated for the range for each mix type. These values were also determined on a project-by-project basis, and the F-ratio was determined from the 2 variances. The F-values were checked for significance at the 95 percent confidence level, and a significant difference was found in only 10 out of 637 cases. The absence of significant differences and the closeness of the average results are certainly evidence that the range method can accurately and consistently predict the standard deviations and that there is no statistical reason for not using the range method as a variability acceptance procedure.

Variability Versus Length of Operation

During the development of the acceptance specification, there was some contention that for the first day or two of plant operation the variability is much higher than it is after the process has been running for a while. Contractors thought that because this argument might be valid the test results for the first 1,000 or so tons should not be used in the variability criterion. In order to verify or refute this contention we made an analysis of accumulated standard deviations plotted against the number of lots tested. This analysis resulted in a graph for each mix and project as shown in Figure 2. The graphs were examined visually, and the variability of each sieve was judged to be either stable, increasing, or decreasing.

The first observations were that about 50 percent of the project variabilities tended to remain stable, and slightly more increased than decreased. It also appeared that the variabilities of the No. 200 sieve and the asphalt content tended to remain more stable than did those for the other sieves. These observations tend to refute the contention that the variability decreases over time of operation.

OBSERVATIONS

Administrators of the Virginia Department of Highways appear to be very satisfied with the operation of the specification. It is obvious that the product being purchased has not diminished in quality, and yet inspection costs have decreased appreciably. Contractors took a wait-and-see attitude on the new specification and, naturally, felt some trepidation. However, after 2 years, by and large they feel that it is successful and that they are getting acceptance and yet are allowed to control their processes as they wish. The acceptance specification is viewed as a progressive step and has led to the use of similar specifications in other areas.

ACKNOWLEDGMENT

The author gratefully acknowledges the data supplied by J. G. G. McGee and D. A. Traynham of the Central Office of the Virginia Department of Highways. Also, many of the analyses were done by S. N. Runkle of the Council's Data Systems and Analysis Section, who also co-authored the final report on this project.

REFERENCES

1. Hughes, C. S. Methods of Compaction Control. AAPT Proc., Vol. 36, 1967.
2. Runkle, S. N., and Hughes, C. S. Review of a Bituminous Concrete Statistical Specification. Virginia Highway Research Council, May 1971.

APPENDIX

SPECIAL PROVISIONS FOR SECTION 212, BITUMINOUS CONCRETE (STATISTICAL QUALITY CONTROL SPECIFICATION, revised 4-1-71)

Section 212.03 of the 1970 specifications is completely replaced by the following:

Section 212.03: Job-Mix Formula—The contractor shall submit, for the Engineer's approval, a job-mix formula for each mixture to be supplied for the project prior to starting work. The job-mix formula shall be within the design range specified in Table A-I, Bituminous Concrete Mixtures, for the particular type of bituminous concrete specified. The job-mix formula shall establish a single percentage of aggregate passing each required sieve size, a single percentage of bituminous material to be added to the aggregate and a single temperature at which the mixture is to be produced. The job-mix formula for each mixture shall be in effect until modified in writing by the Engineer.

Materials from more than one source shall not be used alternately nor mixed when used in surface courses without the written consent of the Engineer. Where additional sources of materials are approved, a job-mix formula shall be established and approved before the new material is used. When unsatisfactory results or other conditions make it necessary, the Contractor shall prepare and submit a new job-mix formula for approval. Approximately one week may be required for the evaluation of a new job-mix formula.

The Marshall design density of a mixture shall not exceed 98.0 percent of the theoretical maximum density. In the event Marshall densities begin to exceed 98 percent of theoretical maximum density during construction the Contractor shall alter the grading of the aggregate or otherwise shall obtain his aggregate from a different source.

Section 212.06 is completely replaced by the following:

Section 212.06: Plant Inspection—The preparation of all bituminous mixtures shall be subject to inspection at the plant. For this purpose the Contractor shall provide a suitable building to be used as a field laboratory in accordance with requirements of Supplemental Specifications for Section 517. The Contractor shall

Table A-I. Bituminous concrete mixtures (design range).

Type	Percentage by Weight Passing Square Mesh Sieves *											Percent Bitumen	Mix Temperature (At Plant)
	2	1 - 1/2	1	3/4	1/2	3/8	No. 4	No. 8	No. 30	No. 50	No. 200		
S-1							100	94 - 100	69 - 77	38 - 49	2 - 6	8.5 - 10.5	245 - 280°F
S-2						100	91 - 100	69 - 77	26 - 34	16 - 24	4 - 8	9.5 - 12.0	245 - 280°F
S-3						100	88 - 100	79 - 87	36 - 44	21 - 29	5 - 9	6.5 - 10.5	210 - 220°F
S-4					100	88 - 100	76 - 90	66 - 74	31 - 39	16 - 24	4 - 8	5.5 - 9.5	245 - 280°F
S-5					100	83 - 97	53 - 67	41 - 49	19 - 27	11 - 19	4 - 8	5.0 - 8.5	245 - 280°F
I - 1			100	88 - 100		86 - 100	81 - 95	74 - 82	39 - 47	20 - 28	4 - 8	5.0 - 7.5	245 - 280°F
I - 2			100	95 - 100		63 - 77	43 - 57	31 - 39		6 - 14	2 - 6	4.5 - 8.0	245 - 280°F
B-1			100	88 - 100			78 - 92	71 - 79	41 - 49	22 - 30	2 - 6	3.0 - 6.5	245 - 280°F
B-2		100		56 - 70			21 - 35	16 - 24			1 - 5	4.0 - 6.0	210 - 220°F
B-3		100		73 - 85			38 - 48	28 - 35			2 - 6	4.0 - 7.0	245 - 280°F
B-4	100	88 - 100		78 - 92			51 - 65	44 - 52	26 - 34		5 - 13	2.5 - 4.0	245 - 280°F
P-1						100	86 - 100	76 - 84	36 - 44	21 - 29	5 - 7	6.5 - 9.5	145 - 155°F
P-2					100	83 - 97	53 - 67	41 - 49	19 - 27	9 - 17	4 - 8	6.5 - 8.5	145 - 155°F
P-3				100		63 - 77	38 - 52	24 - 32			1 - 5	5.5 - 7.5	145 - 155°F

*In inches, except where otherwise indicated. Numbered sieves are those of the U. S. Standard Sieve Series.

furnish, maintain and replace as condition necessitates, the following testing equipment:

2 reflux extractors (2,000 gram capacity)
2 electric hot plates (thermostatically controlled) suitable for use with the above reflux extractors
(One additional reflux extractor and one additional electric hot plate shall be furnished for each 1,000 tons of material produced per day in excess of 2,000 tons except when a lot size of 4,000 tons is used.)
1 beam-type balance meeting the following minimum requirements:
(a) Capacity—Not less than 2,000 grams
(b) Dial—"Over" and "under" with center mark
(c) Beam—12 inch minimum length, 100 gram capacity, notched in increments of 1 gram, with hanging and self-locking poise counterweight
1 set of graduated gram weights
1 electric hot plate or oven for drying sample (temperature range to at least 300° F)
1 mechanical sieve shaker
1 set of sieves (2" through #200 mesh)
1 separator for separating the plus and minus ¾ inch material for bituminous concrete base courses (Minimum dimensions of ¾ inch sieve shall be 12 inches by 12 inches.)
1 set of milk scales
miscellaneous supplies—pans, brushes, scoops or large spoons, several 1,000 ml. graduated beakers and an adequate supply of running water, which is not to exceed 80° F in temperature, shall be provided.

The above mentioned equipment shall be installed ready for operation in a field laboratory meeting the requirements of Supplemental Specifications for Section 517. Additionally, the building shall be adequately ventilated by exhaust fan.

The requirements stated hereinabove shall not be construed as a nullification of the requirements of Sections 106.05 and 200.01.

The Department's representative shall have ready access to all parts of the plant for checking the accuracy of the equipment in use, inspecting the condition and operation of the plant and for any purpose in connection with the materials and their processing.

Section 212.29 is added as follows:

Section 212.29: Acceptance—Sampling for determination of gradation and asphalt content will be performed at the plant, and no further sampling will be performed for these properties. However, should visual examination reveal that the material in any batch or load is obviously contaminated, deficient in asphalt content or not thoroughly mixed, that batch or load will be rejected without additional sampling or testing of the lot. In the event it is necessary to determine, quantitatively, the quality of the material in an individual batch or load, one sample (taken from the batch or load) will be tested and the results compared to the "process tolerance for one test" as described hereinbelow. The results obtained in the testing of a specific individual batch or load will apply only to the batch or load in question. Gradation and asphalt content determinations will be performed in the plant laboratory furnished by the Contractor; however, the Department reserves the right to discontinue the use of the plant laboratory for acceptance testing in the event of mechanical malfunctions in the laboratory equipment and in cases of emergency involving plant inspection personnel. In the event of such malfunctions or emergencies, acceptance testing will be performed at the District or Central Office laboratory until the malfunction or emergency has been satisfactorily corrected or resolved.

Acceptance for gradation and asphalt content will be based upon a mean of the results of four tests performed on samples taken in a stratified random manner from each 2,000 ton lot (4,000 ton lot when the contract item is in excess of 50,000 tons).

A lot will be considered to be acceptable for gradation and asphalt content if the mean of the results obtained from the four tests fall within the following process tolerances allowed for deviation from the job-mix formula:

Sieve	Process Tolerance (percent passing)
Top size	±0.0
1½"	5.5
¾"	5.5
½"	5.5
⅜"	5.5
#4	4.5
#8	4.5
#30	4.5
#50	3.0
#200	1.5
Asphalt content*	0.5

*Asphalt content will be measured as extractable asphalt.

In the event asphalt input is monitored by automated recordation, the above process tolerance for asphalt will not apply. Variability control for asphalt content will be evaluated based upon extractable asphalt. At any time the asphalt content, as evidenced by automated recordation, deviates more than ±0.2 percent from that shown in the job-mix formula, the production shall be halted and corrective action taken to bring the asphalt content to within this tolerance.

The temperature of the mixture at the plant shall not vary more than ±20° F from the approved job-mix temperature. The temperature of the mixture at the time of placement in the road shall not be more than 30° F below the approved job-mix temperature. Loads which do not conform to these temperature tolerances will be rejected.

In the event that the job requires less than 2,000 tons of material; or that the amount of material necessary to complete the job is less than 2,000 tons (4,000 tons for contract items in excess of 50,000 tons); or that the job-mix formula is modified within a lot, the mean results of samples taken will be compared to a new process tolerance, computed as follows:

Process tolerance for one test = process tolerance for mean of four tests/0.5
Process tolerance for mean of two tests = process tolerance for mean of four tests/0.7
Process tolerance for mean of three tests = process tolerance for mean of four tests/0.9

Individual test results and lot averages obtained from acceptance testing will be plotted on control charts as the information is obtained. Standard deviations, when computed, will be made available to the Contractor. However, the Inspector will in no way attempt to interpret test results, lot averages or standard deviations for the Contractor in terms of needful plant or process adjustments.

Section 212.30 is added as follows:

Section 212.30: Adjustment System–An adjustment of the unit bid price will not be made for the value of one test result or the mean value of two or three test results, unless circumstances as stated in Section 212.29 require that the lot size be less than 2,000 tons (4,000 tons for contract items in excess to 50,000 tons). Should the value of one test result or the mean value of two or more test results, as required by Section 212.29 fall outside the allowable process tolerance, an adjustment will be applied to the unit bid price as follows:

Sieve	Adjustment Points for Each 1 Percent That the Gradation Is Out of Process Tolerance
2"	1
$1\frac{1}{2}$"	1
1"	1
$\frac{3}{4}$"	1
$\frac{1}{2}$"	1
$\frac{3}{8}$"	1
#4	1
#8	1
#30	2
#50	2
#200	3

A one point adjustment will be applied for each 0.1 percent that the material is out of the process tolerance for asphalt content.

In the event the total adjustment for a lot is greater than 25 points, the failing material shall be removed from the road. In the event the total adjustment is 25 points or less and the Contractor does not elect to remove and replace the material, the unit price paid for the material will be reduced 1 percent of the unit price bid for each adjustment point. The adjustment will be applied to the tonnage represented by the sample or samples.

The Contractor shall control the variability of his product in order to furnish the project with a uniform mix. When the contract item is greater than 4,000 tons and an adjustment is necessary as indicated in the following table, it shall be for the entire quantity of that type material on the project based upon its variability as measured by the standard deviation.

Sieve Size and Asphalt Content	Standard Deviation		
	1 Adjustment Point	2 Adjustment Points	3 Adjustment Points
$1\frac{1}{2}$"	4.6-5.5	5.6-6.5	6.6-7.5
$\frac{3}{4}$"	4.6-5.5	5.6-6.5	6.6-7.5
$\frac{1}{2}$"	4.6-5.5	5.6-6.5	6.6-7.5
$\frac{3}{8}$"	4.6-5.5	5.6-6.5	6.6-7.5
#4	4.6-5.5	5.6-6.5	6.6-7.5
#8	4.1-5.0	5.1-6.0	6.1-7.0
#30	4.1-5.0	5.1-6.0	6.1-7.0
#50	3.1-4.0	4.1-5.0	5.1-6.0
#200	2.1-3.0	3.1-4.0	4.1-5.0
Asphalt content	0.33-0.42	0.43-0.52	0.53-0.62

The unit bid price shall be reduced by 0.5 percent for each adjustment point applied.

The disposition of material having standard deviations larger than those shown in the table, shall be determined by the Engineer.

Section 212.31 is added as follows:

Section 212.31: Referee System—(a) In the event the test results obtained from one of the four samples taken to evaluate a particular lot do not appear to be representative, the Contractor or the Engineer may request that the results of the questionable sample be disregarded; whereupon, tests will be performed on five additional samples taken from randomly selected locations in the roadway where the lot was placed. The test results of the three original (unquestioned) samples will be averaged with the test results of the five road samples and the mean of the test values obtained for the eight samples will be compared to the following process tolerance:

Process tolerance for mean of eight tests = process tolerance for mean of four tests/1.4

(b) In the event the Contractor elects to question the mean of the four original test results obtained for a particular lot, he may request additional testing of that lot. Upon receipt of written request for additional testing, the Department will test four samples taken from randomly selected locations in the roadway where the lot was placed. The test results of the original four samples will be averaged with the test results of the four additional road samples and the mean of the test values obtained for the eight samples will be compared to the "process tolerance for mean of eight tests" as described hereinabove.

In the event the mean of the test values obtained for the eight samples is within the process tolerance for the mean of the results of eight tests, the material will be considered acceptable. In the event the mean of the test values obtained for the eight samples is outside of the process tolerance for the mean of the results of eight tests, the lot will be adjusted in accordance with the adjustment rate specified hereinabove.

Additional tests, requested by the Contractor under the provisions of Section 212.31 (a) and (b), will be paid for by the Contractor in the event the mean of the test values obtained for the eight samples falls outside of the process tolerances. Such additional tests shall be paid for at a rate of five times the bid price per ton of material per sample.

RATIONAL QUALITY ASSURANCE

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The paper describes aspects of a rational system for the application of statistical quality-control procedures in highway construction. Norms for the judgment of compliance with a double specification limit are defined in terms of parameters that will be meaningful to road engineers. These norms are further qualified in terms of a fundamental relation of the applicable coefficient of variation and the number of observations required for judgment purposes. Corresponding judgment norms are also presented for use when a product that fails to comply with specifications when first submitted is consequently resubmitted for acceptance. Information is also presented that is required for the practical application of the scheme. This includes coefficients of variation that are representative of current practice, desired frequency of sampling, and suggested lot sizes. Finally, the application of the method is illustrated by means of a proposed system logic and a practical example based on a double specification limit.

• HIGHWAY engineers have for many years stressed the need for a rational approach in the judgment of the degree that highway construction processes comply with design specifications (1, 2, 3). Although that need has received some attention in the past, it is generally conceded that no comprehensive scheme is yet available for use in highway engineering (1, 4, 5). Past experience, however, has revealed the important factors that must be taken into account in the development of such a scheme. Some of these are as follows (6):

1. The scheme should be mathematically formulated (this requirement is satisfied only if the properties of the product that are subject to quality assurance are distributed in a reasonably random manner about a mean value);
2. The scheme should be adaptable to comply with the requirements of lower, higher, or double specification limits and should be applicable to both process and acceptance control;
3. The scheme should be based on variability requirements that are representative of existing practice and that can be adjusted from time to time by means of an information feedback service;
4. There should, if possible, be an incentive for the producer to improve the uniformity of the product and thereby to effect modified specification requirements and associated economic benefits to him;
5. There should be a rational means for deciding on the required number of tests, for they directly affect the determination of the judgment norms;
6. So that the same rejection risks apply throughout, provision must be made for the determination of the judgment norms that apply when a product is resubmitted after initially failing to comply with the specified requirements; and
7. The scheme should be relatively easy to apply in practice and should be adaptable to permit desired cost benefits or sophistication in quality-assurance techniques to be obtained.

Sponsored by Committee on Quality Assurance and Acceptance Procedures.

A quality-assurance scheme has recently been developed (6, 7) that substantially complies with these requirements; aspects of this work are described in this paper. Use is made of simple statistical theory associated with the normal and chi-square distributions.

QUANTIFICATION OF JUDGMENT NORMS

Quality assurance is the process used to determine whether the properties of a specified product representing particular design requirements have been satisfactorily met by the corresponding properties of the submitted or measured product. In practice the properties of the measured product cannot be directly compared with those of the specified product, and it is convenient to define a model product that effectively represents the specified product with which the measured product can be compared for judgment purposes. The specified product is quantified for both lower and double specification limits. In the latter case the standard and modified model products are also defined and mathematically formulated, and the use of that information for exercising rational quality assurance is demonstrated by means of a practical example.

Specified Product for Lower Specification Limit

A variable representing a product property that, if it satisfies design requirements, must comply with conditions for a lower specification limit can effectively be defined by a minimum value x_s, below which not more than ϕ percent of the individual values of the magnitude of the variable should fall, and by a maximum value represented by a standard deviation σ_s or coefficient of variation V_s. Because the distribution of the magnitude of the variable can be represented by a normal distribution (6, 8, 11, 12, 13) with a mean value $\bar{x}$, the relation among the various parameters can be formulated as follows (Fig. 1, curve I):

$$\bar{\bar{x}} = x_s + t_\phi \sigma_s = x_s/(1 - t_\phi V_s)$$

where t_ϕ is the standard normal deviate for ϕ.

$\bar{\bar{x}}$ is furthermore the true mean value of a similar population of values that consist of the mean of n individual random values $\bar{x}_n$ instead of the single values x. In this case the standard deviation of the distribution of the $\bar{x}_n$ values about $\bar{\bar{x}}$ represented by σ_n is given as follows (Fig. 1, curve II):

$$\sigma_n = V_s \bar{\bar{x}}/\sqrt{n}$$

If, as in normal practice, $\bar{x}_n$ is taken as representative of the true mean value of the property, then it can be proved that the distribution of single values in this case has a standard deviation σ_3 (Fig. 1, curve III), which is given by

$$\sigma_3 = \sqrt{\sigma_1^2 + \sigma_2^2} = v_s \bar{\bar{x}} \sqrt{(n + 1)/n} = \nu_s \bar{\bar{x}} \tag{1}$$

where $\nu_s = V_s \sqrt{(n + 1/n}$ represents the normalized coefficient of variation. It should be noted that $\nu_s = V_s$ for $n = \infty$. Henceforth in this paper the strictly correct ν instead of V will be used to indicate the applicable coefficient of variation. As a practical approximation, V can be substituted for ν in the relevant equations by assuming that $\sqrt{(n + 1)/n} = 1$.

Specified Product for Double Specification Limit

The equations defining the specified products for lower and upper specification limits can be effectively combined to quantify the corresponding product for a double specification limit. These are as follows:

$$\text{Lower specification limit } \bar{\bar{x}} = x_s/(1 - t_\phi \nu_s) \tag{2}$$

$$\text{Upper specification limit } \bar{\bar{x}}' = x_s'/(1 + t_{\phi}\nu_s) \tag{3}$$

The following additional conditions must, however, be taken into account:

1. There should be a separation h between $\bar{\bar{x}}$ and $\bar{\bar{x}}'$ to allow for inherent variations in the mean of the measured product;
2. The allowable percentage defect ϕ must be the sum of the percentage defects at both x_s and x_s' that, as shown in Figure 2, are respectively $y\phi$ and $(1 - y)\ \phi$; and
3. The absolute mean value $X = (x_s + x_s')/2$ is the target value in double limit specifications.

An analysis of specification and test data from current practice revealed that the separation h effectively varies between $0.75\ \nu_s X$ and $1.25\ \nu_s X$. An average value of $h = \nu_s x$ has, therefore, been assumed for use in this paper.

If the acceptable approximation is made that $\nu_s X = \nu_s\bar{\bar{x}} = \nu_s\bar{\bar{x}}'$, it is evident from data shown in Figure 2 that $\bar{\bar{x}}' - \bar{\bar{x}} = h = \nu_s X\left[t_{(1-y)\phi} - t_{y\phi}\right]$ or

$$h/\nu_s X = \left[t_{(1-y)\phi} - t_{y\phi}\right] \tag{4}$$

The relation between y and ϕ is also shown in Figure 3.

The specified product for double specification limits can be formulated as follows:

$$\left.\begin{aligned} x_s &= X\left[1 - \nu_s(0.5 + t_{y\phi})\right] \\ x_s' &= X\left[1 + \nu_s(0.5 + t_{y\phi})\right] \end{aligned}\right\} \tag{5}$$

Measured Product

The magnitude of a property of a measured product is characterized by the mean value $\bar{x}_n$ determined from a limited number of observation data n. The variability as characterized by the range R_n is determined from the same observation data. Those 2 quantities are then compared with the judgment norms established for the model product as the quality-assurance process is exercised.

Standard Model Product

The product with which a variability ν_s is associated is formulated to serve as a convenient link between the specified and measured products for quality-assurance purposes. One of the main motivations for this requirement is the desirability of using the average of multiple values of the variables n for evaluation and judgment purposes because of the associated increased accuracy. A suitable transformation from a specified to a model product has already been indicated by means of curves I and II shown in Figure 1. A similar transformation for a double specification limit is shown in Figure 4. The effect of using n values of the variable in the second case is suitably taken account of in the modified standard deviation $\nu_s X/\sqrt{n}$ that applies in this case.

Figure 4 shows that various judgment limits can now be defined for the magnitude of the variable. These are as follows:

1. x_a and x_a' are respectively the lower and upper acceptance limits below or above which not more than α_a percent of the population should fall, and it is intended that measured values of $\bar{x}_n$, which are greater than x_a or smaller than x_a', will represent completely acceptable products provided that the variability requirements have been satisfied;
2. x_r and x_r' are similarly the corresponding rejection limits respectively below or above which no more than α_r percent of the same population values should fall, and it is intended that measured values of $\bar{x}_n$, which are either smaller than x_r or greater than x_r', will be completely rejected; and
3. If the measured value of the magnitude falls within the ranges $x_r - x_a$ or $x_a' - x_r'$, the product will be conditionally accepted at reduced payment (6) provided that it complies with variability requirements.

Figure 1. Specified product for lower specification limit.

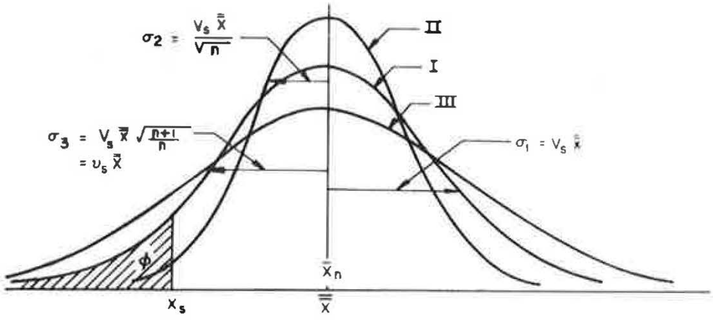


Figure 2. Specified product for double specification limits.

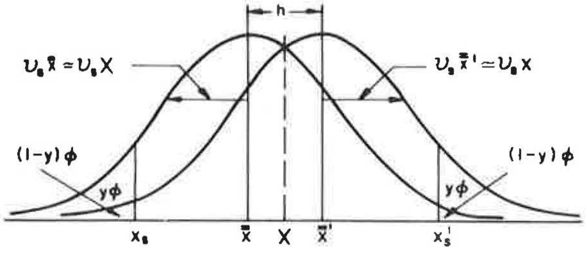


Figure 3. Factor y for different values of h and ϕ.

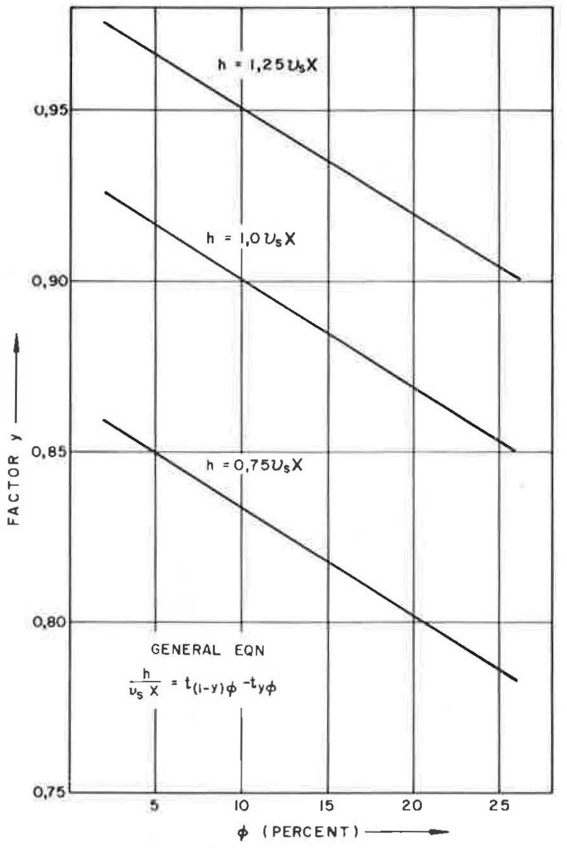
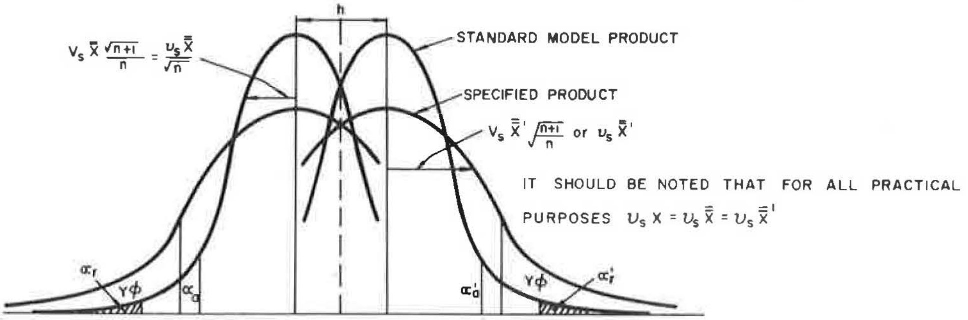


Figure 4. Specified and standard model products.



x_r should be small (assumed to be 0.1 percent) to ensure that rejected products have not been accepted. α_a, on the other hand, would normally be between 1 and 10 percent and can be optimized by taking into account certain economic factors (6, 12).

x_a', x_a', x_r, and x_r' can now be formulated as follows (Fig. 4):

$$\begin{aligned} x_a &= X\,[1 - (t_{\alpha_a}/\sqrt{n})\,\nu_s] - 0.5h \\ x_a' &= 2X - x_a \end{aligned} \tag{6}$$

$$\begin{aligned} x_r &= X\,[1 - (t_{\alpha_r}/\sqrt{n})\,\nu_s] - 0.5h \\ x_r' &= 2X - x_r \end{aligned} \tag{7}$$

Modified Model Product

The specified coefficient of variation ν_s is a maximum allowable (6) value that can be achieved by practically all producers. It is possible, on the other hand, that some producers can, as a matter of course, maintain a variability ν_p that is smaller than ν_s. In this case it is desirable that such a producer should be provided with some economic incentive. The modified model product is, therefore, defined with a variability ν_p, chosen by the producer and mandatory for quality-assurance purposes. The product can be formulated as follows (Fig. 5):

$$\begin{aligned} x_{ap} &= X\{1 - (1/\sqrt{n})\,[t_{\alpha_r}\nu_s - \nu_p\,(t_{\alpha_a} - t_{\alpha_r})]\} - 0.5h \\ &= x_r + X\,[(\nu_p/\sqrt{n})\,(t_{\alpha_r} - t_{\alpha_a})] \\ x_{ap}' &= 2X - x_{ap} \end{aligned} \tag{8}$$

Whereas $\bar{\bar{x}}' - \bar{\bar{x}} = h = \nu_s X$ is required and just sufficient for the standard model product, a situation that can be economically exploited by the producer exists in the case of the modified model product. The required value of h for the latter product is $\nu_p X$, and the available latitude is $\bar{\bar{x}}_p' - \bar{\bar{x}}_p$. This implies that the acceptable product mean of the modified model product can be as low as $X - (\bar{\bar{x}}_p + 0.5\nu_p X) = X(1 - 0.5\,\nu_p) - \bar{\bar{x}}_p$ or as high as $X(1 + 0.5\nu_p) + \bar{\bar{x}}_p$.

The difference between the required mean for the standard and modified model products represents a potential saving to the producer who can maintain ν_p instead of ν_s. Alternatively, the consumer may wish to receive a product with a mean value of X and a coefficient of variation ν_p. In this case the producer would have to be compensated for the potential saving mentioned earlier. Either way there is consequently an economic incentive for a producer to strive for a more uniform product.

Modified Model Product Resubmitted

If a product has been rejected because $\bar{x}_n < x_r$ or $\bar{x}_n > x_r'$ and is resubmitted for judgment, whether or not it has been improved, there is only an α_r^2 percent risk that $\bar{x}_n$ will be either lower than x_r or higher than x_r' (1, 3). To maintain the same judgment standard throughout requires a determination of the corresponding judgment limits that must apply to the second submission of a product to meet this requirement.

Figure 6 shows how this condition applies to a double specification limit. The relevant judgment limits for the magnitude of the variable for the modified model product, when the pooled information from the 2n tests for both submissions is used, can readily be formulated as follows (6, 7):

$${}^{(2)}x_{ap} = \{X[1 - (t_{\alpha_r}/\sqrt{n})\,(\nu_s - \nu_p)] - 0.5h\}\,(1 - k_1\nu_p) \tag{9}$$

and

$${}^{(2)}x_{ap}' = 2X - {}^{(2)}x_{ap} \tag{10}$$

Figure 5. Standard model and modified model products.

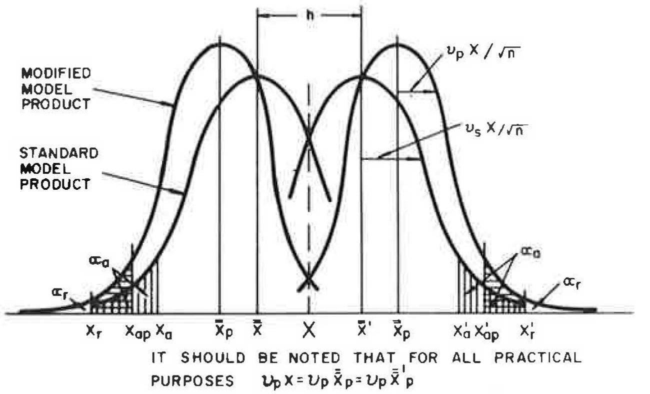


Figure 6. Control of magnitude for second submission.

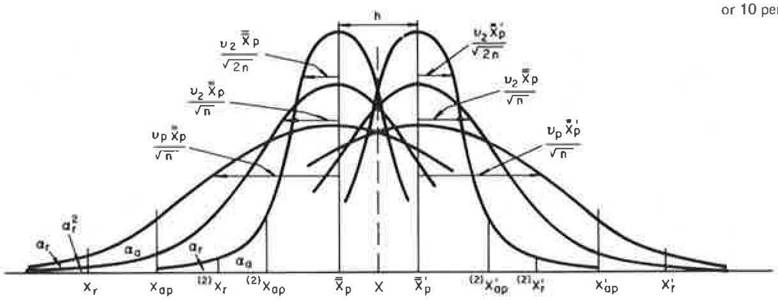


Figure 7. Relation between coefficient of variation ν_s and limit of accuracy p_r for various values of n.

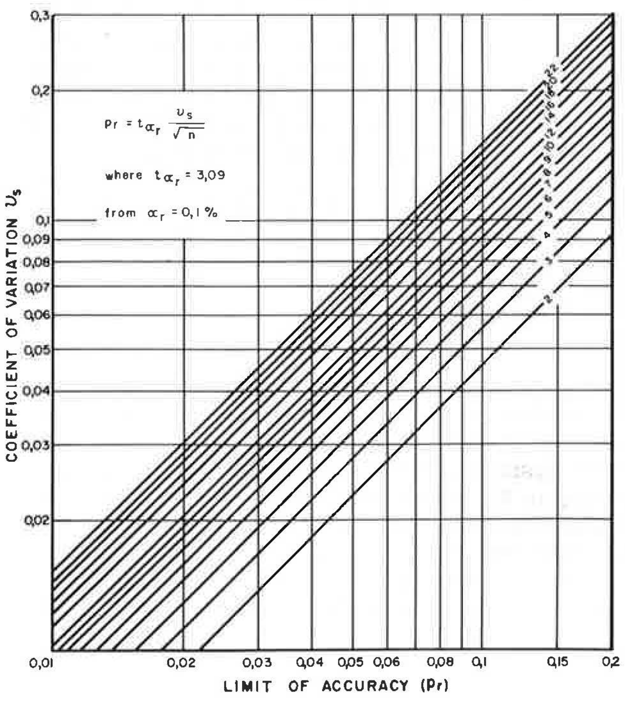


Table 1. Values of $k_{1,\alpha,n}$ for calculating rejection limit $^{(2)}x_r$ and acceptance limit $^{(2)}x_a$ for judgment of magnitude of product property resubmitted after rejection.

	$k_1 = t_{\alpha_r} t_{\alpha_a} / t_{\alpha_r}^2 \sqrt{2n}$ for α =			
n	0.1 percent	2.5 percent	5.0 percent	10 percent
2	1.004	0.637	0.535	0.417
3	0.820	0.520	0.437	0.340
4	0.710	0.451	0.378	0.295
5	0.635	0.403	0.338	0.264
6	0.580	0.368	0.309	0.241
7	0.537	0.341	0.286	0.223
8	0.502	0.318	0.267	0.208
9	0.474	0.300	0.252	0.196
10	0.449	0.285	0.239	0.186
12	0.410	0.260	0.218	0.170
14	0.380	0.241	0.202	0.158
16	0.355	0.225	0.189	0.147
18	0.335	0.212	0.178	0.139
20	0.318	0.201	0.169	0.132

Note: $^{(2)}x_r = \bar{\bar{x}}_p (1 - k_1 \upsilon_p)$ where $\alpha = \alpha_r$ = 0.1 percent; and $^{(2)}x_a = \bar{\bar{x}}_p (1 - k_1 \upsilon_p)$ where $\alpha = \alpha_a$ = 2.5, 5, or 10 percent.

where $k_1 = (t'_{\alpha_a} t'_{\alpha_r})/(t^2_{\alpha_r} \sqrt{2n})$.
Also

$$^{(2)}x_r = {}^{(2)}x_{ap} \tag{11}$$

as in Eq. 9 but with $\alpha_a = \alpha_r$, and

$$^{(2)}x'_r = 2X - {}^{(2)}x_r \tag{12}$$

Values for k_1 are given in Table 1.

The corresponding values for the standard model product can readily be determined from Eqs. 9 to 12 by substituting ν_s for ν_p.

The derivation of the judgment norms for the variability of the variable for double specification limits is identical to that for a lower specification limit and has already been published (13). This aspect will, therefore, not be dealt with here but is taken account of in the example illustrated in a following section.

DETERMINATION OF REQUIRED SAMPLE NUMBER

The required value of n for judgment purposes can be determined by at least 2 methods:

1. A method based on the relation given in Eq. 1 has been developed (8), in which both the cost of testing and the product cost are used, and has merit when reasonably reliable cost data are available from practice; and
2. A method that is perhaps more popular utilizes a relation that can be derived from Eq. 7 where, for a lower specification limit, $h = 0$ and $X = \bar{\bar{x}}$.

Furthermore, in the latter model, by putting $(\bar{\bar{x}} - x_r)/\bar{\bar{x}} = p_r$, where p_r is the limit of accuracy at the rejection limit for a standard model product, the relation between p_r and n can be formulated as follows:

$$p_r = t_{\alpha_r} (\nu_s/\sqrt{n}) \tag{13}$$

where t_{α_r} is the standard normal deviate for α_r. Figure 7 shows this relation for α'_r = 0.1 percent.

INFORMATION REQUIRED FOR APPLICATION OF QUALITY ASSURANCE

In addition to the availability of a rational method for the determination of judgment criteria, it is also essential to have various types of information available to ensure the effective use of quality assurance. This includes aspects such as the minimum yet practically achievable variability that should be specified; the percentage defect ϕ that should be allowed in the specification of properties of products; the economic lot size that should be used; and the various cost items related to testing, materials, construction, and maintenance. Although only limited systematic information is available with respect to most of these items, reasonably representative data have been established for the coefficients of variation representative of a number of product properties of importance in highway construction (6, 14).

Variability of Product Properties

Because variability is an important aspect of quality assurance, it is essential to use values that will ensure the best standard generally achievable by current practice. That was determined by establishing the distribution of coefficients of variation for various product properties from the analysis of extensive data from South African road practice. From these data, the median or V_{50} values of the coefficients of variation were determined as well as the ratio between this value and the 90 percentile value of the distribution or V_{90}. The average value for the ratio V_{90}/V_{50} was found to be 1.7.

More representative values of V_{50} were obtained by taking into account similar values determined from published information from practice in the United States (6). From these data, V_{90} values were again calculated by using the ratio of 1.7 given above. These V_{90} values have been chosen as the specified values V_s of the coefficient of variation to be used for quality-assurance purposes because they best comply with the qualifications stated above. This information is given in Table 2. It is intended that these data should be revised and updated from time to time as more reliable information becomes available. Useful information for the quantification of the variability for specifying grading for both bituminous surfacing and base course materials (6) is shown in Figure 8.

General Information for Quality Control

Although reliable values of certain parameters required in quality assurance are not yet available, approximate data obtained from a literature survey and an opinion survey of practicing engineers are given below as a guide.

1. The percentage defect ϕ varies between 10 and 25 percent, and the lower figures are associated with lower values of V_{90} and vice versa;
2. A value of $\alpha_r = 0.1$ percent is considered satisfactory for practical requirements although values of 0.2 percent or even higher may still be acceptable;
3. α_a should lie between 2.5 and 10 percent, and a preferable value for highway construction purposes is about 5 percent;
4. Depending on the applicable parameter, the limit of accuracy should preferably be below 15 percent and have a probable practical range of 6 to 12 percent; and
5. The rational determination of lot sizes is not yet possible, and currently accepted practice such as a day's work or estimated general lot sizes of about 4,000 m^2 should be used.

PRACTICAL APPLICATION OF QUALITY CONTROL

The effective utilization of quality assurance demands an integrated interaction among certain important functions such as the interest and activities of both producer and consumer as well as the nature and quantity of the available input information. Such a system is shown in Figure 9.

Consumer Discipline

Apart from the consideration, approval, and financing of the product property, the consumer shall be responsible for designing and specifying the desired product property as well as for the associated quality control to ensure that the delivered goods comply with the specified requirements, which at all times should be mutually acceptable to both consumer and producer and practically attainable.

Design Function—This function includes establishing and calculating norms required for control judgments and making them known to the producer discipline by means of the specifications. At the same time a simple scheme should be prepared for use by the application function for acceptance control. This function must also decide on details such as lot size; number of test samples; and test positions, procedures, methods, apparatus, and calibration. This function should constantly draw information from data storage that should be kept as up to date as possible. It is, therefore, essential that information gained by the producer should be fed into storage so that the design and specification function can recognize and readily allow for any new and improved techniques.

Application Function—It is the duty of this function to perform tests, make calculations, and execute judgments on product properties in accordance with the norms established by the design function.

Producer Discipline

This function includes storing and supplying performance and cost data and properly controling the process. The control of quality during the process of manufacture or construction can reduce costs by reducing rejections. The direct supply of test results by the producer to the design and application functions as well as to a central data

Table 2. Recommended V_{50} and V_{90} values for product properties.

		V_{50} Values (percent)			
Course	Property	South Africa	United States	Recom-mended	V_{90} $\equiv V_e$
Wearing and level-ing	Binder content	3.4	5.68	4.9	8.3
	Marshall stability	16.7	13.07	15.5	27.9
	Marshall flow	11.8	15.5	14.3	24.3
	Marshall void content	20.8	20.68	20.7	35.2
	Thickness		11.84	11.8	20.0
Subbase and base	Percentage density (general)	2.7	3.57	3.3	5.6
	Percentage density (asphalt)	1.75	1.25	1.6	2.7
	Thickness	6.4	6.8	6.7	11.4
	Moisture content		14.8	14.8	25.2
	Cement content (stabilization)		13.6	13.6	23.2
Concrete pave-ment	Thickness				
	8 in.		3.6	3.6	6.1
	9 in.		3.2	3.2	5.4
	10 in.		2.6	2.6	4.4
	Strength, 28 days		14.5	14.5	24.6
	Air void content (plastic sheeting)		18.34	18.3	31.1
	Cone slump		31.5	31.5	53.5

Figure 8. Relation between V_{90} and cumulative percentage passing sieve size.

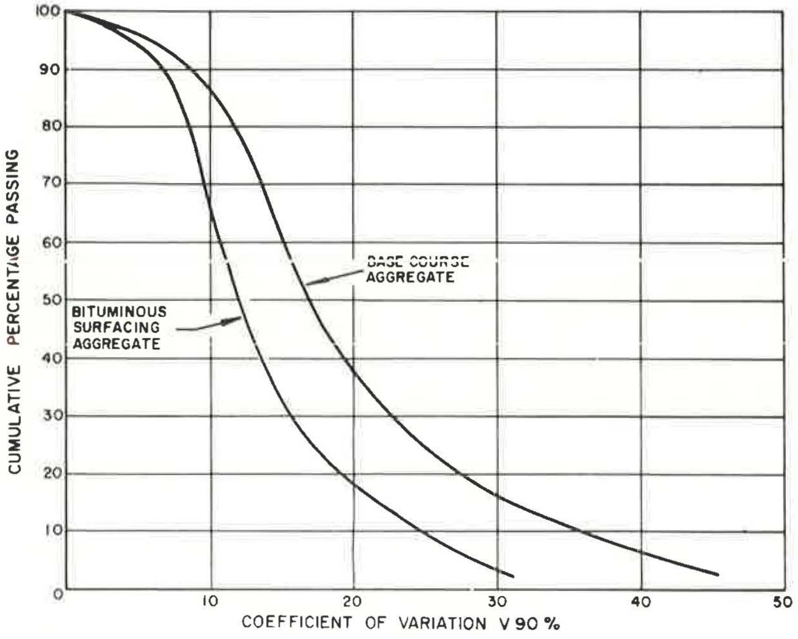
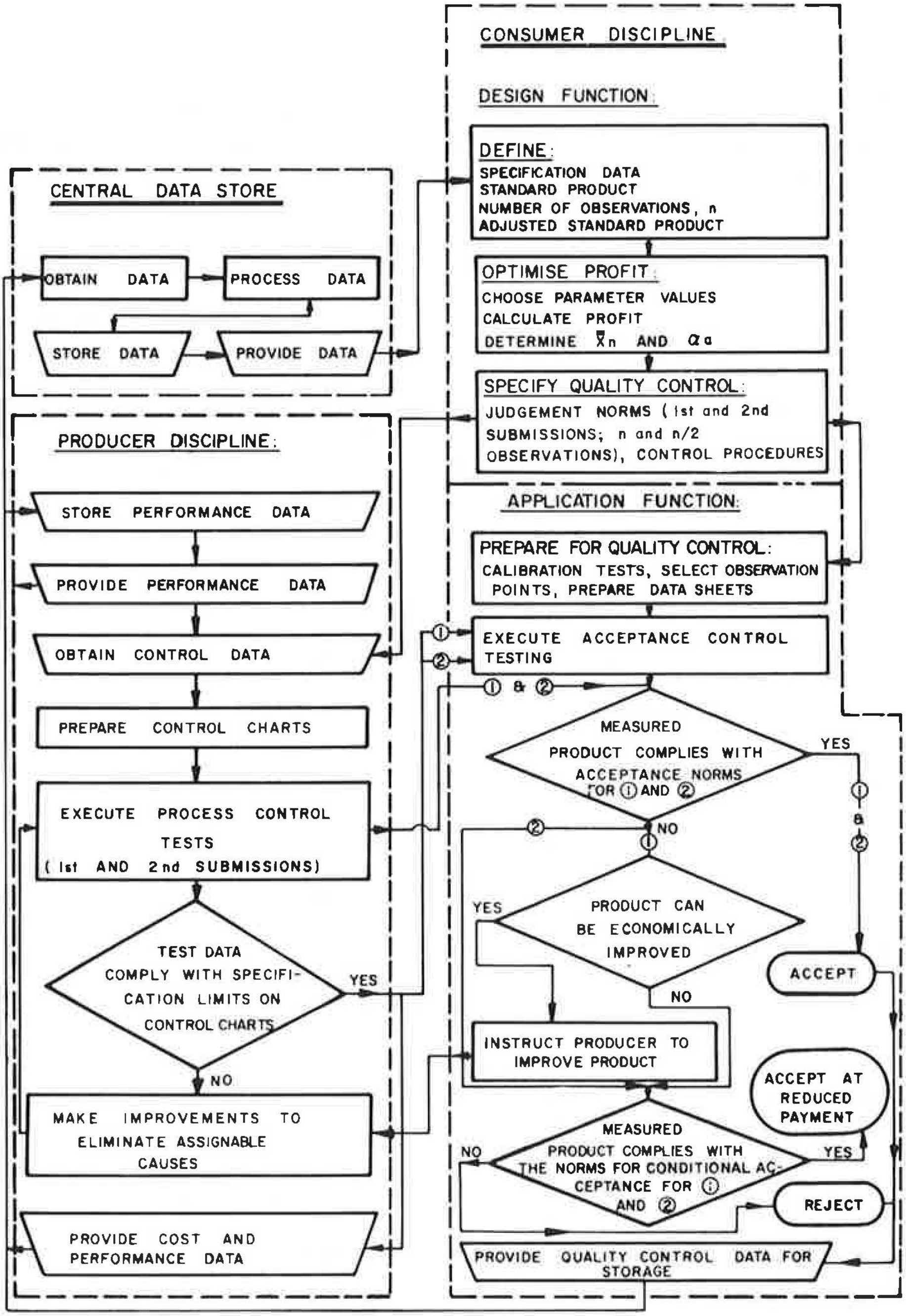


Figure 9. Quality control system for road construction.



NOTE ①, ② RESPECTIVELY 1st AND 2nd SUBMISSIONS storage will contribute largely to continually improved judgment discipline and to design and specification techniques.

Central Data Storage

Some form of central data storage and processing of production costs and quality variations from which the latest parameters could be drawn would be of benefit to the producer discipline and would help to bring about improved cooperation between design and construction disciplines.

PRACTICAL EXAMPLE

The following example of a double limit specification illustrates the practical application of the method.

It is desired to control binder content in a pre-mix to an average of 5 percent by mass of total mix. The choice of parameters for specification requirements is as follows.

1. Average magnitude $X = 5$ percent $= 0.05$;
2. Coefficient of variation $= V_s = V_{90} = 0.083$ (Table 2);
3. Spread of higher and lower population means, i.e., $\bar{\bar{x}}' - \bar{\bar{x}} = h = \nu_s X$, say;
4. Values of ϕ, α_a, α_r, and p_r = 15, 5, 0.1, and 10 percent; and
5. Lot = 1 day's output, say.

The magnitude of the variable is calculated as follows:

1. Number of tests per lot—

$$\sqrt{n + 1/n} = p_r/t_{\alpha_r}\nu_s = 0.10/(3.10 \times 0.083)$$

from which $n = 7.4$ or taken as 8, $p_r = 0.096$ or 9.6 percent, and $\nu_s = \sqrt{[(n+1)/n]}\ V_s = 0.088$.

2. Specification limits—From the data shown in Figure 3, ($h = \nu X$ for $\phi = 15$ and $y = 0.883$), $y\phi = 13.25$ and $t_{y\phi} = 1.115$.

$$\bar{\bar{x}} = X(1 - 0.5\nu_s) = 0.05(1 - 0.5 \times 0.088) = 0.0478$$

$$\bar{\bar{x}}' = 2X - \bar{\bar{x}} = 0.10 - 0.0478 = 0.0522$$

$$x_s = X\left[1 - \nu_s(0.5 + t_{y\phi})\right] = 0.05\left[1 - 0.088(0.5 + 1.115)\right] = 0.0430 \text{ (if required)}$$

$$x_s' = 2X - x_s = 0.10 - 0.043 = 0.0570 \text{ (if required)}$$

This implies that not more than 15 percent of the values of binder content observations shall fall outside the limits of 4.33 and 5.67 percent by mass and that the product property variation shall not exceed $\nu_s = 8.8$ percent.

3. Rejection limits (first submission)—

$$x_r = X(1 - p_r) - 0.5h = 0.05 \times 0.904 - 0.5 \times 0.88 \times 0.05 = 0.0431$$

$$x_r' = 2X - x_r = 0.10 - 0.0431 = 0.0569$$

4. Acceptance limit (first submission)—

$$x_a = X\left\{1 - \nu_s\left[(t_{\alpha_a}/\sqrt{n}) + 0.5\right]\right\} = 0.05\left\{1 - 0.088\left[(1.645/\sqrt{8}) + 0.5\right]\right\} = 0.0455$$

$$x_a' = 2X - x_a = 1.00 - 0.0455 = 0.0545$$

5. Rejection limits (second submission)—

$${}^{(2)}x_r = X(1 - 0.5\nu_s)(1 - k_1\nu_s)$$

Table 3. Factor f_3 values for varying percentages of ϕ and α required for calculating control limits for range variability.

n	α (percent) 0.1	2.5	5.0	10	50	α (percent) 0.1	2.5	5.0	10	50
	ϕ = 10 percent					ϕ = 15 percent				
2	2.827	1.927	1.684	1.416	0.578	3.216	2.194	1.916	1.612	0.657
3	3.335	2.425	2.181	1.924	1.048	3.695	2.686	2.417	2.132	1.161
4	3.679	2.758	2.515	2.258	1.372	3.991	2.993	2.728	2.450	1.488
5	3.930	3.011	2.768	2.503	1.621	4.244	3.251	2.989	2.703	1.750
6	4.135	3.209	2.965	2.700	1.817	4.429	3.436	3.176	2.892	1.947
7	4.302	3.372	3.131	2.868	1.990	4.588	3.596	3.339	3.058	2.122
8	4.442	3.517	3.274	3.007	2.129	4.725	3.743	3.483	3.199	2.265
9	4.565	3.638	3.397	3.126	2.259	4.838	3.854	3.599	3.312	2.394
10	4.674	3.751	3.500	3.233	2.364	4.939	3.961	3.698	3.417	2.498
12	4.860	3.926	3.687	3.415	2.561	5.111	4.128	3.877	3.592	2.694
14	5.014	4.082	3.840	3.572	2.722	5.260	4.284	4.028	3.748	2.855
16	5.150	4.215	3.977	3.706	2.854	5.378	4.400	4.154	3.871	2.980
18	5.261	4.322	4.084	3.819	2.974	5.495	4.515	4.266	3.989	3.107
20	5.357	4.431	4.187	3.920	3.084	5.577	4.611	4.359	4.080	3.210
	ϕ = 20 percent					ϕ = 25 percent				
2	3.629	2.473	2.162	1.818	0.741	4.043	2.755	2.408	2.026	0.826
3	3.988	2.900	2.609	2.302	1.253	4.297	3.124	2.811	2.480	1.350
4	4.269	3.200	2.918	2.621	1.592	4.538	3.403	3.102	2.786	1.692
5	4.479	3.431	3.155	2.852	1.847	4.723	3.620	3.327	3.008	1.948
6	4.655	3.610	3.338	3.040	2.046	4.882	3.789	3.501	3.188	2.146
7	4.798	3.758	3.492	3.199	2.219	5.012	3.929	3.648	3.342	2.318
8	4.918	3.895	3.625	3.329	2.358	5.122	3.957	3.776	3.468	2.456
9	5.025	4.004	3.739	3.441	2.487	5.220	4.160	3.884	3.575	2.584
10	5.119	4.105	3.833	3.541	2.589	5.307	4.258	3.974	3.671	2.685
12	5.281	4.266	4.006	3.711	2.783	5.457	4.408	4.140	3.835	2.876
14	5.415	4.410	4.147	3.858	2.940	5.582	4.546	4.275	3.977	3.030
16	5.535	4.528	4.274	3.984	3.067	5.694	4.662	4.398	4.098	3.155
18	5.631	4.630	4.372	4.088	3.184	5.784	4.755	4.491	4.199	3.270
20	5.715	4.728	4.467	4.182	3.290	5.862	4.850	4.582	4.289	3.375

Note: $\bar{\bar{R}}$, R_a, or R_r = $f_1 R_{\alpha,n}\sigma_s = f_3\sigma_s$, where α = 0.1 and 50 percent respectively for R_r and $\bar{\bar{R}}$ and $\alpha = \alpha_a$ = 2.5, 5.0, and 10 percent for R_a.

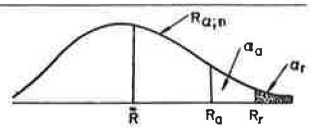


Table 4. Factor k_3 values for varying percentages of ϕ and α required for calculating acceptance limit R''_a and rejection limit R''_r at resubmission for range variability.

n	α (percent) 0.1	2.5	5.0	10	α (percent) 0.1	2.5	5.0	10
	ϕ = 10 percent				ϕ = 15 percent			
2	2.026	1.518	1.384	1.233	2.339	1.753	1.599	1.427
3	2.502	1.942	1.795	1.629	2.769	2.148	1.986	1.803
4	2.839	2.249	2.093	1.916	3.086	2.445	2.275	2.084
5	3.100	2.487	2.321	2.145	3.336	2.676	2.497	2.307
6	3.314	2.677	2.514	2.335	3.540	2.861	2.686	2.494
7	3.494	2.844	2.676	2.490	3.715	3.024	2.844	2.646
8	3.654	2.990	2.822	2.629	3.866	3.164	2.986	2.782
9	3.788	3.114	2.941	2.750	3.996	3.285	3.102	2.901
10	3.907	3.231	3.054	2.859	4.110	3.399	3.212	3.007
12	4.111	3.424	3.246	3.051	4.306	3.585	3.400	3.195
14	4.287	3.592	3.410	3.216	4.475	3.749	3.560	3.356
16	4.441	3.736	3.550	3.350	4.622	3.889	3.695	3.488
18	4.575	3.863	3.674	3.471	4.752	4.013	3.816	3.605
20	4.694	3.970	3.791	3.583	4.865	4.115	3.929	3.714
	ϕ = 20 percent				ϕ = 25 percent			
2	2.600	1.949	1.777	1.587	2.896	2.171	1.980	1.767
3	2.993	2.322	2.146	1.949	3.224	2.502	2.312	2.100
4	3.295	2.610	2.456	2.225	3.502	2.774	2.581	2.365
5	3.533	2.834	2.654	2.444	3.726	2.989	2.790	2.578
6	3.730	3.014	2.829	2.628	3.912	3.161	2.968	2.756
7	3.897	3.173	2.984	2.777	4.072	3.315	3.118	2.901
8	4.044	3.310	3.124	2.911	4.213	3.448	3.254	3.032
9	4.170	3.428	3.237	3.027	4.332	3.561	3.363	3.145
10	4.280	3.538	3.344	3.131	3.437	3.669	3.468	3.247
12	4.468	3.720	3.528	3.315	4.617	3.844	3.646	3.426
14	4.631	3.880	3.684	3.473	4.772	3.999	3.797	3.579
16	4.773	4.015	3.815	3.600	4.910	4.131	3.925	3.704
18	4.897	4.136	3.933	3.715	5.030	4.248	4.039	3.816
20	5.007	4.235	4.044	3.824	5.136	4.344	4.148	3.922

Note: $R''_a = k_3 \upsilon_s \bar{\bar{x}}$ ($\alpha = \alpha_a$ = 2.5, 5.0, and 10 percent), and $R''_r = k_3 \upsilon_s \bar{\bar{x}}$ ($\alpha = \alpha_r$ = 0.1 percent), where $k_3 = R_{\alpha,(2n-1)}\sqrt{\left[(n-1)\chi^2_{\alpha_r,(n-1)}\right] / \left[\chi^2_{\phi,(n-1)}\,\chi^2_{\alpha_r^2,(n-1)}\right]}$.
For $R_{\alpha,(2n-1)}$, values are given in standard tables for the distribution of the range for corresponding values of α and $(2n - 1)$.

Figure 10. Data for acceptance control.

CONTRACT, TERRAIN, PRODUCT AND OTHER IDENTIFICATION

DATA OF THE LOT

CONTROL NORMS

PRODUCT CHARACTERISTIC: BINDER CONTENT

NUMBER OF TESTS (n) = 8

FIRST SUBMISSION		RESUBMISSION
	MEAN	
(1) $0{,}0455 < \bar{X}_n < 0{,}0545$	ACCEPT	$0{,}0469 < \bar{X}^{*}_{2n} < 0{,}0531$ (1)
(2) $0{,}0431 < \bar{X}_n < 0{,}0455$ OR $0{,}0545 < \bar{X}_n < 0{,}0569$	ACCEPT CONDITION-ALLY	$0{,}0459 < \bar{X}_{2n} < 0{,}0469$ OR (2) $0{,}0531 < \bar{X}_{2n} < 0{,}0541$
(3) $\bar{X}_n < 0{,}0431$ OR $\bar{X}_n > 0{,}0569$	REJECT	$\bar{X}_{2n} < 0{,}0459$ OR $\bar{X}_{2n} > 0{,}0541$
	VARIABILITY	
(1) $R_n < 0{,}0142$	ACCEPT	$R^{*}_{2n} < 0{,}0122$ (1)
(2) $0{,}0192 > R_n > 0{,}0142$	ACCEPT CONDITIONALLY	$0{,}0157 > R_{2n} > 0{,}0122$ (2)
(3) $R_n > 0{,}0192$	REJECT	$R_{2n} > 0{,}0157$ (3)

* BASED ON n VALUES EACH OF 1st AND 2nd SUBMISSION

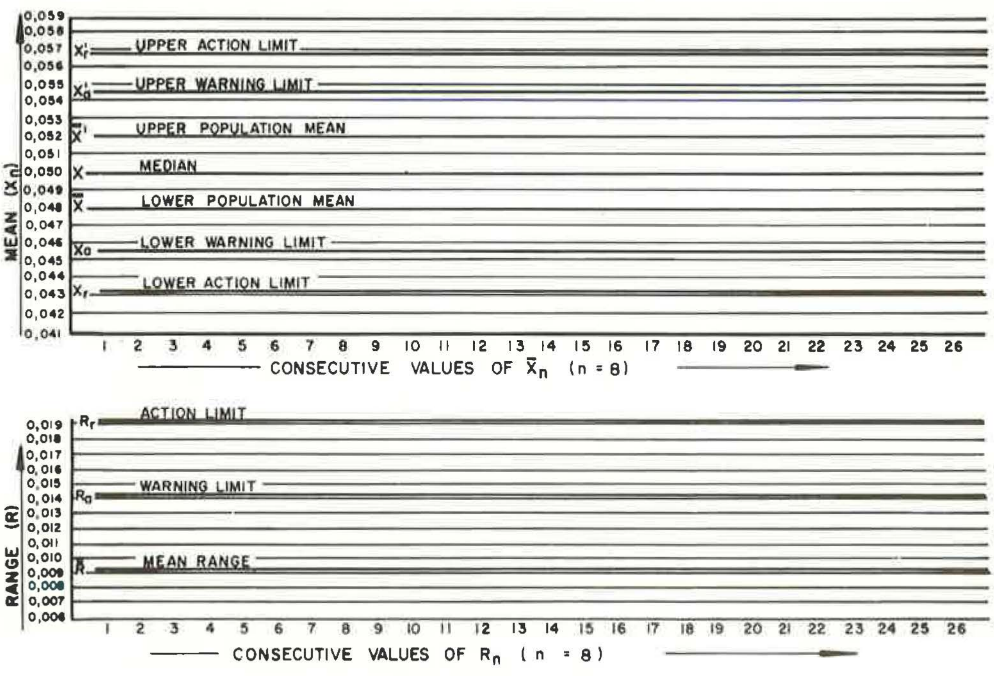
CONTROL TESTING

FIRST SUBMISSION			SECOND SUBMISSION		
TEST Nº	X_n	a) X_n (max) & b) X_n (min)	TEST Nº	X_n	a) X_{2n} (max) & b) X_{2n} (min)
1					
2					
3		a):			a)
4		b):			b):
5					
6					
7					
8					
	$\bar{X}_n$:	R_n		$\bar{X}^{*}_{2n}$:	R^{*}_{2n}

JUDGEMENT RESULT

ACCEPT	
CONDITION ACCEPT	
REJECT	

Figure 11. Control charts for magnitude and variability of binder content.



$$^{(2)}x_r = 2X - {}^{(2)}x_r$$

From data given in Table 1 ($\alpha_r = 0.1$ percent and $n = 8$), $k_2 = 0.502$; therefore,

$$^{(2)}x_r = 0.05\ [1 - (0.5 \times 0.088)]\ [1 - (0.502 \times 0.088)] = 0.0459$$

$$^{(2)}x_r = 0.100 - 0.0459 = 0.0541$$

6. Acceptance limits (second submission)—

$$^{(2)}x_a = X(1 - 0.5\nu_s)\ (1 - k_1\nu_s)$$

$$^{(2)}x_a = 2X - {}^{(2)}x_a$$

From data given in Table 1 ($\alpha_a = 5$ percent and $n = 8$), $k_1 = 0.267$; therefore,

$$^{(2)}x_a = 0.05\ [1 - (0.5 \times 0.088)]\ [1 - (0.267 \times 0.083)] = 0.0469$$

$$^{(2)}x_a = 0.10 - 0.0969 = 0.0531$$

For the variability of the variable, the range is selected to represent variability rather than the standard deviation for practical reasons. The general expression for range is given by $R_\alpha = f_3\bar{\bar{X}}_p\nu_p$ (where f_3 may be interpolated from data given in Table 3) for the mean range $\bar{\bar{R}}_p$, the acceptance limit R_{ap}, and the rejection limit R_r (6, 7, 13). $^{(2)}R_\alpha = k_3\bar{\bar{X}}_p\nu_p$ (where k may be interpolated from data given in Table 4) for $^{(2)}R_{ap}$ and $^{(2)}R_r$. For the example, the value of $\bar{\bar{R}}_p$ can be calculated as follows: $\bar{\bar{R}}_p = f_3\bar{\bar{X}}_p\nu_p$, where $\nu_p = 0.088$, f_3 is the value for $\phi = 13.25$ and $\alpha = 50$, and $\bar{\bar{X}}_p$ is the mean of the product value as specified and $\alpha = 0.05$. Therefore,

$$\bar{\bar{R}}_p = 2.218 \times 0.05 \times 0.083 = 0.00920$$

Similarly, for the first submission,

$$R_{ap} = 0.0142\ (f_3 = 3.410 \text{ for } \alpha = 5 \text{ percent and } n = 8)$$

$$R_r = 0.0192\ (f_3 = 4.626 \text{ for } \alpha = 0.1 \text{ percent and } n = 8)$$

and for the second submission,

$$^{(2)}R_{ap} = 0.0122\ (\text{for } k_3 = 2.929 \text{ for } \alpha = 5 \text{ percent and } n = 8)$$

$$^{(2)}R_r = 0.0157\ (\text{for } k_3 = 3.792 \text{ for } \alpha = 0.1 \text{ percent and } n = 8)$$

From the data given above, an acceptance control sheet (Fig. 10) may now be prepared by the design function for use by the application function. From time to time revision of this sheet may be called for to allow for the modified product coefficient of variation ν_p in place of ν_p if the quality of the product property merits this action. Control charts as shown in Figure 11 can be used to plot information required to exercise process control. Although in this example $n = 8$ has been used for convenience, a value of n used for process control is normally lower than that used for acceptance control.

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DISCUSSION

R. L. Davis, Koppers Company, Inc., Verona, Pennsylvania

The authors of this paper are to be congratulated on exploring in such detail the acceptance and rejection limits for both first and second submission of a product. In discussing the number of observations or tests, the authors mention a method based on optimization of cost and then go on to develop an alternative or popular method based on a standard normal deviation. I would have much preferred that they pursue the method of economic optimization of cost because this is usually the primary objective of quality assurance plans. There are probably instances where safety and legal requirements are of overriding importance, but in most instances a quality assurance plan is to protect the buyer from the economic consequences of a poor quality product. Therefore, the problem is best approached by balancing the cost of the quality assurance program against the savings that will be realized through the level of quality assured by this program.

Intrinsic to the problem of economic optimization in rational quality assurance is to decide whether to use acceptance sampling alone or a combination of quality control and acceptance sampling. The amount of acceptance sampling necessary for a certain level of quality assurance is related to the information coming from the quality control program. Where the buyer is intimately familiar with the quality control program, he often can judge the level of quality assurance with little or no acceptance testing. The buyer pays for both the quality control program and the acceptance sampling plan, and he should not overlook the benefits that can come from the proper use of both.

In my opinion, the quality of some products of the highway industry can best be ensured through the buyer's participation in the quality control program. There has been a drive in recent years to remove the buyer from the quality control of all highway products to allow the seller to make full use of his ingenuity in improving the product and reducing the costs. There are instances where the buyer has contributed to higher costs by being too restrictive, but there are other areas where the best approach to quality assurance involves the buyer in the quality control of the product.

One of these is in the acceptance of the finished roadway. The important thing is to build it right in the first place. A road that is poorly constructed can seldom be successfully corrected afterwards. Penalties (which in the long run are paid by the buyer) will not correct fundamental errors in construction. Therefore, the major effort of all concerned should be the proper control of quality in the first place. Few highway engineers have any confidence in their ability to judge the useful life of a road by merely viewing the finished pavement. Most think that it is necessary for them to be involved in the quality control program to have assurance of the quality of the finished roadway.

The buyer should approach penalties with the realization that in the long run he will pay them as he will all the costs of the products he buys. This does not mean that a system of penalties may not be a good investment for a buyer, but he should look at what he is buying with his money and determine whether he is getting a proper return for his money.

I would much prefer that the authors use standard deviations rather than coefficients of variation in describing the variations of the various properties described in their paper. This is a personal observation, and I am not sure that all engineers would share my views on the greater simplicity of the use of standard deviations.

I enjoyed reading this paper and would be interested in reports on the application of this approach to acceptance and rejection of road materials and construction.

AUTHORS' CLOSURE

The authors wish to thank Mr. Davis for his constructive comments.

We agree that the determination of the optimum sample size based on economic considerations is desirable. This approach, together with other aspects concerning the choice of other parameters such as α_a to ensure maximum economic gain, has in fact been developed and published elsewhere (6, 15).

Either the standard deviation σ or the coefficient of variation V can be used to describe variability, and the theory presented in the paper is, with the proper adaptation, applicable to both cases. In the first case σ is independent of the mean $\bar{\bar{x}}$, while in the second case σ must be proportional to $\bar{\bar{x}}$. According to information analyzed by the authors as well as independently substantiated (16), the second case is more applicable to practical conditions, and V instead of σ was, therefore, chosen to represent variability.

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