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Foreword

The reports you will find in this RECORD will contribute new information of value to you if you are an engineer, contractor or supplier of materials or equipment, with interests in the construction of bituminous pavements. In addition to findings from the American scene one report in particular provides some insight into the more advanced paving practices in Europe.

Research, in the field of construction practices for highways, is proving to be a great revealer, particularly as to actual results on the larger undertakings, using current improvements in materials and newly developed types of equipment. The area of bituminous pavement operations is no exception. Pavement research, however, is different from much other research in that, broadly speaking, it does not lend itself to test-tube type of study. In construction practices, no one can argue with what is shown by the results of full-scale tests. This makes it easier to get official adoption of revisions for additional specifications and construction requirements, many of which may have been handed down for half a century.

In the first report, "High Temperature Pneumatic Compaction," it is shown that the use of a release agent in the spray water used to prevent sticking, permits pneumatic-tire rollers to be used very effectively for breakdown rolling with pavement temperatures as high as 255 F and in some instances 275 F. The method results in an improved pavement density and structure which appears to be independent of the tire pressure. It works equally well for intermediate pneumatic compaction. However, it does not work well in a team that includes a steel tire roller used to iron out grooves in the surface.

For a long time highway engineers have been concerned about hardening developed by petroleum asphalts in hot mixes between the time of mixing and placement and also the effects of such hardening on the durability of the pavement. The exhaustive studies included and analyzed in the second report contribute one more milestone in knowledge of this complicated process. It traces the hardening through mixing and hauling into the early life of the pavement and while the conclusions offer no quantitative limitation on hauling time, they do recommend prudence in such limitation and the protection of the hot material en route.

"A Program for Smoother Roads" sets forth in detail the development of mechanics for control of bituminous resurfacing operations. Every highway department has at one time or other, been faced with the problem of rebuilding deteriorated wearing surfaces on both bituminous and concrete highways. With unlimited funds it is not a difficult task but in an effort to restore a like-new profile and crown it often happens that the quantities and cost get out of hand. Using, on a statewide basis, a minimum cover and average cross-sectional area system geared to computers, the methods reported enable the designer to economize, retain control and still produce a smooth new riding surface with imperceptible undulations.

The fourth report is by a Swiss engineer and covers the European application of the automatic screed in controlling bituminous pavement surfaces. The very exacting specifications to be met are presented and analyzed together with the high quality and durability of the surfaces obtained by use of the screeds. The report, amply supplemented with pictures, conveys to the reader a message of deliberate thoroughness aimed at long life for the highway. Incidentally, in this connection the author is employed by the contractor and in Switzerland the contractor is required to guarantee the pavement for its first five years of service.

The final report covers research with the great outdoors as the laboratory. It is not theoretical, but as written by an engineer in a top-management position, it is highly thought provoking. It shows that semiannual condition surveys find flaws in highways that might otherwise be overlooked. The theme is that whether attributable to design or construction the same deficiencies should not be repeated time after time. The condition survey is the tool for evaluation.

Contents

High Temperature Pneumatic Compaction

R. J. SCHMIDT and L. E. SANTUCCI, Chevron Research Co., Richmond, Calif.; and W. A. GARRISON, Contra Costa County Public Works Dept., Martinez, Calif.

A field test is described which compares the core densities and air permeabilities obtained on asphalt concrete (AC) when compacted by several rolling sequences. The different procedures used include steel wheel rolling alone and in combination with pneumatic compaction made under a variety of test conditions. The conditions include both low and high pressure tires, moderate and high pavement compaction temperatures, and the use of pneumatic rollers for intermediate or breakdown compaction. Pneumatic compaction was conducted at pavement temperatures as high as 275 F without preheating the tires by means of a release agent added to the water lubricating the tires. Pavement densities as high as 95 percent of laboratory compacted values were obtained at 195-220 F. Values less than 90 percent were obtained by pneumatic compaction at 135 F, the highest temperature possible to roll without the AC sticking to water-lubricated tires.

•THE DEGREE of compaction of an asphalt concrete (AC) pavement is possibly the most important factor determining its properties and durability. Compaction is paramount in controlling the stability (i.e., resistance to plastic deformation) and toughness (1) of a well-designed AC mix. It also controls the durability of the pavement because it can significantly influence the permeability and void content which can limit the weathering rate of the asphalt binder. The fatigue resistance of the AC also appears to be related to the degree of compaction (2).

The stability, as measured by the Marshall or Hveem test, can vary as much from a poorly compacted to a well-compacted mix as it varies when the aggregate system is changed from gravel to crushed aggregate. A well-compacted crushed quartzite may have a Hveem stability of 55 and uncrushed gravel, 35 (3). Yet, the same crushed quartzite AC mix can vary in Hveem stability from 37 to 55 if its compaction is increased from 140 to 146 pcf by more thorough compaction of the sample tested (3). Similarly, pavements which are not thoroughly compacted are soft or tender immediately after construction (1). They are easily damaged by traffic or by such abuses as ladies' high-heeled shoes. High permeability pavements, which may result from poor compaction, are most conducive to rapid weathering of the asphalt binder (4). The severity of all three of these shortcomings can be reduced by proper compaction of the AC during construction.

A high degree of compaction is not readily achieved during construction. Additional compaction cannot always be obtained by using a greater number of roller passes or by using a heavier steel-wheeled roller (5, 6). If the mix is too unstable or the roller is too heavy, excessive rolling may actually decompact the mix. Where very thorough compaction is required, rubber-tired pneumatic compactors are used between breakdown and finish steel wheel rolling. The most effective pneumatic compaction is obtained at high temperatures where the viscosity of the asphalt binder is low. However,

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the temperature at which these pneumatic compactors can be used is limited by the point at which the AC begins to stick to the tires.

Very high degrees of pneumatic compaction have been obtained by Rensel (7) who was able to roll at high temperatures (275-300 F) without sticking provided that the tires were preheated. Calman (8) has tried, with limited success, to use various materials on the tires to prevent sticking.

This paper describes the degree of compaction obtained on a field study comparing several commonly used compaction procedures. Included are high temperature pneumatic compaction studies made possible by means of a release agent which prevents AC from sticking to cold pneumatic compactor tires at high-pavement temperatures (250-275 F).

DESIGN OF FIELD TEST

This field comparison of compaction procedures was made on a paving project in Moraga, California. Paving was done under the direction of the Contra Costa County Public Works Department and constructed by the O. C. Jones Paving Co.

The grading of the crushed gravel used (Fig. 1) complied with the California Division of Highways' ¾-in. maximum type B mix. A 6.2 percent design asphalt content was selected for the mix. This particular gradation and asphalt content were chosen because the mix had the potential of densifying under sufficient compaction to a low voids content (2 percent) and, because at high pavement temperatures, with the slightly rich asphalt content selected, the mix would have the greatest tendency to stick to the pneumatic compactor tires.

Seven different rolling sequences were compared. In one of these sequences, steel-wheel rolling was used. In another, breakdown was accomplished with a pneumatic

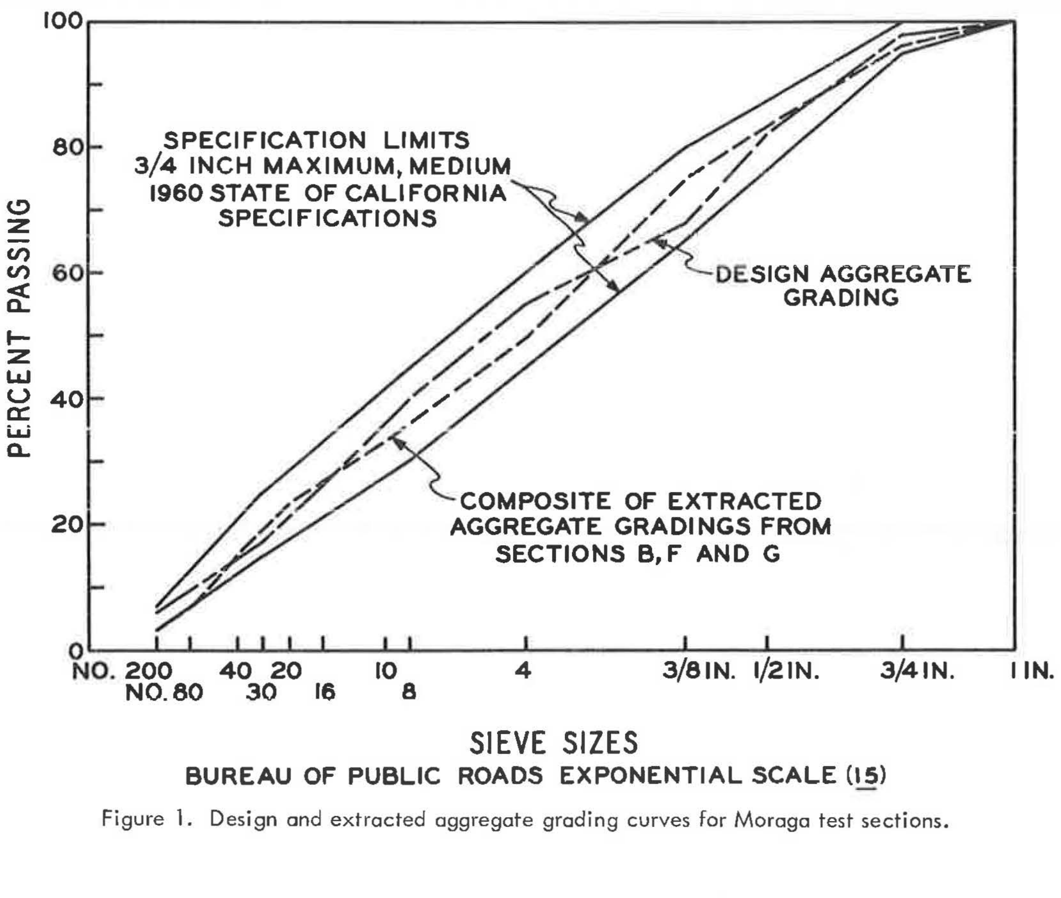

Figure 1. Design and extracted aggregate grading curves for Moraga test sections.

TABLE 1

SUMMARY OF LABORATORY AND FIELD RESULTS ON MORAGA TEST SECTIONS

Section	Compaction Procedure[a]	Pneumatic Tire Pressure (psi)	Mix Temperature During Rolling, F			Average Density of Pavement Cores (pcf)	Hveem Laboratory Density (pcf)	Relative Compaction (%)	Maximum Theoretical Density of Mix (pcf)	Extracted Asphalt Content From Cores (%)	Air Voids of Pavement Cores (%)	Air Flow Rates in Place (ml/min)
			Breakdown	Pneumatic	Finish							
A	12-ton steel		280-285			136.4	150.4	90.7	154.3	5.2	11.6	1205
	8-ton steel finish				145-150							
B	12-ton steel		215-225			132.6	147.9	89.7	153.8	4.7	13.8	1230
	5-ton pneumatic	40		110-135								
	8-ton steel finish				100-120							
C	12-ton steel		210-220			143.1	151.6	94.4	153.5	5.0	6.8	65
	5-ton pneumatic[b]	40		195-220								
	8-ton steel finish				175-195							
D	12-ton steel		245-275			141.2	151.0	93.5	153.3	5.6	7.9	200
	5-ton pneumatic[b]	40		240-250								
	8-ton steel finish				140-150							
E	12-ton steel		250-270			141.4	149.8	94.4	153.6	5.5	7.9	260
	5-ton pneumatic[b]	90		210-235								
	8-ton steel finish				145-155							
F	12-ton steel		250-270			141.4	149.8	94.4	153.3	5.5	7.8	135
	5-ton pneumatic[b]	90		170-235								
	8-ton steel finish				145-155							
G	5-ton pneumatic[b]	40	205-230			135.8	151.0	89.9	152.8	5.3	11.1	770
	8-ton steel finish				140-160							

[a]Refer to Table 2 for more complete description of compaction equipment.

[b]Three percent Chevron rollereeze added to roller water.

compactor. In still others, intermediate pneumatic compaction was conducted at a variety of pavement temperatures and tire pressures.

DESCRIPTION OF FIELD TEST

The final pavement thickness was approximately 2 in. Sections B through F were laid on March 25, 1965, during a period of bright sunlight. During this time the ambient temperature was approximately 75 F. Sections A and G were laid on March 29, 1965, a day which was cloudy and overcast with an ambient temperature of approximately 55 F.

In all but one case (section G) where pneumatic compaction was used for breakdown rolling, a 12-ton tandem steel-wheel roller was used on the breakdown roll. In most cases, breakdown was followed by pneumatic compaction of some type. All sections were finish rolled with an 8-ton tandem roller. All rolling sequences are indicated in Table 1, and the equipment used is indicated in Table 2.

Section A

All Steel Wheel Rolling. Steel-wheel breakdown rolling at 280-285 F,[a] followed by finish rolling at 150 F, resulted in an in-place density of 90.7 percent relative compaction (percent of the laboratory compaction obtained by the Hveem procedure on recompacting the cores taken from this section for density measurement). This value is typical of the compaction obtained under these conditions.

Section B

Intermediate Pneumatic 5-Ton Compaction, Four Coverages with 40-psi Tires at Low Pavement Temperatures. This intermediate pneumatic compaction was conducted at the highest temperature possible (130-135 F) without the AC pavement sticking to water-lubricated tires. Each spot on the pavement was rolled four times by one of the tires on the pneumatic compactor. A relative density of 89.7 percent was obtained. This value is not significantly greater than that obtained with steel rolling only. A limited amount of intermediate pneumatic compaction made at low pavement temperatures with a lightweight low tire pressure compactor does not appear to be of any value. When rolled at pavement temperatures greater than 160 F, the AC rolled up on the tires (Fig. 2).

Section C

Intermediate Pneumatic 5-Ton Compaction, Four Coverages with 40-psi Tires at High Pavement Temperatures. Section C is similar to section B except that the pneumatic compaction was conducted at high temperatures (195-220 F). The tires were lubricated with water containing 3 percent of a release agent. While using this material in the water, no sticking of this mix to the pneumatic tires at rolling temperatures as

TABLE 2

COMPACTION EQUIPMENT USED ON MORAGA TEST SECTIONS

Type of Compaction	Roller	Roller Wt (tons)	Compaction Wheel		Tiller Wheel	
			Dia. (in.)	Wt (lb/lin in.)	Dia. (in.)	Wt (lb/lin in.)
Steel breakdown	Tandem Bros	12	60	276	48	176
Pneumatic	Nine-wheel Bros	5	—	—	—	—
Steel finish	Tandem Galion	8	53	211	40	142

[a]Temperatures referred to are temperatures at the center of the approximately 2-in. thick lift of AC laid in the test sections.

Figure 2. Mix sticks at moderate temperature (160 F) to pneumatic tires lubricated with water.

high as 250 F occurred. Under these conditions, the pneumatic roller followed closely behind the paver (Fig. 3). Some mixes have been compacted with pneumatic compactors at temperatures as high as 300 F without sticking, provided the tires are lubricated with the release agent. A relative compaction of 94.4 percent was obtained under these rolling conditions. This considerable increase in compaction over that obtained in case B was obtained by increasing the intermediate compaction temperature from about 130 to 210 F.

Section D

Intermediate Pneumatic 5-Ton Compaction, Variable Coverages with 40-psi Tires at High Pavement Temperatures. This section was compacted by a procedure similar to that in section C except that the compactor inched its way across the pavement by moving laterally, progressing one tire width with each pass. Under these conditions, the center of the paved area was covered eight to nine times and the edges, approximately four times. This additional number of roller passes did not increase the compaction obtained. The relative compaction value of 93.5 percent is compared to 94.4 percent obtained with four passes of the roller. These differences are not significant.

Section E

Intermediate Pneumatic 5-Ton Compaction, Four Coverages with 90-psi Tires at High Pavement Temperatures. This section was compacted under conditions similar to those in section C, except that the tire pressure was increased to 90 psi. The relative compaction of 94.4 percent obtained is identical to the value obtained under the same conditions except with 40-psi tire pressure.

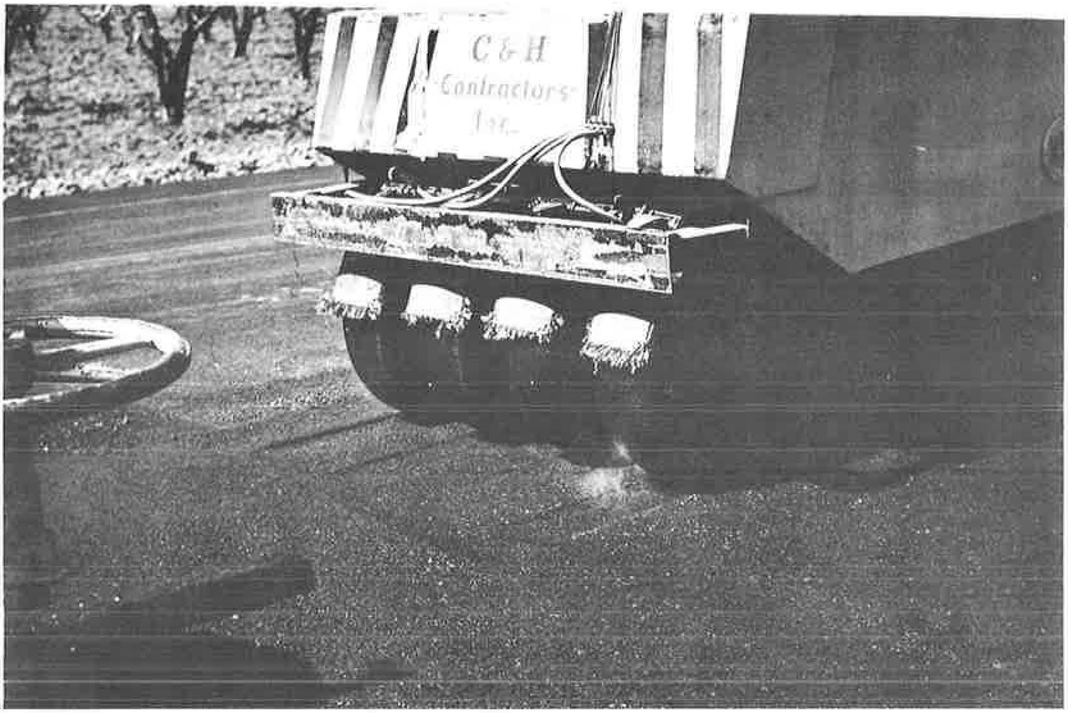

Figure 3. No sticking of mix at high temperature (250 F) to pneumatic tires lubricated with release agent.

Section F

Intermediate Pneumatic 5-Ton Compaction, Multiple Passes with 90-psi Tires at High Pavement Temperatures. The procedure used in this section was similar to that used in section E except that pneumatic rolling was continued until the roller "walked" out of the mix, i.e., until the AC stability increased as a result of a combination of compaction increase and temperature decrease to a point where it could support the roller without appreciable displacement by the tires. A value of 94.4 percent relative compaction was obtained by this procedure as well. The additional number of passes was apparently of no value.

Section G

Pneumatic 5-Ton Breakdown Compaction, Four Coverages with 40-psi Tires, and Steel Wheel Finish. Steel-wheel breakdown was not used on this section. The four coverages of the pneumatic roller at 205-230 F left deep grooves and were removed by the finish roller at approximately 150 F. Poor compaction (89.9 percent relative compaction) was obtained under these conditions. The results were comparable to steel-wheel breakdown without intermediate pneumatic compaction (section A) and low temperature, lightweight intermediate pneumatic compaction (section B).

FOLLOWUP FIELD AND LABORATORY TESTING

A 100-ft section from each test section was examined for the relative compaction obtained by means of the different rolling procedures described. An air permeability representative of each of these sections was established by measuring in-place air flow rates (9) according to the pattern described in Figure 4. A 4-in. diameter core was taken at each spot where an air flow measurement had been made. The density of each of these cores was determined by the procedure of Santucci and Schmidt (10). Air voids

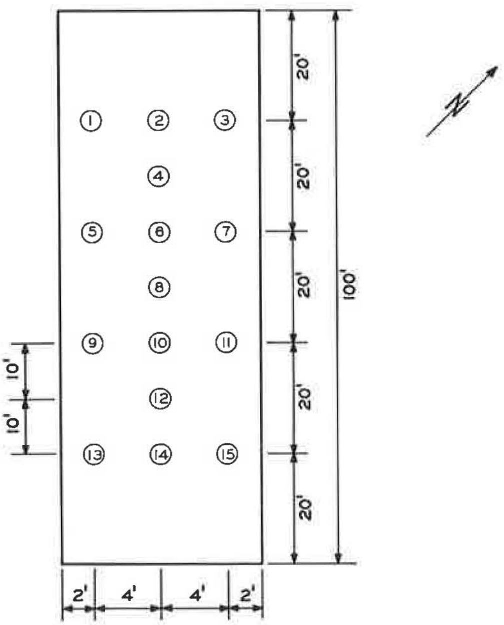

Figure 4. Air permeability and coring pattern on typical Moraga test section.

were calculated by means of the procedure described in the Asphalt Institute Manual, Series No. 2 (11). Rice's method (12) was used to determine the maximum theoretical density of each mix.

In-place air flow rates from each of the sections correlated with calculated air voids' values (Fig. 5). The curve of best fit to the experimental data was calculated by the method of least squares (13), and modified slightly for simplicity in calculation by rounding off the exponent. The equation for this best fit curve is given by:

$$A = 100/(1 + 1/Q)$$

where

A = air voids, percent, and
Q = 0.024 (air flow rate, ml/min)$^{0.25}$

Air flow rate measurements were taken after all rolling had been completed and at a mix temperature of 150 F or less. For this particular mix, which approaches the classification of 6A or 6B in a paper by Hein and Schmidt (14), it was not necessary to correct the air flow rates for temperature.

After the density on each core was determined, the cores from each section were collected and warmed in a 140 F oven, broken up by hand, and mixed together. A representative sample of each of these composite mixes from each section was extracted by means of the modified Abson procedure (ASTM Designation D762-49). A wet screen sieve analysis of the aggregate remaining after the Abson extraction is shown in Figure 1. This grading curve represents the composite curve from three of the test sections.

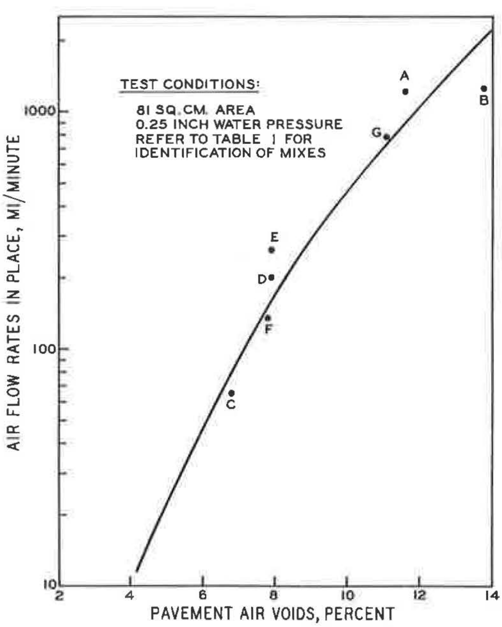

Figure 5. Prediction of pavement air voids from field measured air flow rates.

Hveem stabilities and densities were run on the mixes recovered from the test road and trucks. The average stability was 50. Hveem laboratory densities are given in Table 1.

Penetration and viscosity measurements were made on the asphalts obtained from the extracts. Data on asphalts recovered from the first and second days' field testing are given in Table 3.

DISCUSSION OF RESULTS

Intermediate pneumatic compaction at high pavement temperatures resulted in more complete compaction than was obtained with steel-wheel rolling alone. Pneumatic compaction with a lightweight pneumatic compactor was of no value when

TABLE 3

PROPERTIES OF ASPHALTS RECOVERED FROM MORAGA TEST SECTIONS

Asphalt Property	Composite of Sections B, C, D, E, and F[a]	Composite of Sections A and G[b]	Ref. of Test Method
Penetration at 77 F	68	65	ASTM D 5-61
Viscosity:			
275 F (capillary), stokes	4.39	4.43	ASTM D 445-64
140 F (capillary), kilopoises	2.68	2.83	ASTM D 2170-63T

[a]Placed 3-25-65.
[b]Placed 3-29-65.

made at 135 F, a temperature low enough to prevent sticking between the tires and pavement.

The low densities obtained when pneumatic breakdown rolling was done (section G) were not anticipated. The breakdown pneumatic rolling procedure is known to give good compaction (7) if intermediate pneumatic rolling is used immediately to iron out ridges and grooves left by the breakdown roller. As it was, the finish steel-wheel roller used on section G actually broke down the uncompacted ridges and decompacted the highly compacted grooves left by the pneumatic breakdown roller. Wide variations in densities and air flow rates occurring transversely across the lane width suggest that the finish roller did not effectively even out these crests and depressions.

CONCLUSIONS

The following conclusions were reached from this investigation.

1. With the aid of 3 percent of a release agent in the spray water, a pneumatic tire roller can be used on an asphalt mix at temperatures at least as high as 250-255 F without the mix sticking to the tires. With water only, pickup on the tires of a pneumatic roller is frequently severe at 150-160 F.
2. The compaction procedure of steel-wheel breakdown rolling (high-temperature, intermediate pneumatic rolling) and steel finish rolling gives high density, low permeability, and low air void pavements.
3. The final pavement density obtained appears to be independent of the tire pressures on lightweight pneumatic rollers used for high-temperature intermediate rolling. The densities obtained with 40-psi tire pressures were no different than those obtained with tires pressured to 90 psi. With a heavier pneumatic compactor, higher pressure tires would, no doubt, cause increased compaction.
4. The data showed that breakdown pneumatic rolling must be followed immediately by intermediate pneumatic compaction to iron out the grooves left during breakdown rolling. If ironed out by steel rollers at low temperatures, decompaction occurs and low densities result.

High density, low permeability, low air voids, and highly durable pavements can be obtained with proper compaction procedures. The use of a pneumatic roller for intermediate compaction of a mix at high temperatures has been demonstrated as a most effective way of obtaining these desirable properties.

REFERENCES

1. Santucci, L. E., and Schmidt, R. J. Setting Rate of Asphalt Concrete. Highway Research Board Bull. 333, pp. 1-9, 1962.

2. Monismith, C. L. Asphalt Mixture Behavior in Repeated Flexure. Rept. No. TE-64-2, Inst. of Eng. Res., Univ. of Calif. at Berkeley, Nov. 1964.
3. Hveem, F. N., and Vallerga, B. A. Density Versus Stability. Proc., AAPT, Vol. 21, pp. 237-262, Jan. 1952.
4. Heithaus, J. J., and Johnson, R. W. A Microviscometer Study of Road Asphalt Hardening in the Field and Laboratory. Proc., AAPT, Vol. 27, pp. 17-34, Feb. 1958.
5. Schmidt, R. J., Kari, W. J., Bower, H. C., and Hein, T. C. Behavior of Hot Asphaltic Concrete Under Steel Wheel Rollers. Highway Research Board Bull. 251, pp. 18-37, Jan. 1960.
6. Lovering, W. R. Achieving Adequate Compaction of Asphalt Paving. Modern Highways, May 1959.
7. Rensel, P. Simplified Quality Control of Asphalt Concrete. Presented at Northwest Roads and Streets Conf., Univ. of Washington, Jan. 1965.
8. Calman, E. California's Experience on Compaction of Asphalt Concrete Pavements. Proc., 4th Annual Highway Conf., Univ. of the Pacific, 1961.
9. Ellis, W. H., and Schmidt, R. J. A Method for Measuring the Air Permeabilities of Asphalt Concrete Pavements. ASTM Spec. Tech. Publ. 294, pp. 85-91, June 1960.
10. Santucci, L. E., and Schmidt, R. J. Immediate Core Volume by Presaturation. ASTM Spec. Tech. Publ. 309, pp. 34-48, June 1961.
11. Mix Design Methods for Asphalt Concrete and Other Hot Mix Types. Asphalt Institute Manual, Series No. 2, 2nd ed., p. 133, May 1963.
12. Rice, J. M. Volumetric Methods for Measuring Asphalt Content and Effective Gravity of Aggregates in Bituminous Mixtures. Proc., AAPT, Vol. 22, pp. 284-301, Jan. 1953.
13. Bennett, C. A., and Franklin, N. L. Statistical Analysis in Chemistry and the Chemical Industry, John Wiley and Sons, Inc., 1954.
14. Hein, T. C., and Schmidt, R. J. Air Permeability of Asphalt Concrete. ASTM Spec. Tech. Publ. 309, pp. 49-62, June 1961.
15. Goode, J. F., and Lufsey, L. A. A New Graphical Chart for Evaluating Aggregate Gradations. Proc., AAPT, Vol. 31, pp. 176-207, Jan. 1962.

Hardening of Asphalt in Hot Bituminous Mixes During the Hauling Process

PAUL H. WRIGHT and RADNOR J. PAQUETTE
Respectively, Assistant Professor, and Professor, Georgia Institute of Technology

Concern has been expressed by highway engineers that asphalt in hot bituminous mixes may harden excessively during the hauling process, particularly if the haul distance is long. Such hardening could conceivably cause early brittleness in asphalt cement under service conditions and shorten the pavement's life.

The research reported herein was performed at Georgia Tech. under the sponsorship of the Georgia Highway Department and the U.S. Bureau of Public Roads to determine the significance and seriousness of asphalt hardening during the period of haul.

Samples of hot bituminous mix were taken from trucks traveling en route from the mixing plant to the paving site at times of one, two, and four hours after preparation of the mix. In addition, mix samples were taken immediately after mixing and after the material had been placed on the roadway. A total of about 100 field samples were taken from ten trucks, including asphalts manufactured from four sources of crude.

Recovery of the asphalt was accomplished by the Abson method. Absolute viscosity was used as the primary measure of asphalt hardness. A microviscometer was used to measure viscosity at temperatures of 60, 77, and 95 F. Viscosities were reported at 0.05 reciprocal seconds. Penetration and ductility tests were also conducted. To provide some indication of molecular changes, thin asphalt films were placed between sodium chloride plates and tested on an infrared spectrophotometer.

Significant hardening occurred in all of the asphalts studied. The extent of hardening varied with the asphalt source. Also, asphalt viscosities varied significantly with the position and depth from which the mix sample was taken from the truck bed. Asphalt samples taken from the uppermost 1 in. were generally softer than those in the remainder of the mix.

Asphalt ductility decreased with increase in haul time. Changes in the infrared spectra occurred at wave lengths of 2.9, 5.9, and 9.7 microns.

•AS EARLY as 1929, it was known that asphalt in hot bituminous mixes hardens during the mixing process (1). Since that time the nature and magnitude of the hardening during mixing has been the subject of extensive research. In 1937, Brannon (2) reported that mixing time as well as temperature affected the hardness of asphalts later recovered from pavements. A recent comprehensive study by Bright and Reynolds (3) showed that the hardening of asphalt in hot mixes is also dependent on the rate and

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manner in which mix samples are cooled. The severity of hardening during mixing remains a matter of widespread interest and controversy (4, 5).

A great deal of research effort has also been directed to evaluating and measuring the hardening that occurs during a pavement's service life. Brannon's study (2) of asphalts recovered from wearing surfaces indicated a steady decrease in penetration with age amounting to a decrease of 47 percent at 30 months. White (6) asserted in 1947 that asphalt content is the greatest single factor determining the amount of drop in penetration of asphalt in service. In a test road project in Michigan, Parr, and Serafin (7) found that penetration of asphalt extracted from cores decreased up to the age of 17 months, and then remained constant or increased slightly thereafter. The work of Simpson, Griffin and Miles in the Zaca-Wigmore project (8) confirmed the finding that most of the hardening occurs in the first 16 to 20 months, and reported that pavements with high air void content hardened more rapidly than those with low air voids, giving credence to White's earlier statement concerning the effect of asphalt content. The Zaca-Wigmore project also showed that hardening of an asphalt is greatest at the top of a pavement and decreases with increasing depth in the pavement.

In contrast to the amount of research on hardening during mixing and during a pavement's service life, little research has been done to measure hardening during the hauling process. Bright and Reynolds (3) took a cursory look at this problem in their study of the effect of mixing temperature on asphalt hardening. They followed trucks en route from the plant to the paver, took samples, and measured the hardness of the recovered asphalt. Their research indicated that some hardening of the asphalt did occur, but the results were spotty and generally inconclusive.

The purpose of the research reported here was to determine if significant asphalt hardening does occur during the haul period, and if so, to measure and evaluate the severity of the haul effect.

PROCEDURE

To measure the effect of haul time on asphalt hardening, samples were taken from trucks en route from a central mixing plant to the construction site at the following times: at central plant at time of loading; one, two, and four hours after mixing; and immediately after placement but before compaction.

Mixes were prepared at temperatures of approximately 300 and 320 F. No attempt was made to vary the mixing time from that normally used by the manufacturer. All but two of the mixes were type E surface mixes, Georgia State Highway Department specifications, with a 60-70 penetration grade asphalt. The remaining mixes were type A binder mixes.

The trucks were driven at normal haul speeds and were in constant motion except for periodic stops made for sampling. The mixes were not covered during the haul period.

Samples were taken from the truck bed in accordance with the general recommendations of ASTM D979-51. The total weight of each sample was approximately 10 lb. Each sample was composed of subsamples taken from three arbitrarily chosen locations across the width and length of the truck bed. Samples were taken from the crust (top 1 in.) and from a depth of approximately 12 in.

The field samples were placed in cloth sampling bags and cooled by quenching in water. (To test the effect of the rate of cooling, a single sample was taken from each truck at the time of mixing and allowed to cool normally to air temperature.) The samples were stored in water from the time of sampling until removal for the extraction procedure.

After removal from the water, the samples were dried in a 250 F oven for approximately 30 min. Care was taken to insure that the temperature of the sample did not exceed 150 F during the drying process. A special series of tests was conducted which compared viscosities of asphalts recovered from samples dried at normal air temperatures to those from samples dried in the oven. It was concluded that no significant hardening resulted from drying the samples in the oven.

The asphalt was extracted by means of a Dulin rotarex centrifugal extractor using an analytical reagent grade benzene. The asphalt was recovered by the Abson method, ASTM D1856, with slight modifications. Instead of the specified glass aeration tube, a copper tube similar to that specified by ASTM D762 was used to admit carbon dioxide to the asphalt benzene solution during the distillation procedure. It was not always possible to dry the sample and extract and recover the asphalt in an 8-hr period.

Absolute viscosities were determined with a microviscometer (Fig. 1) using the procedure proposed by Griffin, et al. (9). Viscosities were determined for a shear rate of 0.05 reciprocal seconds and at test temperatures of 60, 77, and 95 F.

Penetration tests were conducted on the asphalt samples in accordance with ASTM D5-61. Ductility tests were run at a test temperature of 48 ± 5 F and a rate of pull of 2 cm/min.

For each series of tests, infrared spectrograms were made for the original asphalt and for samples taken immediately after mixing and after a 4-hr haul. Because the amount of absorption of infrared light is directly related to the thickness of asphalt film, special care was required to obtain asphalt films of a constant thickness. Asphalt films were formed between 2-cm by 4-cm sodium chloride plates, using a film former (Fig. 2).

After the film former had been precisely adjusted by means of the plates and a 0.003-in. gage, it was heated in a 250 F oven along with the asphalt sample. With special care, the heated asphalt sample was then placed between the plates and pressed to a thickness of 76 ± 3 microns by the film former.

The asphalt films were irradiated by a Beckman model IR8 infrared spectrophotometer.

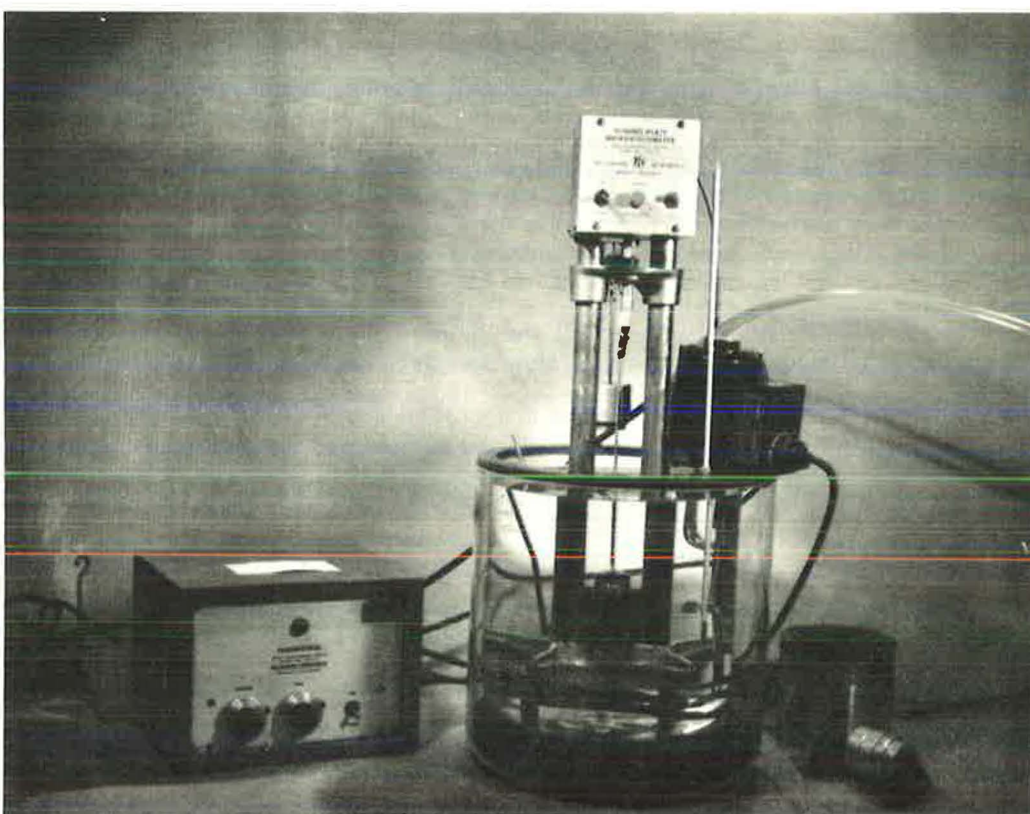

Figure 1. Sliding plate microviscometer.

Figure 2. Film former.

RESULTS

Microviscosity Test Results

It is convenient to consider the microviscosity test results as part of one of three groups of experiments: (a) the basic test series, (b) control experiments, (c) special tests to define sources of variation.

In the basic test series, nine trucks were stopped en route from the mixing plant to the construction site and field samples were periodically taken during the haul process. At least ten mix samples were taken for each truck. These samples were stored in water, and later, in the laboratory, asphalt samples were extracted, recovered, and tested by the microviscometer.

To provide a comparison with results expected in normal construction procedures, three control experiments were conducted. In each of these experiments, samples were taken from trucks with normal delay (about 10 to 20 min) and with a delay of 4 hr. To test the possible effect of haul time on the long-term hardening characteristics of asphalt under service conditions, core samples were taken from the pavements constructed from each of these six truck loads of bituminous mix.

In addition, three special series of tests were made to isolate and define sources of variation in the viscosity data.

In all, more than 800 microviscosity tests were conducted during this study.

Basic Test Series. —The viscosity results for the basic test series showed a recurring pattern of hardening during the haul process. Although there were significant variations in the data attributable to at least seven sources, the data clearly showed

TABLE 1

MEAN ABSOLUTE VISCOSITY OF NINE SERIES OF TESTS VS HAUL TIME

Haul Time	Mean Absolute Viscosity (megapoises)		
	60 F	77 F	95 F
Original asphalt	18.9	2.19	0.265
After mixing	18.5	2.52	0.377
1 hr after mixing	22.1	3.78	0.551
2 hr after mixing	26.6	5.07	0.686
4 hr after mixing	30.4	7.10	1.208
After placement	27.3	5.20	0.683
1 week after placement	34.0	7.73	1.196

that hardening does occur during haul and that the extent of hardening depends on the duration of haul.

Mean absolute viscosities for the basic series of tests are given in Table 1; viscosity is related to duration of haul in Figure 3; Figure 4 is a semilogarithmic plot of viscosity and test temperature.

Absolute viscosities of samples taken after the 4-hr haul were on the order of two or three times as great as those of samples taken immediately after mixing.

The results showed an unexpected decrease in viscosity during the process of placing the mix on the roadway. This apparent anomaly was evidently due to heterogeneity of the mix in the truck bed and a failure to obtain samples from the truck truly representative of the entire mix. Detailed results for the basic series of experiments are given in Table 2.

Effect of Asphalt Source. —As expected, wide variation in hardening patterns was noted for different asphalt sources (Fig. 5).

The asphalt from Shell Oil Company, Atlanta, showed the greatest change in viscosity during haul, increasing by more than threefold. This asphalt was a mixture produced from crude oils from foreign (Venezuela) and domestic (Louisiana) sources. It was the only asphalt source sampled processed in a continuous mixing plant.

The asphalt supplied by Hunt Oil, Tuscaloosa, Ala., showed a moderate change in viscosity during the haul process. It was the only one tested produced entirely from domestic crudes. The supplier listed its source as Warrior Basin (Mississippi and Alabama).

The Humble Oil asphalt also showed a moderate amount of hardening during haul, although the viscosity values were relatively small. The supplier indicated that it was a blend of Venezuelan (Lagunillas) and domestic asphalts (probably Texas or Louisiana).

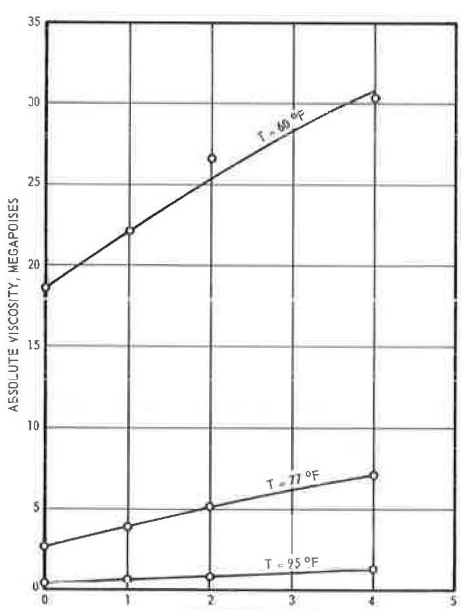

Figure 3. Mean absolute viscosity vs haul duration, average of nine series of tests.

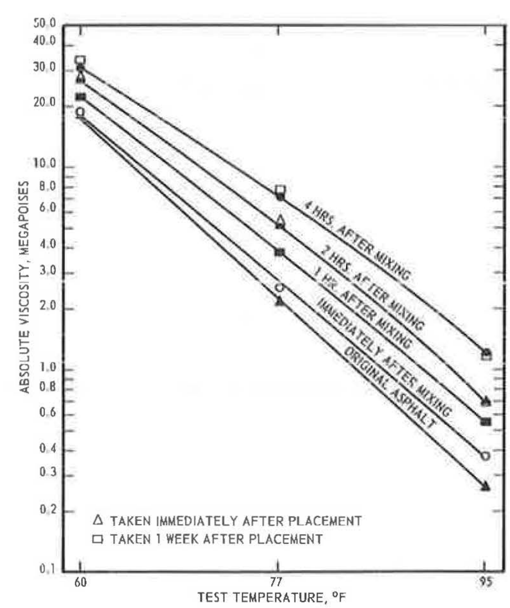

Figure 4. Mean absolute viscosity vs test temperature.

TABLE 2

ABSOLUTE VISCOSITY VS TIME OF SAMPLING

Asphalt Source	Sample Series	Mixing Temperature (deg F)	Absolute Viscosity (megapoises)						
			Original Asphalt	After Mixing	1 Hr Later	2 Hr Later	4 Hr Later	After Placement	1 Week Later
(a) At 60 F									
Humble Oil,	1100	315	20.0	26.2	31.8	36.9	35.0	37.1	34.9
Charleston	1100	315	20.0	25.0	27.0	34.9	40.0	37.0	33.0
Humble Oil,	1200	300	21.9	27.1	19.1	37.2	33.5	37.5	30.8
Charleston	1200	300	21.9	27.5	21.4	36.5	34.8	36.4	36.8
Hunt Oil,	2100	320	18.1	6.9	20.4	15.4	20.1	20.5	16.7
Tuscaloosa	2100	320	18.1	7.7	20.9	14.3	19.8	19.7	16.7
Hunt Oil,	2200	316	17.2	19.0	19.4	14.5	26.7	19.0	33.5
Tuscaloosa	2200	316	17.2	17.8	20.0	14.5	24.1	30.0	35.0
Hunt Oil,	B2100[a]	320	18.1	18.9	23.0	30.0	29.1	24.9	25.4
Tuscaloosa	B2100[a]	320	18.1	15.9	22.2	35.5	23.1	27.7	24.7
Hunt Oil,	B2200[a]	295	17.2	19.2	12.6	21.5	27.3	20.2	26.6
Tuscaloosa	B2200[a]	295	17.2	18.3	16.1	18.5	25.0	17.2	25.7
Shell Oil,	4100	320	16.9	17.4	24.6	38.4	40.0	31.8	39.2
Atlanta	4100	320	16.9	17.6	26.7	34.5	39.0	31.9	35.7
Shell Oil,	4200	300	17.8	19.2	30.1	27.5	35.0	22.4	36.0
Atlanta	4200	300	17.8	19.6	29.8	27.8	35.5	21.3	37.1
Shell Oil,	3100	325	19.6	14.8	17.5	21.2	28.0	31.2	60.0
Savannah	3100	325	19.8	15.2	15.8	19.9	30.3	26.0	65.0
(b) At 77 F									
Humble Oil,	1100	315	1.75	1.98	3.50	6.74	4.24	5.20	5.92
Charleston	1100	315	1.75	2.40	3.08	6.52	4.21	5.52	7.53
Humble Oil,	1200	300	1.67	2.15	2.02	4.07	3.94	4.69	6.55
Charleston	1200	300	1.67	2.39	2.00	4.39	4.30	4.49	6.45
Hunt Oil,	2100	320	2.79	1.28	4.35	2.76	4.18	4.30	3.44
Tuscaloosa	2100	320	2.79	1.31	4.33	2.73	4.56	4.75	3.40
Hunt Oil,	2200	316	2.65	2.70	3.65	2.92	5.20	6.00	8.71
Tuscaloosa	2200	316	2.65	2.65	3.75	3.23	4.95	5.55	8.75
Hunt Oil,	B2100[a]	320	2.79	3.32	5.08	7.38	8.58	5.31	6.28
Tuscaloosa	B2100[a]	320	2.79	2.95	5.06	8.12	5.66	5.28	6.59
Hunt Oil,	B2200[a]	295	2.65	3.55	2.95	4.34	5.54	4.24	6.03
Tuscaloosa	B2200[a]	295	2.65	3.39	2.62	4.32	5.45	3.29	5.75
Shell Oil,	4100	320	2.31	2.72	4.29	8.61	10.9	7.35	7.59
Atlanta	4100	320	2.31	2.60	4.52	8.13	10.5	7.22	7.74
Shell Oil,	4200	300	2.42	3.02	5.82	5.67	8.50	3.97	10.3
Atlanta	4200	300	2.42	3.62	6.15	5.50	8.65	4.56	9.51
Shell Oil,	3100	325	1.93	1.70	2.53	2.92	13.9	6.22	14.7
Savannah	3100	325	1.97	1.60	2.26	2.84	14.6	5.71	13.9
(c) At 95 F									
Humble Oil,	1100	315	0.183	0.207	0.340	0.664	0.392	0.493	0.660
Charleston	1100	315	0.183	0.250	0.302	0.586	0.388	0.503	0.672
Humble Oil,	1200	300	0.189	0.242	0.209	0.385	0.422	0.425	0.698
Charleston	1200	300	0.189	0.238	0.178	0.330	0.560	0.470	0.664
Hunt Oil,	2100	320	0.382	0.224	0.634	0.460	0.798	0.680	0.563
Tuscaloosa	2100	320	0.382	0.200	0.632	0.492	0.946	0.813	0.507
Hunt Oil,	2200	316	0.355	0.534	0.690	0.430	0.968	1.04	1.79
Tuscaloosa	2200	316	0.355	0.531	0.715	0.495	0.962	1.06	1.76
Hunt Oil,	B2100[a]	320	0.382	0.516	0.788	1.50	1.51	0.758	1.24
Tuscaloosa	B2100[a]	320	0.382	0.522	0.733	1.55	1.38	0.824	1.21
Hunt Oil,	B2200[a]	295	0.355	0.642	0.409	0.585	1.09	0.493	0.901
Tuscaloosa	B2200[a]	295	0.355	0.595	0.498	0.612	1.16	0.526	0.964
Shell Oil,	4100	320	0.283	0.417	0.662	0.938	1.77	0.921	0.996
Atlanta	4100	320	0.283	0.417	0.725	0.930	1.56	0.893	0.989
Shell Oil,	4200	300	0.287	0.422	0.905	0.830	1.10	0.533	1.45
Atlanta	4200	300	0.287	0.436	0.914	0.812	1.14	0.574	1.45
Shell Oil,	3100	325	0.226	0.198	0.302	0.378	2.86	0.795	2.43
Savannah	3100	325	0.217	0.190	0.275	0.362	2.74	0.499	2.58

[a]Binder mix.

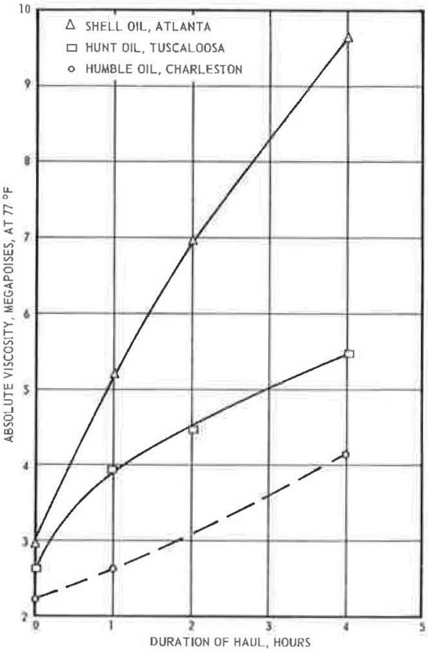

Figure 5. Absolute viscosity vs haul duration.

The viscosities of the 2-hr samples varied widely from the expected pattern. The reason is not known, but it is thought to be due in part to the manner in which field samples were taken.

Only one series of samples was taken from the plant which used asphalt from the Shell Oil Co., Savannah, Ga. Viscosities were the smallest of those tested. Moderate increases in viscosity occurred with increases in the haul period. Curiously, the viscosities roughly doubled during the first week after the pavement was constructed.

Effect of Mixing Temperature. —One of the original aims of the project was to measure the effect of extreme mixing temperatures by mixing the bituminous samples at 250 and 350 F. Because of practical and economic considerations it was decided in the early stages of the project that use of such extreme mixing temperatures was not feasible. Mixes prepared at these temperatures would not meet Georgia Highway Department specifications. This course of action led to two alternatives: using mixes which did not meet specifications, or wasting 10 to 15 tons of bituminous material for each truck sampled. Furthermore, mixes prepared at 250 F with some heat loss during the haul process would have been difficult, if not impossible, to place and compact. It was therefore decided to prepare the mixes at a high temperature and a low temperature, but to keep both temperatures within the specification limits. The average high temperature used was approximately 320 F and the low temperature was about 300 F.

Statistical analysis of the results showed that mixing temperature was a significant source of variation. Except for those taken just after mixing, the samples mixed at the high temperature tended to be slightly harder than those mixed at the low temperature. The average magnitude of this difference, however, was on the order of 20 percent or less. It could be concluded from these results that variations in mixing temperature of 20 F (within the range provided by the specifications) are of little practical concern even though they account for significant variations in the asphalt viscosity.

Unexpectedly, the samples taken just after mixing which were mixed at high temperatures showed smaller viscosities than those mixed at low temperatures. Possibly, this anomaly occurred strictly because of chance variations. A statistical variance-ratio test (F-test) indicated that the variation in results at the two mixing temperatures were significantly different at the 5 percent level. However, a t-test at the 10 percent level failed to show significant difference between the means of the viscosities at the high and low temperatures.

Average viscosities for high and low mixing temperatures for samples taken at different haul times are shown in Figure 6.

Effect of Mix Heterogeneity. —It is common knowledge among those who have observed bituminous paving operations that asphalt mixes tend to form a hard crust at the upper surface during the hauling process. Despite the fact that this crust amounts to a small percentage of the total volume of the mix, it was decided during the planning stages of this project to take three samples from the crust during haul at times of 1, 2, and 4 hours after mixing for comparisons with those samples taken beneath the surface.

Believing that the hardening phenomenon was primarily an oxidation process, and one which depended on exposure to air, the investigators expected that the asphalt

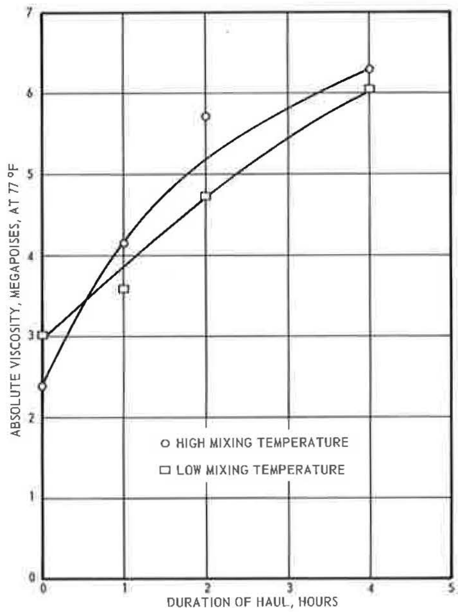

Figure 6. Absolute viscosity vs haul duration.

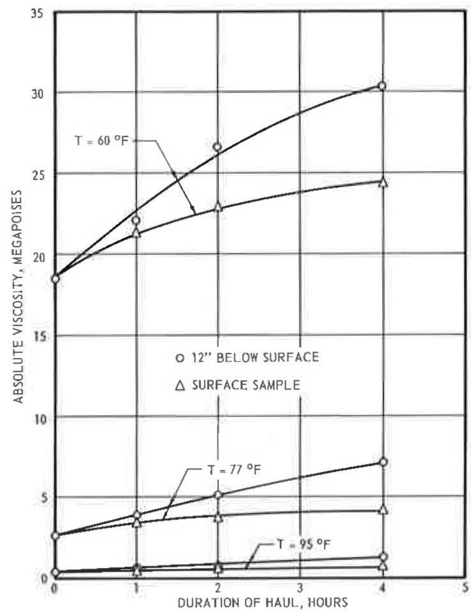

Figure 7. Absolute viscosity vs haul duration.

samples taken from the uppermost 1 in. of material would yield higher viscosities than samples taken beneath the crust. However, the results showed that viscosities of asphalt samples taken from the crust were significantly lower (softer asphalt) rather than higher at all three laboratory test temperatures. This is thought to be due to a vertical temperature gradient which has been observed while the truck is in motion. Temperatures of the material in the crust were approximately 120 F, whereas the temperatures 12 in. beneath the surface were roughly 300 F. Indeed, the low temperatures at the surface account for the formation of the hard crust which softens when the truck stops and the temperature of the surface approaches the temperature of the mass underneath.

Figure 7 compares average viscosities of crust samples with those taken 12 in. beneath the surface for three test temperatures. Detailed test data for crust and non-crust samples are given in Table 3.

The results of the crust series of tests coupled with unexplained variations in the test data prompted the researchers to examine more closely the heterogeneity of the mix within the truck bed. To obtain a measure of mix heterogeneity, a special series of samples was taken. As before, a truck was loaded with bituminous material and driven for 4 hr. However, no samples were taken en route to the construction site. At the end of the 4-hr haul period, nine representative samples were carefully taken from various locations within the mass. Samples were taken from the front, middle, and back of the truck bed and at depths of 1, 12, and 24 in. An analysis of variance indicated that the effects of both position and depth of sampling were significant at the 0.1 percent level. Graphs of averages of these results are shown in Figures 8 and 9.

The smallest and largest viscosities differed as much as 250 percent (Figs. 8, 9 and Table 4), which explains to a large extent the sizable variations that occurred earlier in the main series of tests and the apparent decrease in viscosity during the process of placing the mix on the roadway.

The highest viscosities were obtained from a depth of 12 in. and the lowest from a depth of 24 in. These results imply that the main series of tests, sampled at 12-in. depths, represents an upper limit of the hardening occurring throughout the truck bed. The probable reason for the lower viscosities for the samples taken at a depth of 24 in. is that this material is insulated from the air by the material lying above it.

TABLE 3

ABSOLUTE VISCOSITY OF SAMPLES FROM CRUST VS SAMPLES FROM 12 IN. BENEATH SURFACE

Source	Series	Absolute Viscosity (megapoises)					
		1 Hr After Mixing		2 Hr After Mixing		4 Hr After Mixing	
		Crust	Non-Crust	Crust	Non-Crust	Crust	Non-Crust
(a) Test Temperature 60 F							
Humble Oil,	1100	30.1	31.8	32.0	36.9	38.0	35.0
Charleston	1100	29.1	27.0	27.5	34.9	38.0	40.0
Humble Oil,	1200	14.3	19.1	24.1	37.2	24.0	33.5
Charleston	1200	20.0	21.4	23.9	36.5	22.5	34.8
Hunt Oil,	2100	14.8	20.4	18.5	15.4	13.6	20.1
Tuscaloosa	2100	13.4	20.9	18.1	14.3	12.8	19.8
Hunt Oil,	2200	21.7	19.4	19.2	14.5	26.0	26.7
Tuscaloosa	2200	21.7	20.0	19.0	14.5	24.0	24.1
Hunt Oil,	B2100	16.7	23.0	17.4	30.0	20.9	29.1
Tuscaloosa	B2100	24.1	22.2	18.0	35.5	18.9	23.1
Hunt Oil,	B2200	22.2	12.6	21.9	21.5	20.6	27.3
Tuscaloosa	B2200	19.9	16.1	22.2	18.5	21.8	25.0
Shell Oil,	4100	22.4	24.6	30.3	38.4	30.5	40.0
Atlanta	4100	23.6	26.7	26.0	34.5	31.2	39.0
Shell Oil,	4200	25.8	30.1	26.0	27.5	25.6	35.0
Atlanta	4200	25.5	29.8	23.7	27.8	24.6	35.5
Shell Oil,	3100	18.3	17.5	22.1	21.2	23.6	28.0
Savannah	3100	19.0	15.8	22.9	19.9	23.4	30.3
Average		21.3	22.1	22.9	26.6	24.4	30.35
(b) Test Temperature 77 F							
Humble Oil,	1100	3.95	3.50	4.19	6.74	4.27	4.24
Charleston	1100	3.94	3.08	3.43	6.52	3.97	4.21
Humble Oil,	1200	0.99	2.02	2.94	4.07	2.34	3.94
Charleston	1200	1.16	2.00	2.25	4.39	2.28	4.30
Hunt Oil,	2100	2.76	4.35	3.10	2.76	2.35	4.18
Tuscaloosa	2100	2.47	4.33	2.97	2.73	2.14	4.56
Hunt Oil,	2200	4.18	3.65	4.06	2.92	5.28	5.20
Tuscaloosa	2200	4.18	3.75	3.80	3.23	5.77	4.95
Hunt Oil,	B2100	4.10	5.08	3.17	7.38	5.08	8.58
Tuscaloosa	B2100	3.98	5.06	3.25	8.12	3.80	5.66
Hunt Oil,	B2200	3.08	2.95	3.49	4.34	4.45	5.54
Tuscaloosa	B2200	3.34	2.62	3.42	4.32	3.95	5.45
Shell Oil,	4100	4.18	4.29	4.93	8.61	5.71	10.9
Atlanta	4100	4.05	4.52	4.72	8.13	5.45	10.5
Shell Oil,	4200	4.73	5.82	5.16	5.67	5.22	8.50
Atlanta	4200	4.67	6.15	5.04	5.50	5.16	8.65
Shell Oil,	3100	2.54	2.53	4.06	2.92	3.37	13.9
Savannah	3100	2.55	2.26	3.80	2.84	3.06	14.6
Average		3.38	3.78	3.77	5.07	4.09	7.10
(c) Test Temperature 95 F							
Humble Oil,	1100	0.378	0.340	0.312	0.664	0.410	0.392
Charleston	1100	0.368	0.302	0.358	0.586	0.328	0.388
Humble Oil,	1200	0.115	0.209	0.245	0.385	0.233	0.422
Charleston	1200	0.132	0.178	0.254	0.330	0.237	0.560
Hunt Oil,	2100	0.424	0.634	0.600	0.460	0.451	0.798
Tuscaloosa	2100	0.400	0.632	0.673	0.492	0.353	0.946
Hunt Oil,	2200	0.850	0.690	0.676	0.430	0.993	0.968
Tuscaloosa	2200	0.850	0.715	0.677	0.495	1.02	0.962
Hunt Oil,	B2100	0.760	0.788	0.500	1.50	0.706	1.51
Tuscaloosa	B2100	0.698	0.733	0.487	1.55	0.622	1.38
Hunt Oil,	B2200	0.435	0.409	0.608	0.585	0.886	1.09
Tuscaloosa	B2200	0.530	0.498	0.646	0.612	0.960	1.16
Shell Oil,	4100	0.485	0.662	0.533	0.938	0.668	1.77
Atlanta	4100	0.458	0.725	0.577	0.930	0.625	1.56
Shell Oil,	4200	0.545	0.905	0.779	0.830	0.659	1.10
Atlanta	4200	0.600	0.914	0.760	0.812	0.660	1.14
Shell Oil,	3100	0.287	0.302	0.360	0.378	0.356	2.86
Savannah	3100	0.286	0.275	0.416	0.362	0.280	2.74
Average		0.478	0.551	0.526	0.686	0.580	1.208

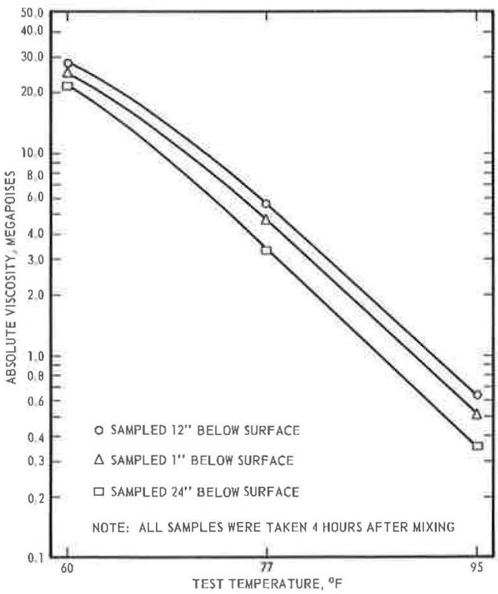

Figure 8. Absolute viscosity vs test temperature.

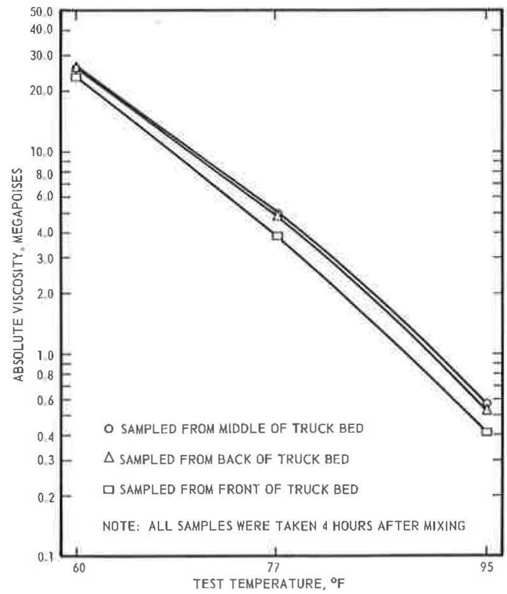

Figure 9. Absolute viscosity vs test temperature.

TABLE 4

EFFECT OF LOCATION OF SAMPLE IN TRUCK BED ON ABSOLUTE VISCOSITY[a]

Depth of Sample (in.)	Absolute Viscosity (megapoises)			Avg.
	Back	Middle	Front	
(a) Test Temperature 60 F				
1	28.5	24.2	24.3	25.0
	25.5	24.8	22.9	
12	28.4	30.5	24.7	28.0
	28.3	32.0	24.2	
24	21.5	21.7	22.0	21.9
	21.4	22.4	22.4	
Avg.	25.6	25.9	23.4	
(b) Test Temperature 77 F				
1	5.87	4.24	4.12	4.71
	5.59	4.49	3.92	
12	5.75	6.76	4.32	5.61
	5.63	6.90	4.28	
24	3.26	3.67	3.31	3.32
	3.13	3.64	2.92	
Avg.	4.87	4.95	3.81	
(c) Test Temperature 95 F				
1	0.605	0.484	0.430	0.506
	0.615	0.462	0.439	
12	0.610	0.826	0.489	0.634
	0.602	0.830	0.444	
24	0.360	0.386	0.339	0.357
	0.331	0.392	0.332	
Avg.	0.521	0.563	0.412	

[a]Asphalt source: blend, Venezuelan and domestic; asphalt grade: 60-70 pen.; gradation of mix: type E surface mix.

Relatively small viscosities were noted for the samples taken from the front of the truck bed. This is thought to be due to the probable lower temperatures in this portion of the mix.

Effect of Field Sample Cooling Procedure. —For the main series of tests, field samples were quenched in water immediately after sampling. However, as an adjunct to the basic experiment, an additional sample was taken from each truck just after preparation of the mix. It was allowed to cool gradually to air temperature, providing a measure of the effect of method of cooling on the asphalt viscosity. A variance-ratio test showed the method-of-cooling effect to be significant at the 0.1 percent level. Of the 54 comparisons made, 36 of the air-cooled viscosities were higher than the viscosities of the quenched samples. Average viscosities of the air-cooled samples were approximately 20 to 40 percent higher than the quenched samples (Table 5). These results are consistent with the results of a study by Bright and Reynolds (3) which showed large differences in percent penetration retained for quenched and unquenched samples, especially at high mixing temperatures.

Control Tests. —Three control experiments were conducted to provide a measure of the effect of long haul times on the hardness of asphalt in pavements during the first year of pavement life. Core samples were periodically taken from pavement sections for which the bituminous mix had been hauled for 4 hr and from comparable sections

TABLE 5

EFFECT OF METHOD USED IN COOLING FIELD SAMPLES ON ABSOLUTE VISCOSITY

Series	Absolute Viscosity (megapoises)					
	60 F		77 F		95 F	
	Air Cooled	Quenched	Air Cooled	Quenched	Air Cooled	Quenched
1100	20.5	26.2	1.82	1.98	0.196	0.207
	21.4	25.0	1.94	2.40	0.189	0.250
1200	11.0	27.1	0.978	2.15	0.121	0.242
	13.2	27.5	1.15	2.39	0.129	0.238
2100	19.2	6.88	3.92	1.28	0.833	0.224
	19.8	7.65	3.78	1.31	0.797	0.200
2200	18.6	19.0	3.70	2.70	0.530	0.534
	19.2	17.8	3.62	2.65	0.555	0.531
B2100	23.5	18.9	3.89	3.32	0.663	0.516
	21.2	15.9	3.90	2.95	0.818	0.522
B2200	18.7	19.2	3.72	3.55	0.502	0.642
	14.7	18.3	3.53	3.39	0.549	0.595
3100	19.4	14.8	2.78	1.69	0.360	0.198
	20.5	15.2	2.92	1.60	0.352	0.190
4100	30.0	17.4	5.40	2.72	0.560	0.417
	29.4	17.6	5.71	2.60	0.583	0.417
4200	36.8	19.2	6.27	3.02	0.832	0.422
	35.3	19.6	7.07	3.62	0.772	0.436
Avg.	21.8	18.5	3.67	2.52	0.519	0.377

for which the mix had been hauled less than 30 min. Results of these control tests are given in Table 6.

The data shown for the control tests appear to be highly variable, and the pattern of hardening is irregular. However, the results show that during the first few weeks of pavement life asphalt hauled for 4 hr is harder than that hauled for less than 30 min. This differential in hardness is still evident in core samples taken 10 months after construction.

The true character of the pavement hardening phenomenon is obscured due to variations in viscosity with pavement depth (8, 10). The values given in Table 6 may be considered as average viscosities for the entire surface course.

To illustrate the extent of variations of hardening with pavement depth, six pavement cores, 10 months of age, were sliced into thin sections with a diamond saw and the viscosities of the asphalt recovered from these slices were determined by the microviscometer. These data are given in Table 7. These results indicate that asphalt viscosity decreases nonlinearly with pavement depth, the viscosity of the asphalt at the surface being on the order of 50 percent greater than that ½ in. below the surface. Similar results have been found by Coons (11) in his study of eleven pavement surfaces ranging in age from 1 to 13 yr.

Results of Penetration Tests

Penetration tests were conducted for all the asphalt cement samples using the standard procedure given in ASTM D5-61. In most cases, these findings closely paralleled the results of the microviscosity tests. The penetration results and the absolute viscosities at 77 F were exponentially related (Fig. 8). The following least-squares regression model was developed for these variables:

$$\text{Absolute Viscosity at 77 F, megapoises} = 3591.3\ (\text{penetration})^{-1.719}$$

Although the regression coefficients for this model were significant at the 1 percent level, the correlation coefficient of 0.81 suggests that only about two-thirds of the variation in viscosity is explained by the variations in penetration. There is considerable scatter away from the best-fit curve (Fig. 10).

Basic Test Series. —The penetration results confirmed that hardening does occur during the haul process. The average penetration values decreased during the 4-hr

TABLE 6

CONTROL TEST DATA—ABSOLUTE VISCOSITY VS TIME OF SAMPLING

Time of Sampling	Absolute Viscosity (megapoises)					
	Hunt Oil, Tuscaloosa		Shell Oil, Savannah		Shell Oil, Atlanta	
	Not Delayed	Delayed 4 Hr	Not Delayed	Delayed 4 Hr	Not Delayed	Delayed 4 Hr
(a) At 60 F						
Original asphalt	17.2	17.2	19.6	19.6	16.9	16.9
	17.2	17.2	19.8	19.8	16.9	16.9
After mixing	22.0	19.0	16.5	14.8	18.8	17.4
	21.2	17.8	15.4	15.2	18.4	17.6
1 hr later	–	19.4	–	17.5	–	24.6
	–	20.0	–	15.8	–	26.7
2 hr later	–	14.5	–	21.2	–	38.4
	–	14.5	–	19.9	–	34.5
4 hr later	–	26.7	–	28.0	–	40.0
	–	24.1	–	30.3	–	39.0
After placement	26.6	30.0	16.3	31.2	20.6	31.8
	27.2	19.0	16.4	26.0	19.5	31.9
1 wk later	34.0	33.5	23.3	60.0	31.7	39.2
	35.3	35.0	24.1	65.0	29.4	35.7
1 mo later	27.6	42.1	40.3	31.0	32.0	38.8
	27.4	41.8	39.0	33.5	32.2	35.0
3 mo later	–[a]		–[b]	–[b]	–[b]	35.5
						38.2
6 mo later	–[a]		–[b]	–[b]	–[b]	–[b]
(b) At 77 F						
Original asphalt	2.65	2.65	1.93	1.93	2.31	2.31
	2.65	2.65	1.97	1.97	2.31	2.31
After mixing	4.35	2.70	1.73	1.70	2.49	2.72
	4.33	2.65	1.77	1.60	2.45	2.60
1 hr later	–	3.65	–	2.53	–	4.29
	–	3.75	–	2.26	–	4.52
2 hr later	–	2.92	–	2.92	–	8.61
	–	3.23	–	2.84	–	8.13
4 hr later	–	5.20	–	13.9	–	10.9
	–	4.95	–	14.6	–	10.5
After placement	5.53	6.00	1.93	6.22	2.54	7.35
	5.42	5.55	1.91	5.71	2.80	7.22
1 wk later	8.08	8.71	3.41	14.7	5.65	7.59
	8.20	8.75	3.39	13.9	5.68	7.74
1 mo later	7.85	12.2	8.50	8.42	6.64	8.87
	7.44	12.7	8.55	8.05	6.86	8.44
3 mo later	–[a]		10.4	7.52	8.48	6.88
			10.3	7.76	9.25	7.10
6 mo later	–[a]		10.3	14.6	7.80	10.0
			11.9	13.9	7.27	9.78
10 mo later	14.7	19.3	15.2	15.3	15.5	17.4
	15.9	18.8	15.3	13.9	16.1	18.8
(c) At 95 F						
Original asphalt	0.382	0.355	0.227	0.226	0.287	0.283
	0.382	0.355	0.216	0.216	0.287	0.283
After mixing	0.670	0.534	0.218	0.198	0.354	0.417
	0.659	0.531	0.218	0.190	0.368	0.417
1 hr later	–	0.690	–	0.302	–	0.662
	–	0.715	–	0.275	–	0.725
2 hr later	–	0.430	–	0.378	–	0.938
	–	0.495	–	0.362	–	0.930
4 hr later	–	0.968	–	2.86	–	1.77
	–	0.962	–	2.74	–	1.56
After placement	0.780	1.04	0.243	0.795	0.410	0.921
	0.798	1.06	0.268	0.499	0.437	0.892
1 wk later	1.30	1.79	0.374	2.43	0.857	0.996
	1.29	1.76	0.378	2.58	0.878	0.989
1 mo later	1.19	2.08	1.01	1.11	0.855	1.16
	1.15	2.20	1.01	1.05	0.892	1.16
3 mo later	–[a]		1.17	0.941	1.10	1.09
			1.18	0.961	1.07	1.11
6 mo later	–[a]		1.29	2.05	1.03	1.57
			1.27	1.96	1.02	1.56

[a]Not sampled.

[b]Samples too hard to test at 60 F.

TABLE 7

EFFECT OF DEPTH IN PAVEMENT ON ABSOLUTE VISCOSITY—TEN-MONTH OLD CORES AT 77 F

Depth Below Surface (in.)	Absolute Viscosity (megapoises)					
	Hunt Oil, Tuscaloosa		Shell Oil, Savannah		Shell Oil, Atlanta	
	Not Delayed	Delayed 4 Hr	Not Delayed	Delayed 4 Hr	Not Delayed	Delayed 4 Hr
1/8	19.8	21.3	22.4	14.2	22.8	23.9
	21.5	21.9	23.1	13.7	24.4	25.4
1/2	14.2	17.3	13.9	16.5	13.3	12.4
	15.1	16.8	13.4	13.9	13.1	14.8
7/8	12.7	17.4	13.7	15.2	13.3	16.3
	13.1	18.6	13.9	14.1	14.3	17.2
1 1/4	12.2	19.2	12.5	–	12.5	17.1
	14.0	20.0	12.7	–	12.5	17.7

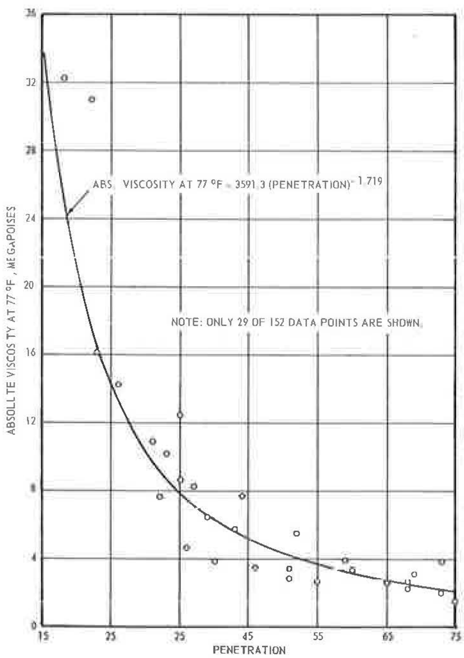

Figure 10. Absolute viscosity vs penetration.

haul period to about 70 percent of the average penetration of the samples taken just after mixing. Penetration test results for the basic test series are given in Table 8. Figure 11 plots the average penetration values vs duration of haul.

The asphalts showed an apparent gain in penetration during placement. This is consistent with the viscosity results, and as previously stated, was evidently due to heterogeneity of the mix in the truck bed and a failure to obtain truly representative field samples.

Effect of Asphalt Source. —Although the data were more variable, the penetration results showed the same pattern of hardening for the three sources as indicated by the microviscosity tests. The greatest loss in penetration during haul was shown by the Shell Oil, Atlanta sample, and the smallest loss by the Humble Oil, Charleston sample. Average penetration values decreased 24, 20, and 14, respectively, for the Shell Oil, Atlanta, Hunt Oil, Tuscaloosa, and Humble Oil, Charleston samples. Figure 12 plots the the penetration results for these three sources.

TABLE 8

PENETRATION VALUES VS TIME OF SAMPLING

Source	Series	Penetration Value[a]						
		Orig. Asphalt	After Mixing	1 Hr Later	2 Hr Later	4 Hr Later	After Placement	1 Wk Later
Humble Oil, Charleston	1100	75	69	61	55	64	53	49
Humble Oil, Charleston	1200	75	77	73	52	54	52	52
Hunt Oil, Tuscaloosa	2100	66	80	36	46	41	49	51
Hunt Oil, Tuscaloosa	2200	66	55	53	56	42	43	35
Hunt Oil, Tuscaloosa	B2100	66	69	59	44	51	57	57
Hunt Oil, Tuscaloosa	B2200	66	60	72	67	52	41	35
Shell Oil, Atlanta	4100	49	63	50	43	34	42	32
Shell Oil, Atlanta	4200	49	60	42	50	40	55	38
Shell Oil, Savannah	3100	49	60	55	52	24	37	25
Avg.		60	66	56	52	45	48	42

[a]Five sec., 100 gm, 77 F.

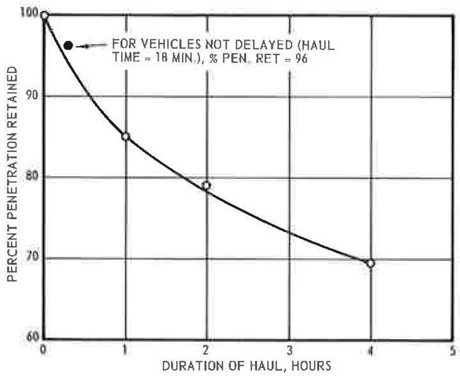

Figure 11. Average percent penetration retained vs haul duration.

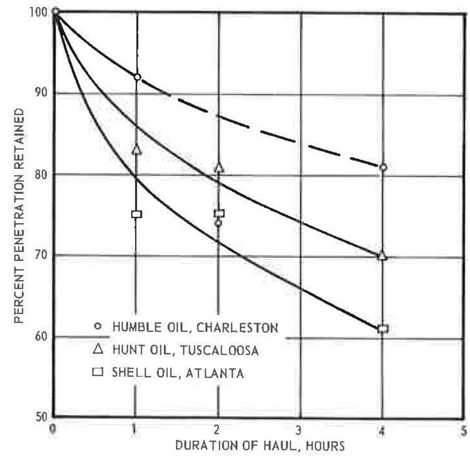

Figure 12. Percent penetration retained vs haul duration.

Effect of Mixing Temperature. —The penetration results failed to show a well-defined relationship between mixing temperature and asphalt hardening. However, it was found that about 60 percent of the asphalt samples mixed at high temperatures yielded smaller penetrations than those mixed at the lower temperatures.

Effect of Mix Heterogeneity. —The penetration results strengthened the viscosity findings with regard to mix heterogeneity. These values were consistent with earlier findings indicating that during haul more hardening occurs beneath the surface of a bituminous mix than at the surface. The penetration values for crust samples tended to be greater than for non-crust samples, especially after a 4-hr haul period (Table 9). Figure 13 shows averages of these data.

Table 10 gives penetration results for the special series of tests made to measure mix heterogeneity more precisely. These results strongly indicate, as previously suggested, that hardening is most severe at a depth of about 12 in. After a 4-hr haul period, the samples taken at 12-in. depths showed an average penetration of 43 compared with penetration values of 51 and 57 for samples taken at depths of 1 and 24 in., respectively.

Little difference occurred in the penetration results for samples taken from the front, middle, and back of the truck bed.

Effect of Field Sample Cooling Procedure. —The penetration results corroborated the results of the viscosity tests which showed that the manner in which field samples are cooled significantly influences the magnitude of hardening. Comparisons were made for the nine series of tests of air-cooled and water-quenched samples. In six of the nine comparisons, lower penetrations were noted for the air-cooled samples.

TABLE 9

PENETRATION VALUE OF SAMPLES FROM CRUST VS SAMPLES FROM 12 IN. BENEATH SURFACE

Source	Series	Penetration Value[a]					
		1 Hr Crust	After Mixing Non-Crust	2 Hr Crust	After Mixing Non-Crust	4 Hr Crust	After Mixing Non-Crust
Humble Oil, Charleston	1100	73	61	65	55	63	64
Humble Oil, Charleston	1200	87	73	68	52	68	54
Hunt Oil, Tuscaloosa	2100	40	36	40	46	58	41
Hunt Oil, Tuscaloosa	2200	48	53	46	56	43	42
Hunt Oil, Tuscaloosa	B2100	63	59	67	44	63	51
Hunt Oil, Tuscaloosa	B2200	64	72	58	67	58	52
Shell Oil, Atlanta	4100	39	50	50	43	40	34
Shell Oil, Atlanta	4200	53	42	50	50	48	40
Shell Oil, Savannah	3100	53	55	48	52	43	26
Avg.		58	56	55	52	54	45

[a]Five sec., 100 gm, 77 F.

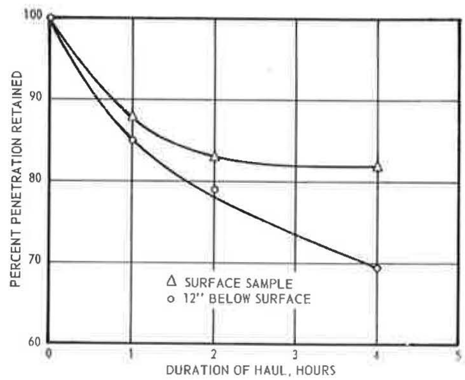

Figure 13. Percent penetration retained vs haul duration.

On the average, penetration values were 60 and 66 for the air-cooled and quenched samples, respectively (Table 11).

Control Tests. —Penetration control tests were made in an attempt to evaluate the effect of long haul times on the hardness of asphalt in pavements during the first year of pavement life. The penetration control test results, (Table 12) generally paralleled the viscosity control test results previously discussed.

Results of Ductility Tests

Ductility tests were conducted for samples taken just after mixing for comparison with those taken after a 4-hr haul

TABLE 10

PENETRATION VALUES—EFFECT OF LOCATION OF SAMPLE IN TRUCK BED

Depth of Sample (in.)	Penetration Value[a]			Avg.
	Back	Middle	Front	
1	45	54	54	51
12	47	39	43	43
24	58	56	57	57
Avg.	50	50	51	

[a]Five sec., 100 gm, 77 F.

TABLE 11

PENETRATION VALUE—EFFECT OF METHOD USED TO COOL FIELD SAMPLES

Series	Penetration Value[a]	
	Air Cooled	Quenched
1100	81	69
1200	97	77
2100	40	80
2200	50	55
B2100	59	69
B2200	71	60
4100	51	63
4200	42	60
3100	51	60
Avg.	60	66

[a]Five sec., 100 gm, 77 F.

TABLE 12

CONTROL TEST DATA—PENETRATION VS TIME OF SAMPLING

Time of Sampling	Penetration Value[a]					
	Hunt Oil, Tuscaloosa		Shell Oil, Savannah		Shell Oil, Atlanta	
	Not Delayed	Delayed 4 Hr	Not Delayed	Delayed 4 Hr	Not Delayed	Delayed 4 Hr
Original asphalt	66	66	49	49	49	49
After mixing	45	55	58	60	47	63
1 hr later	—	53	—	55	—	50
2 hr later	—	56	—	52	—	43
4 hr later	—	42	—	24	—	34
After placement	41	43	56	37	44	42
1 wk later	35	35	46	25	43	32
1 mo later	37	35	39	37	41	38
3 mo later	—[a]		35	34	44	40
6 mo later	—[b]		35	34	42	39

[a]Five sec., 100 gm, 77 F.
[b]Not sampled.

period following the general procedures of ASTM D113-44. However, the procedure was modified in two important respects. First, the rate of pull was 2 cm/min rather than the specified 1 cm/min. Second, the tests were conducted at 48 ± 5 F. Although these modifications lessen the value of the ductility results for comparison and correlation with other research, the tests were of significant value in detecting changes in ductility during the 4-hr haul period. A statistical t-test comparing the ductilities of the asphalt before and after haul was significant at the 0.1 percent level, indicating with a high degree of certainty that there was a decrease in ductility during the haul period.

Results of the ductility tests are given in Table 13. Although the extent of loss in ductility varies with the asphalt source, a loss in ductility was indicated during the haul period for all asphalt sources tested. Virtually no loss in ductility was noted for the control test samples hauled less than 30 min.

Infrared Spectroscopy Results

To provide some clues concerning the probable chemical changes occurring during the hauling process, a study was made of the infrared absorption characteristic of the asphalts used in this investigation. For each asphalt source, spectrograms were made for the original asphalt, and for samples taken immediately after mixing and after a 4-hr haul. Asphalt films, 76 ± 3 microns in thickness, were irradiated between sodium chloride plates by use of a Beckman model IR 8 spectrophotometer.

Changes in absorption were most noticeable at wave lengths of 2.9, 5.9, 9.7 microns, identified by previous researchers, respectively, as the hydroxyl, carbonyl, and the carbon-oxygen-carbon band (12). The wave length of 9.7 microns may also be indications of oxygen, nitrogen, or sulfur linkages (13).

The results strengthen the widely held belief that the chemical reaction contributing most to asphalt degradation is that of oxidation (14).

Effects of Long-Period Storage of Hot Asphalt Mixes

A special study was made to evaluate the effects of storing hot bituminous mixes for 24 hr or longer. In an attempt to simulate this effect, a 10-gal bucket was filled with hot bituminous mix obtained from a nearby mixing plant. This material was immediately covered, transported to the laboratory, and placed in a 250 F oven. Another 10-gal bucket was concurrently filled with the mix, covered, and taken to the laboratory where it was maintained at normal air temperatures. Subsamples were taken from the buckets at the following times: 4, 8, 24, 96 and 168 hr after mixing.

The asphalt was recovered from the subsamples by the Abson technique, and the hardness was measured by penetration and microviscosity tests.

After a period of 96 hr, viscosities of the asphalt maintained at 250 F were as much as 24 times as great as that maintained at normal air temperatures. (This varied with viscosity test temperature.) Figure 14 plots these results for a viscosity test temperature of 77 F.

Practically no change in penetration occurred in the samples stored at normal air temperatures, whereas the penetration of the asphalt stored in the oven decreased in an exponential pattern from 63 to 15 during the 7-day period (Fig. 15).

How well these tests simulate the storage of bituminous mixes in insulated bins is a matter of conjecture. However, the results suggest that it would be unwise to

TABLE 13

LOSS IN DUCTILITY DURING FOUR-HOUR HAUL PERIOD

Sample Series	Ductility[a]	
	After Mixing	After Placement
1100	127.00	8.00
1200	46.25	0.00
2100	14.75	6.50
2100-W[b]	8.50	7.50
2200	9.00	5.30
B2100	13.00	6.75
B2200	9.25	7.75
4100	18.00	8.50
4100-W[b]	11.25	10.50
4200	22.25	16.25
3100	33.50	7.75
3100-W[b]	16.30	

[a]Ductility at 48 F with rate of pull = 2 cm/min.
[b]Control tests, average haul period = 18 min.

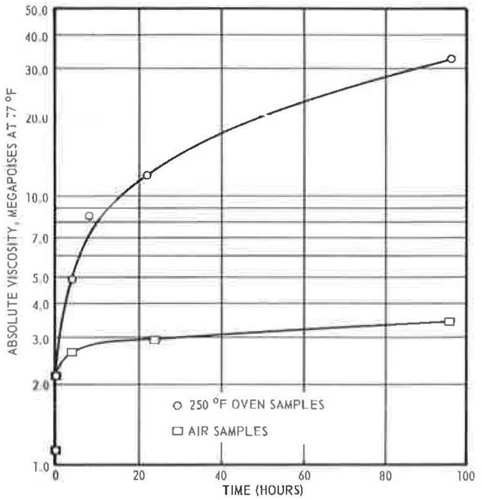

Figure 14. Absolute viscosity vs storage time.

use mixes stored in this manner lacking evidence that extreme hardening does not occur. Similarly, the use of a mix kept for long periods in a truck bed would also be highly questionable.

EVALUATION OF RESULTS

It would be desirable to evaluate the seriousness of hardening caused during the haul process in terms of how much, if any, such hardening would be likely to decrease the pavement life. There are several reasons why such a direct and simple evaluation is not possible, all of which relate to our limited understanding of the hardening process and its influence on pavement behavior. The first reason is the scarcity of available data for asphalt hardening under service conditions. Although several excellent research efforts have been made in this area, little viscosity information is yet available for asphalts taken from pavements during the latter part of a pavement's service life. Furthermore, available hardening data are complicated by differences in environmental conditions such as traffic, climate, and mix gradation.

A second complicating factor which precludes a straightforward evaluation of the results of this study is the generally accepted knowledge that asphalts from different crude origin tend to vary widely in hardening susceptibility.

Finally, the durability of a pavement cannot be stated simply in terms of accepted measures of asphalt hardness such as viscosity or penetration. In the words of Simpson, et al. (8):

> It is not possible to set an upper limit of asphalt viscosity beyond which failure under traffic will occur because the deflection of the road determines the magnitude of tensile stress developed which in turn leads to failure.

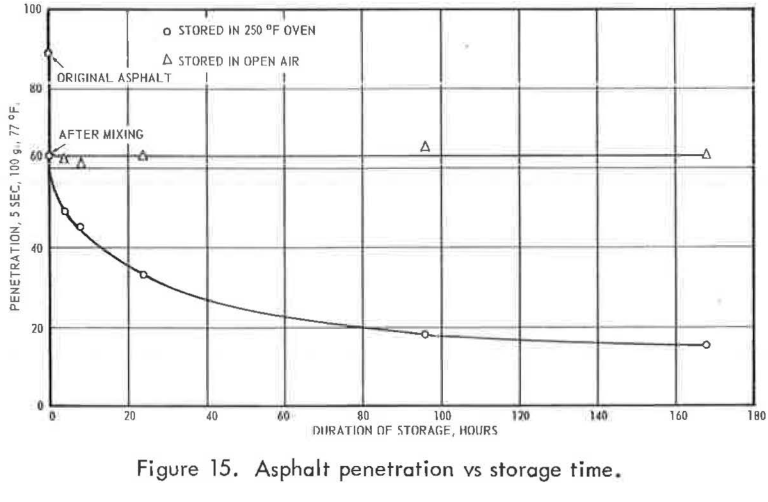

Figure 15. Asphalt penetration vs storage time.

Despite the inability to evaluate in quantitative terms the long-term effects of the hardening during haul, most asphalt technologists would probably agree that any hardening, however small, must be regarded as undesirable. A fuller appreciation of the seriousness of this effect must await further research. In the meantime, a prudent course of action would be to keep the haul time as short as possible, consistent with economy and practicality.

CONCLUSIONS

1. Asphalt in hot bituminous mixes significantly hardens during haul, the extent of hardening varying with duration of haul and asphalt source.
2. Asphalt hardening during haul is both a time and temperature phenomenon which is affected by variations in mix temperature existing during the haul period. This is believed to be the principal reason that the viscosity of asphalt sampled during haul was significantly related to position and depth from which the sample was taken from the truck bed.
3. Asphalt sampled from the uppermost 1-in. crust in a truck bed hardens less during haul then the asphalt sampled from the hotter mix within the mass.
4. The results of this research indicate that high mixing temperatures may result in slight increases in asphalt hardening even though the mixing temperature is maintained within the 300 to 320 F range.
5. The hardening that occurs during haul may be evident in cores taken from pavements as much as 10 months after placement.
6. Viscosity of asphalt extracted from pavement cores decreases nonlinearly with pavement depth, and the viscosity of the asphalt at the pavement surface may be 50 percent or more greater than that of the asphalt ½ in. below the pavement surface.
7. Asphalt ductility decreases during the haul process.
8. The changes occurring in asphalt during the haul process are most noticeably reflected by changes on an infrared spectrogram at wave lengths of 2.9, 5.9, and 9.7 microns.

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REFERENCES

1. Bateman, J. H., and Lehmann, H. L. The Effect of the Mixing Temperature on the Physical Properties on an Oil Asphalt in a Sheet Asphalt Mixture. ASTM Proc., 29, Pt. II, 1929.
2. Brannon, T. W. Penetration Tests Being Made on Recovered Asphalt. Civ. Eng., Vol. 7, No. 4, Apr. 1937.
3. Bright, Richard, and Reynolds, E. T. Effect of Mixing Temperature on Hardening of Asphaltic Binder in Hot Bituminous Concrete. Highway Research Board Bull. 333, pp. 20-38, 1962.
4. Gotolski, William H. High Temperature Effects on Bituminous Mixes. Jour. of Highway Div., ASCE, Vol. 86, No. HW3, Sept. 1960.
5. Disc. of High Temperature Effects on Bituminous Mixes. Jour. of Highway Div., ASCE, Vol. 87, No. HW1, March 1961.
6. White, Oscar A. Durability of Asphalt in Hot-Mixed Asphaltic Concrete Pavement. Highway Research Board Proc., Vol. 27, 1947.
7. Parr, W. K., and Serafin, P. J. Four Years Test Trends On The Michigan Bituminous Experimental Road. AAPT Proc., Vol. 28, 1959.

8. Simpson, W. C., Griffin, R. L., and Miles, T. K. Correlation of the Microfilm Durability Test With Field Hardening Observed in the Zaca-Wigmore Experimental Project. ASTM Spec. Tech. Pub. No. 277, p. 52, 1959.
9. Griffin, R. L., Miles, T. K., Penther, C. J., and Simpson, W. C. Sliding Plate Microviscometer for Rapid Measurement of Asphalt Viscosity in Absolute Units. ASTM Spec. Tech. Pub. No. 212, p. 36, 1956.
10. Pauls, J. T., and Halstead, W. J. Progressive Alterations in a Sheet Asphalt Pavement Over a Long Period of Time. Public Roads, Vol. 30, 1959.
11. Coons, Richard. Research for Master's thesis, Georgia Inst. of Tech., unpubl.
12. Wright, J. R., Campbell, P. G., and Fridinger, T. L. Relationship Between Intensity of Carbon Arc and Asphalt Oxidation. ASTM Spec. Tech. Pub. No. 347, 1963.
13. Schweyer, H. E. Spectral Absorption of Asphaltic Materials. Analytical Chemistry. Vol. 30, No. 2, Feb. 1958.
14. Campbell, P. G., Wright, J. R., and Bowman, P. B. The Effect of Temperature and Humidity on the Oxidation of Air-Blown Asphalts. Mat. Res. and Stds., Vol. 2, No. 12, Dec. 1962.

A Program for Smoother Roads

GLENN ANSCHUTZ, Engineer of Photronics,
GLENN O. McDONALD, II, Programmer, and
JACK L. CONKLIN, Methods Analyst, State Highway Commission of Kansas, Topeka

The demands of a traveling public for smooth, all-weather roads have produced a number of plans for the improvement of the riding surfaces of new or reworked highways. This paper is concerned with this problem; it discusses asphaltic laydown equipment with string-line sensoring devices for surface controls and the use of the electronic computer for the mathematical computations required for such controls. The authors have attempted to provide sufficient information relating to program control and reasoning and the mechanics of input and output data so that the reader can use the body of the program involved and produce other programs which may differ in local detail. A sample problem is included to aid in the presentation of the approaches used.

•REALIZING that the traveling public dramatically and quickly reacts to rough roads, highway engineers have, through the years, concerned themselves with old and new road surfaces to palliate abrupt vertical curvature or broken pavements. An economical and practical approach to the revitalization of older surfaces which has been used by many highway departments is the application of asphaltic surfaces of several types. In addition, to provide the smoothest surfaces possible on such construction and on final layers of new construction, a computed grade line has been used to control this surfacing. In the field, such a grade is represented by a surveyed string line. A traveling string line has also been used with much success toward the achievement of good driving surfaces.

For several years the Kansas State Highway Commission has been concerned with the rebuilding of deteriorated wearing surfaces on bituminous and concrete highways which have sufficient width and base for the continuation of their use. By placing a thin layer of asphaltic concrete on these surfaces, it has been possible to continue their use as desirable highways. For many years the material used for this construction was road-mixed asphalt, placed with a blade. As better equipment became available, this method was discarded in favor of central mixing plants and asphalt laydown machines. This technique was in use until the early 1960's when Frank Drake invented a device which made it possible to construct a controlled, rather than a uniform, thickness overlay. By controlling the overlay thickness, a more uniform and smoother riding surface has been obtained.

This intricate control is now required equipment on all laydown machines used on Kansas highways. A sensor is used on a string line which has been preset to the elevation of the surface to be constructed. As the device senses a closure or departure of the string line from the existing surface, the thickness of the material being placed on the road is increased or decreased accordingly. The string line, which is usually placed along the centerline of the roadway, must be set by the ordinates or elevations of a predetermined reference grade.

The establishment of a reference grade by the tangent and vertical curve method is not used because the cost of the amount of surfacing material required would be prohibitive. Instead, the existing nonuniform grade is modified to establish a sufficiently

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improved grade while maintaining the general existing grade trend. This grade can be described by any of several methods, and can be computed either manually or by electronic computer. The Kansas Highway Commission uses a computer to establish the modified grade line, which is described both by elevations and fill ordinates above the original grade line.

PRELIMINARY SURVEY

The complete procedure for the design and construction of an improved wearing surface consists of an original survey, computations for a modified grade, and the actual construction of the project.

The preliminary survey provides the data or information by which an existing grade line can be slightly adjusted so that its characteristics are smooth, and when it is used for spreading asphalt a surface of good riding qualities is attained. The primary purpose of the survey and grade-line adjustment is to place asphaltic mixtures in the varying thicknesses that tend to improve the ridability of the finished surface.

Filling sags, cutting humps, and building ramps on either side of abrupt vertical excurvatures are the tools available to straighten or improve the grade line. To utilize these tools, knowledge of the location, character, and magnitude of each irregularity is required, and this information is obtained in a preliminary survey. This can be illustrated by plotting the existing grade-line elevations to a distorted scale on profile paper. The scale, 1 in. = 25 ft horizontally and 1 in. = 0.5 ft vertically, magnifies the irregularities so that they are easily detected and may be treated or eliminated by manual straightening with a flexible curve or straightedge. The field notes are generally submitted to the design department for grade-line adjustment by the electronic computer.

The vertical measurement is the most important field measurement. The actual locating of the original profile points does not require a great deal of accuracy or detail work. If existing blue top stakes, right-of-way stakes, pavement edge or any other set line is available, a transit is not needed to establish the line to be profiled. When such a line is available, chaining and cross-chaining works very well in establishing the line. Much can be gained by careful work when taking level readings, and little is to be gained by taking excessive time to make exact locations of each profile point.

A series of profile points should be placed along the centerline of the existing roadway surface with one located at each station and each 25-ft interval between stations. Each profile point should be marked with such permanence that it may be readily located at the expected time of construction. A spike and washer has been effectively used for this purpose. Profile points located at stations should be distinguished from those at the 25-ft intervals so that chances for error in placing the thickness values are held to a minimum.

A profile should be run and the data entered in a standard loose-leaf field book. The levels should be based on an assumed elevation and carried through the entire length of the project without making the usual corrections at turn points and bench marks. A correction in the levels at any point in the profile causes a vertical equation which imposes a wave in the adjusted grade line. The height of instrument may be checked at a bench mark for approximate elevation, but should not be corrected. The height of instrument for an instrument setup can be checked by taking a check rod reading on the last profile point of the previous instrument setup.

Existing crown cross-sections at various points along the centerline are needed for the computation of the adjusted grade line and the calculation of the estimate of quantities of material for the leveling course. The distance between locations of cross-sections should not exceed 500 ft. A cross-section should be taken immediately before the beginning of superelevation of each curve and immediately after the end of superelevation. A minimum of three sections should be taken within the fully superelevated portion of each horizontal curve. The sections should be so spaced and of sufficient number to give representative information on each curve.

The cross-section should contain five points of rod readings: centerline, the quarter point (6 ft right and left), and the pavement edges. It is not intended that a lot of time be spent on the cross-section; however, it is important that extreme care be exercised when selecting the place to take the rod reading.

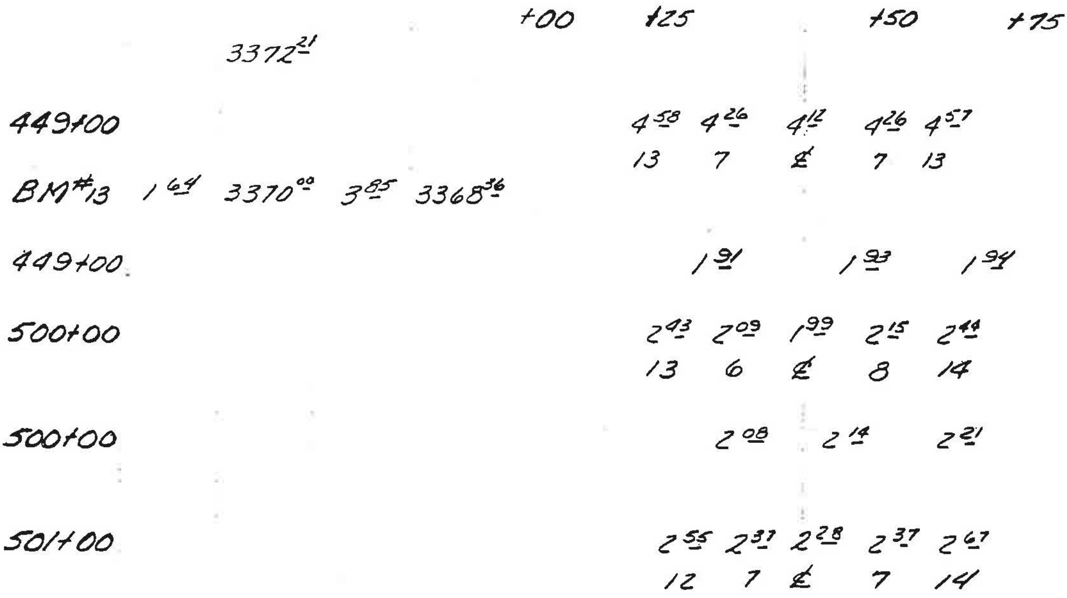


Figure 1. Field notes.

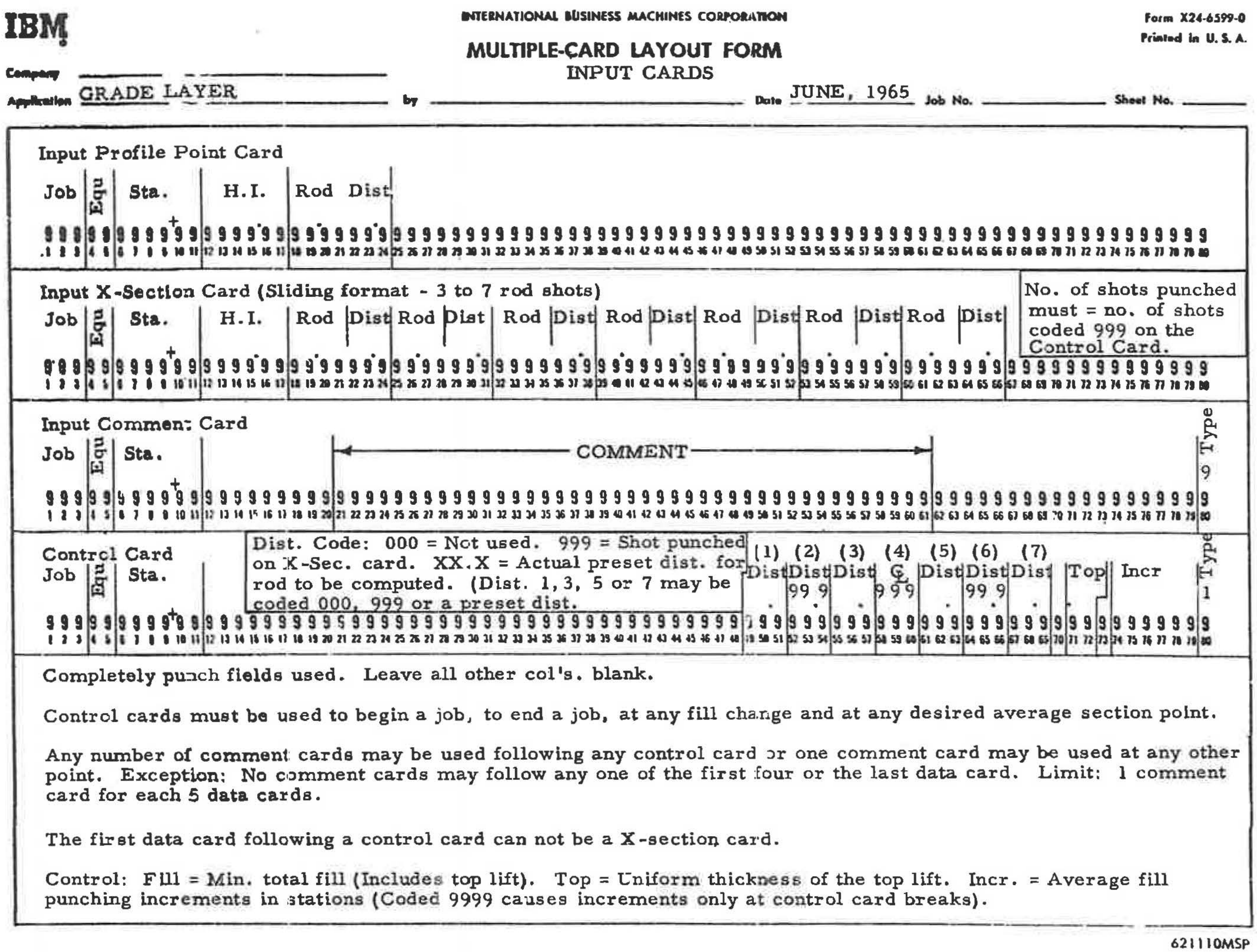
Rod readings for profiling and surface cross-sectioning should be read to the nearest one-hundredth of a foot. Careful reading of the rod should be exercised; however, it is not necessary to wave the rod and use the precautions necessary in making turning point and bench mark readings.

The procedures described follow quite closely the specifications used by the Kansas Highway Commission. They are outlined in detail in the construction manual for overlay operations. Other specifications and procedures may be developed which produce similar results. However, if electronic computation is to be utilized, a uniform surveying procedure is necessary to produce accurate and economical computations. Field notes should also follow a set pattern. Consistency of format in original data provides for easy review of such data, reduces the possibility of error when key punching the data and aids in the writing of, and in the compliance with, operating instructions. A typical set of field notes used by Kansas is shown in Figure 1.

COMPUTATIONS FOR MODIFIED GRADE

On completion of the survey, the field notes are generally transmitted to the design department, and a designing unit is assigned to the project. A preliminary check of the notes is made at this time. The field notes are then delivered to the computer section for processing. Various card formats can be developed for the field information. The Kansas program is designed around the card formats shown in Figure 2. After the data have been placed into proper format, they are ready for computer processing.

The Kansas program was designed for an IBM 1620 60K computer. To allow maximum flexibility, the processing is developed by three separate computer runs. The first pass performs an edit of the data. This pass checks for abrupt changes and irregularities in the centerline profile, for any major deviation in similarity of each cross-section from the previous cross-section processed, and the stationing for incorrect sequence. As these conditions are detected, they are recorded on a typewriter output. The results of this pass are manually checked to determine if noted irregularities are actual errors or existing road surface conditions. Incorrect station sequence is also checked in this pass and noted by typewriter output. Errors of this nature are corrected and the average cross-section run is made. Average cross-sections may be computed over any specified segment of roadway. The averages are retained on output punched cards.



IBM

INTERNATIONAL BUSINESS MACHINES CORPORATION

Form X24-6599-0
Printed in U. S. A.

MULTIPLE-CARD LAYOUT FORM

INPUT CARDS

Company ______ Application GRADE LAYER by ______ Date JUNE, 1965 Job No. ______ Sheet No. ______

Input Profile Point Card

Job | Equ | Sta. | H.I. | Rod Dist

Input X-Section Card (Sliding format - 3 to 7 rod shots)

Job | Equ | Sta. | H.I. | Rod | Dist | Rod | Dist | Rod | Dist | Rod | Dist | Rod | Dist | Rod | Dist | Rod | Dist

No. of shots punched must = no. of shots coded 999 on the Control Card.

Input Comment Card

Job | Equ | Sta. | COMMENT | Type 9

Control Card

Job | Equ | Sta.

Dist. Code: 000 = Not used. 999 = Shot punched on X-Sec. card. XX.X = Actual preset dist. for rod to be computed. (Dist. 1, 3, 5 or 7 may be coded 000, 999 or a preset dist.

(1) Dist | (2) Dist 99.9 | (3) Dist | (4) ℄ 999 | (5) Dist | (6) Dist 99.9 | (7) Dist | Top | Incr | Type 1

Completely punch fields used. Leave all other col's. blank.

Control cards must be used to begin a job, to end a job, at any fill change and at any desired average section point.

Any number of comment cards may be used following any control card or one comment card may be used at any other point. Exception: No comment cards may follow any one of the first four or the last data card. Limit: 1 comment card for each 5 data cards.

The first data card following a control card can not be a X-section card.

Control: Fill = Min. total fill (Includes top lift). Top = Uniform thickness of the top lift. Incr. = Average fill punching increments in stations (Coded 9999 causes increments only at control card breaks).

621110MSP

Figure 2. Kansas program.

The use of an IBM 407 accounting machine provides a listing in the form of plotted cross-sections. The original data for cross-sections have been retained in a format of rod readings, distances from a centerline and controlling heights of instrument. The average cross-sections produced by the second pass are also in the input format. A listing of this output data is produced along with the plotted cross-section. The high point of the average cross-section replaces the height of instrument reading and carries a rod reading of 0 ft. Each average cross-section is computed from the original cross-sections within a segment of the project as controlled by the designer. The output data are returned to the designer for evaluation. Additional runs can be made to improve the project, or if errors persist.

After satisfactory results have been obtained from the first two runs, the grade layer run is made. A fill ordinate is computed and punched out for each profile point and centerline cross-section point. Each fill ordinate is computed to the nearest one-hundredth of a foot and is accompanied by the elevation of the modified grade and the algebraic difference in grade at that point. If the designer specifies that the leveling course be constructed in two layers, the fill ordinates of the bottom layer and the overall fill ordinates are computed. All leveling is accomplished in the bottom layer while the top layer maintains a uniform thickness. At each change of minimum fill as specified by the designer, an average centerline fill ordinate is computed. The final output is listed on a 407 accounting machine and returned to the designer.

The designer uses the average cross-sections in conjunction with the average fill ordinates to compute asphalt and aggregate quantities for the project. Incorporation of these quantities with the grade-layer listing completes the overlay portion of the design plans.

THEORY OF COMPUTATION

Computer programs can be developed using a number of concepts. The basic theory of computation used in the program discussed may be outlined as follows.

The grade-layer program uses a five-point method to compute a modified grade. Five consecutive profile points of the existing grade are analyzed and used to compute a point on the modified grade line. After the computations are made for each point, the grade line is advanced one point within the computer and the process is repeated. Thus each point on the existing grade occupies five consecutive positions for analysis as it is advanced through the computer. The computer uses several different methods for the computation of each point on the modified grade. The configuration of the five points of the existing grade and the average departure of the modified grade from the existing grade up to the point being computed, determine the method to be used. For the purpose of explanation, the five points and all relating data are referred to the notations shown in Figure 3.

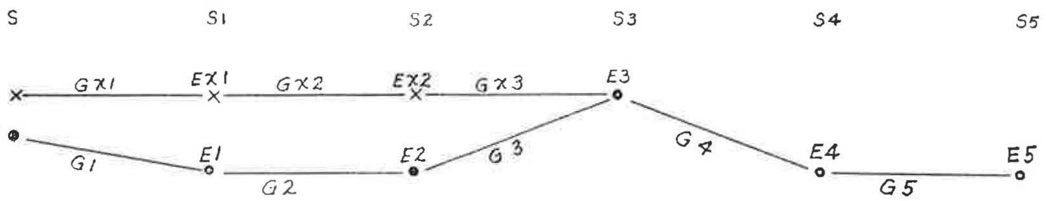


Figure 3.

DEFINITIONS

E1 through E5: original elevations of 5 points being processed;
EX1 through EX2: modified elevations of points 1 and 2;
G1 through G5: grades approaching original elevations of points 1 through 5;
GX1 and GX2: grades approaching modified elevations of points 1 and 2;

GX3: grade between modified elevation of point 2 and original elevation at point 3;
D1 through D4: algebraic grade differences between grades to original elevations at points 1 through 4;
DX1 through DX3: grade differences between grades to modified elevations at points 1 through 3;
S1 through S5: stations of 5 points being processed; and
S: station of point just before S1.

The following figures and the analysis of their configurations show the methods used for the computation of the modified grade line.

CASE I

If the analysis of the original elevations at stations S1 through S5 does not indicate a crest condition, a line is constructed from E3 to E5 and EY4 is established on the line at station S4 (Fig. 4). If EY4 is not lower than E4, a line is then constructed from EX1 to EY4 and EX2 is established on that line at station S2.

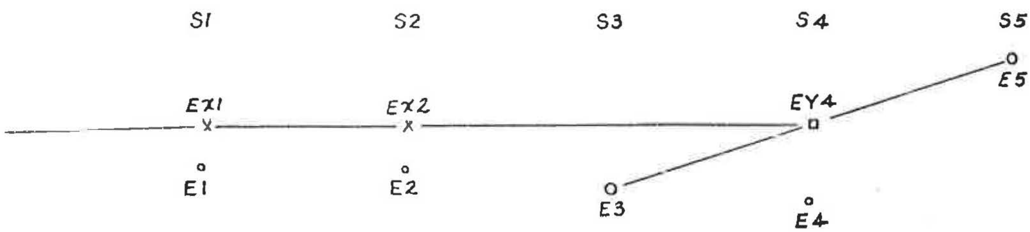


Figure 4.

If EY4 is lower than E4 (Fig. 5), a line is constructed from EX1 to E4, and EX2 is established on that line at station S2.

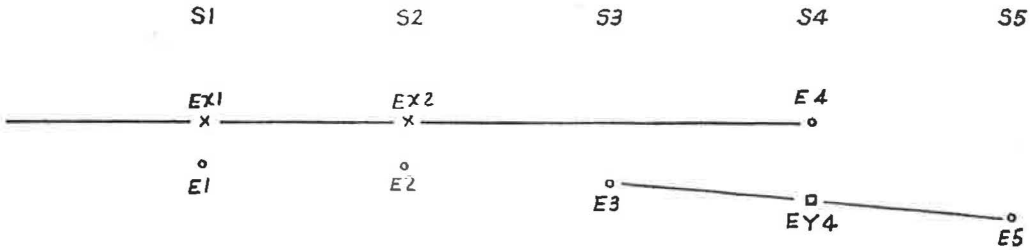


Figure 5.

After the foregoing computation of EY4 has been made, the grade G3 is computed between E2 and E3 (Fig. 6). The grade GY4 is then computed using E3 and the lower of EY4 and E4. The grade difference DY3 is computed at station S3 using G3 and GY4. If DY3 is negative, the previous EX2 is discarded. A line is then constructed from EX1 to E3 and EX2 is established on this line at station S2. If DY3 is positive, EX2 is not recomputed.

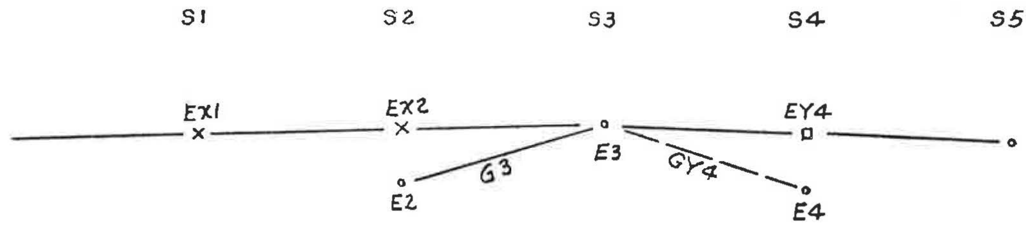


Figure 6.

CASE II

On the completion of case I, the original elevations at stations S1 through S4 are analyzed for a sag condition. If the grades connecting each of these points are increasing in a positive direction, the condition is interpreted as a sag. Therefore, to override the excessive fill that would result from the normal leveling computed in case I, the modified grade is lowered to follow closely the trend of the vertical curve of the existing points. Thus EX2 is modified by the following formula:

$$EX2 = E2 + 0.33333\ (EX2 - E2)$$

If a sag condition exists only at stations S2, S3, and S4 but not at S1, the entrance of a sag condition is indicated and EX2 is modified by a lesser amount than previously shown for a full sag condition:

$$EX2 = E2 + 0.50000\ (EX2 - E2)$$

If a sag condition exists at stations S1, S2, and S3 only, the exit of a sag condition is indicated. EX2 is modified by the following formula:

$$EX2 = E2 + 0.66667\ (EX2 - E2)$$

If the original grade does not meet any of the foregoing sag conditions, EX2 is altered at this juncture.

CASE III

If, on the analysis of the five original points, a crest condition exists, this case is used in place of case I and case II. To determine this condition, the original grade differences D1 through D4 are computed at stations S1 through S4. If all differences are negative, the grade condition is interpreted as a crest and the average grade difference DX of these points is computed by the formula:

$$DX = \frac{D1\ (S2-S) + D2\ (S3-S1) + D3\ (S4-S2) + D4\ (S5-S3)}{2\,(S5-S)}$$

After the foregoing computation has been made, EX2 is computed so that the modified grade difference DX2 at station S2 is equal to DX.

MATERIAL SALVAGING PROCESS

If processing has been executed according to cases I and II, the following analysis and computations should be made. The average vertical difference V between EX2 and E2 for all previous points is computed. EX2 is then modified by the following method:

If $V < 0.020$ ft: $EX2 = EX2 - 0.010$ ft
If $0.020 \text{ ft} < V < 0.030$ ft: $EX2 = EX2 - 0.014$ ft
If $0.030 \text{ ft} < V < 0.040$ ft: $EX2 = EX2 - 0.018$ ft
If $V > 0.040$ ft: $EX2 = EX2 - 0.022$ ft

The reason for this step is to eliminate an excessive difference between the original grade line and the modified grade line. In the case of a relatively uniform original grade where the majority of the grades are under 4 percent and the vertical curves over 400 ft long, a minimum of correction has to be made on the modified grade to keep its grade trend near that of the original grade. If the original grade line follows closely to the ground line in rolling terrain, the modification tendency is toward a flatter grade that is impractical to construct using asphalt surfacing materials. If this condition exists, it is necessary to place further control on the modified grade so that it will closely follow the trend of the original grade. Regardless of the grade under consideration, it is not the intent of this program to produce a radically different grade. The purpose of this program is to correct minor irregularities so that the best riding surface possible may be constructed from the amount of surfacing material allowed.

At the completion of the computation of EX2 in any of the processes executed by this program, EX2 is checked against E2. If the elevation of EX2 becomes less than that of E2, EX2 is replaced by E2. The minimum fill ordinate specified by the designer is then added to EX2 and the fill ordinate to be used for construction is computed by the formula:

$$\text{Fill} = \text{EX2} - \text{E2}$$

THEORY OF DESIGN

On the assignment of a project, the design department checks the field notes for mathematical errors. Input comment and cross-section control forms are coded and along with the field notes are submitted to the computer section for key punching and the edit run on the computer.

The edit run listing is checked and the errors are corrected on input profile point and/or input cross-section forms. The project is then broken into non-superelevated segments which connect superelevated curves and the superelevated curve segments. Cross-section control forms are coded for each segment of roadway. The correction forms and the control forms are submitted to the computer section for key punching and the cross-section computer run.

Average cross-sections produced by the cross-section computer run are analyzed for crown condition and the minimum centerline overlay thickness is determined. This is accomplished by applying a predetermined template to the average sections so that no point on the sections is at an elevation any closer than 0.08 ft to template elevation. The centerline ordinate is then scaled and 0.02 ft is added for compaction. This value is coded on the control forms and resubmitted to the computer section for key punching and the leveling course run on the computer.

At this time, quantities are computed. The fill ordinates are incorporated into the plans, completing the design stage of the program.

CONCLUSIONS

The program introduced in this paper is only a part of the total process involved in producing smoother riding surfaces. However, this program, which uses an electronic computer, has been a key element in smoother roads for Kansas. The design time has decreased measurably; a year's resurfacing plans, which in the past required one design unit six months to complete, is now completed in one or two months. The values computed for material quantities have also been improved measurably.

The final judge of the program is the road user. Roughometer tests have proven that resurfacing projects using the string-line method are smoother in most every case. The total roughometer distance measured in a resurfacing year has shown a marked improvement in surface smoothness. There is no question that smoother roads are obtained and with the computer program to reduce design costs, "a program for smoother roads" is here to stay.

MODEL PROBLEM

To aid the user of such a process, a short sample run is included in the Appendix. The sample includes cross-section notes, first input cards, corrections as indicated by the output of the computer typewriter, corrected cards, second output, final cross-sections, and a sample of listed output. In addition to the sample problem, a listing of operator instructions and a program flow chart are also included.

Appendix

Example of Centerline Profile and Cross Section Notes:

CENTERLINE PROFILE

Date ________ Instr. ________

Weather ________ Rod ________

Sta	+Rod	HI	-Rod	Elev.	+00	+25	+50	+75
BM	6.49	106.49		100.00	Note: No rods in this area.			
200					6.30	6.33	6.10	5.94
201					5.68	5.43	5.16	4.96
202			Note: All rods in this area must go with the above HI.		4.77	4.58		
+50 X-Section					$\frac{4.34}{12L}$ $\frac{4.35}{9}$ $\frac{4.33}{4}$ $\frac{4.31}{CL}$ $\frac{4.28}{2}$ $\frac{4.32}{10}$ $\frac{4.38}{12R}$			
202								3.93
203					3.50	3.08	2.72	2.35
204					2.02*	1.75#		
TP	7.58	112.32	1.75#	104.74	Note: No rods in this area.			
TPck	7.84		2.02*	104.47				
204							7.26	6.90
206					6.49	6.09	5.72	5.44
207 X-Section			Note: All rods in this area must go with the above HI.		$\frac{5.18}{12L}$ $\frac{5.08}{2}$ $\frac{5.07}{CL}$ $\frac{5.11}{3}$ $\frac{5.18}{12R}$			
207						4.72	4.40	4.18

An Example Sheet supplied field personnel for the taking of field notes.

Field Notes for Sample Problem

page 1

					+00 +25 +50 +75
		3372^{21}			
449+00					4^{58} 4^{26} 4^{12} 4^{26} 4^{57}
					13 7 ℄ 7 13
BM#13	1^{64}	3370^{00}	3^{85}	3368^{36}	3368^{34}
449+00					1^{91} 1^{93} 1^{94}
500+00					2^{43} 2^{09} 1^{99} 2^{15} 2^{44}
					13 6 ℄ 8 14
500+00					2^{08} 2^{14} 2^{21}
501+00					2^{55} 2^{37} 2^{28} $\cancel{3^{37}}$ 2^{37} 2^{67}
					12 7 ℄ $\cancel{2}$ 7 14
501+00					2^{36} 2^{51} 2^{65}
502+00					2^{99} 2^{89} 2^{82} 2^{91} 3^{19}
					12 7 ℄ 8 14
502+00					2^{98} 3^{12} 3^{30}
503+00					3^{76} 3^{61} 3^{49} 3^{59} 3^{86}
					12 8 ℄ 7 13
503+00					3^{71} 3^{91} 4^{13}
⊙	2^{60}	3368^{28}	4^{32}	3365^{68}	℄ rail @ 504+00
504+00					2^{80} 2^{65} $\cancel{3^{60}}$ 2^{60} 2^{69} 2^{85}
					13 6 ℄ 6 12
504+00					2^{80} 2^{96} 3^{10}

These corrections were made to the input card after analyzing the Typewriter Error Message output from Pass 1.

page 2

					+00	+25		+50		+75
		3368^{28}								
505+00					3^{50}	3^{30}	3^{23}	3^{28}	3^{41}	
					13	5	℄	5	12	
505+00						3^{39}		3^{52}		
+68					3^{90}	3^{74}	3^{66}	3^{74}	3^{94}	
					13	7	℄	5	12	
506+00					4^{07}	3^{95}	3^{88}	3^{96}	4^{13}	
					14	7	℄	5	11	
506+00						4^{09}				
506+50					4^{52}	4^{36}	4^{27}	4^{39}	4^{74}	
					14	8	℄	5	12	
506+00										4^{41}
507+00					4^{80}	4^{63}	4^{54}	4^{67}	5^{00}	
					14	8	℄	5	11	
507+00						4^{66}		4^{82}		4^{93}
BM#14	2^{89}	3365^{80}	5^{37}	3362^{91}	3362^{90}					
508+00					2^{76}	2^{57}	2^{54}	2^{74}	2^{96}	
					14	7	℄	7	12	
508+00						2^{60}		2^{65}		2^{74}
509+00					3^{12}	2^{83}	2^{79}	2^{99}	3^{40}	
					13	7	℄	6	12	
509+00						2^{86}		2^{94}		3^{08}

page 3

+00 +25 +50 +75

		3365^{80}							
510+00					3^{38}	3^{22}	3^{18}	3^{34}	3^{25}
					13	7	¢	5	12
510+00					3^{28}			3^{38}	3^{54}
511+00					3^{80}	3^{71}	3^{70}	3^{85}	4^{20}
					13	8	¢	7	11
511+00					3^{87}			4^{08}	4^{23}
512+00					4^{53}	4^{37}	4^{34}	4^{45}	4^{74}
					13	7	¢	5	11
512+00					4^{45}			4^{59}	4^{74}
⊙	1^{53}	3362^{46}	4^{87}	3360^{93} ¢ nail 513+00					
513+00					1^{77}	1^{58}	1^{53}	1^{63}	1^{79}
					13	6	¢	6	11
513+00					1^{63}			1^{78}	1^{88}
514+00					2^{19}	2^{02}	1^{99}	2^{10}	2^{35}
					12	6	¢	7	12

GRADE LAYER CONTROL CARDS

Route		Project				Type	Work Class	Acc.
No.	Sec.	Class	No.	Sec	Agr. No.			

FOR COMPUTER SECTION

Job No. ______

Date Received ______

Estimated Completion Date ______

Request Received By ______

To be coded completely by requesting department

TEMPLATE CONTROL DISTANCES

JOB	EQU	STA	D 1	D 2	D 3	C	D 5	D 6	D 7	Leveling Course Uncomp.	INCR	Total Comp.	
1 2 3	4 5	6 7 8 9 10 11	49 50 51	52 53 54	55 56 57	58 59 60	61 62 63	64 65 66	67 68 69	71 72	74 75 76 77	78 79	80
		X X XX+X X	X X.X		X X.X		X X.X		X X.X	.X X	X X X X	.X X	
000	00	049900	000	999	999	999	999	999	000		9999		1
000	00	050608	000	999	999	999	999	999	000		9999		1
000	00	051400	000	999	999	999	999	999	000		9999		1
				999		999		999					1
				999		999		999					1
				999		999		999					1
				999		999		999					1
				999		999		999					1
				999		999		999					1
				999		999		999					1

KPO: PUNCH ZEROS IN ALL UNFILLED SHOWN COL'S.

Control cards must be used to begin a job, to end a job, at any fill change and any required average section point. Dist. Code: 000 = Not used. . 999 = Shot punched on X-sec. card. Actual dist = dist for rod to be computed. (D1, D3, D5 & D7 may be coded 000, 999, or a preset dist.)

INCR = Required average fill increments in stations (coded 9999 computes average fill's at control card breaks only).

First & second passes

STA 501+00	ERRATIC SEC.
STA 504+00	ERRATIC SEC.
STA 504+00	FILL 1.14

SAMPLE PROBLEM

TYPEWRITER ERROR

LISTING FROM PASS 1.

GRADE LAYER
CONTROL CARDS

Route		Project				Type	Work Class	Acc.
No.	Sec.	Class	No.	Sec	Agr. No.			

For Computer Section

Job No. ____________

Date Received ____________

Estimated Completion Date ____________

Request Received By ____________

To be coded completely by requesting department

TEMPLATE CONTROL DISTANCES

JOB	EQU	S T A	D 1	D 2	D 3	C	D 5	D 6	D 7	Leveling Course Uncomp.	I N C R	Total Comp.	
1 2 3	4 5	6 7 8 9 10 11	49 50 51	52 53 54	55 56 57	58 59 60	61 62 63	64 65 66	67 68 69	71 72	74 75 76 77	78 79	80
		X X XX+X X	X X.X		X X.X		X X.X		X X.X	.X X	X X X X	X X	
000	00	049900	000	999	999	999	999	999	000	14	9999	37	1
000	00	050608	000	999	999	999	999	999	000	17	9999	25	1
000	00	051400	000	999	999	999	999	999	000		9999		1
				999		999		999					1
				999		999		999					1
				999		999		999					1
				999		999		999					1
				999		999		999					1
				999		999		999					1
				999		999		999					1

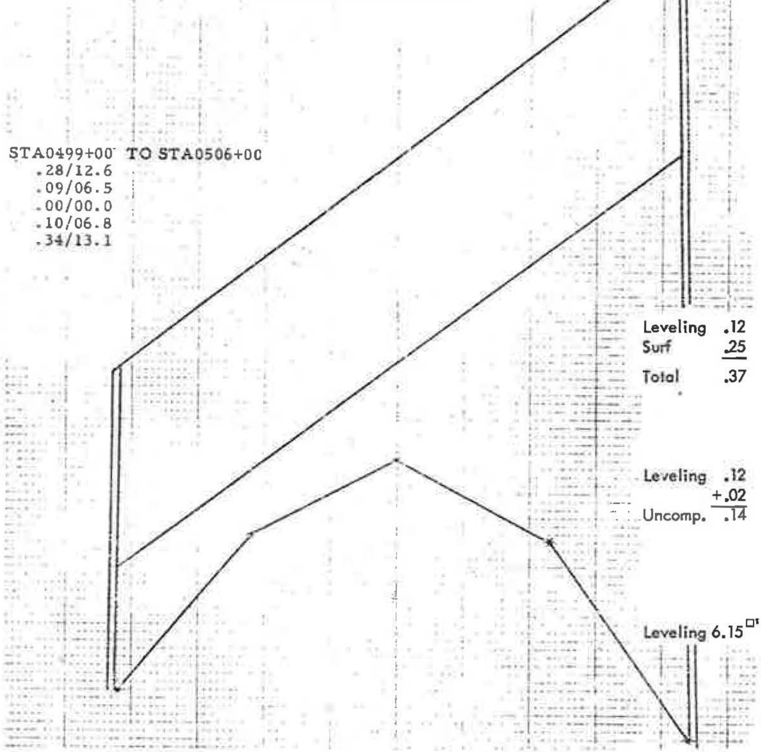
KPO: PUNCH ZEROS IN ALL UNFILLED SHOWN COL'S.

Control cards must be used to begin a job, to end a job, at any fill change and any required average section point.
Dist. Code: 000 = Not used. .999 = Shot punched on X-sec. card. Actual dist = dist for rod to be computed.
(D1, D3, D5 & D7 may be coded 000, 999, or a preset dist.)

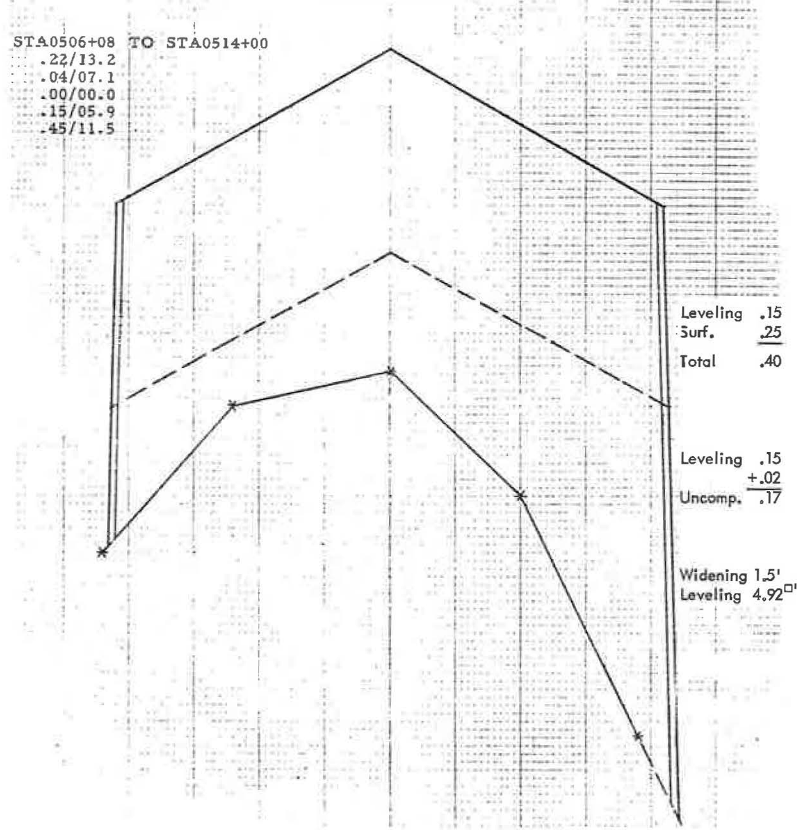
INCR = Required average fill increments in stations (coded 9999 computes average fill's at control card breaks only).

Third pass

Sample Problem - Typical Average Cross Sections

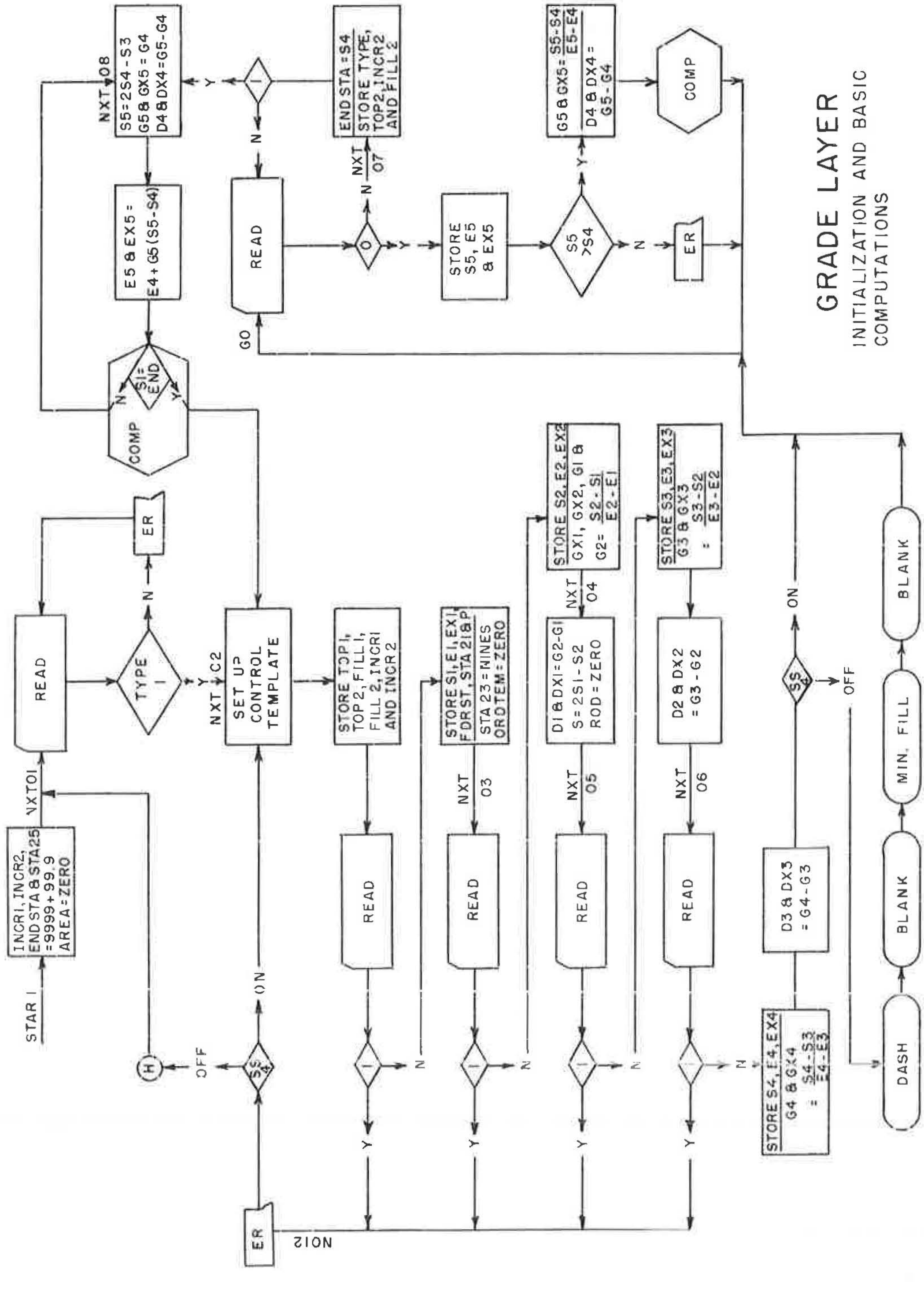


Sample Problem - Typical Average Cross Sections

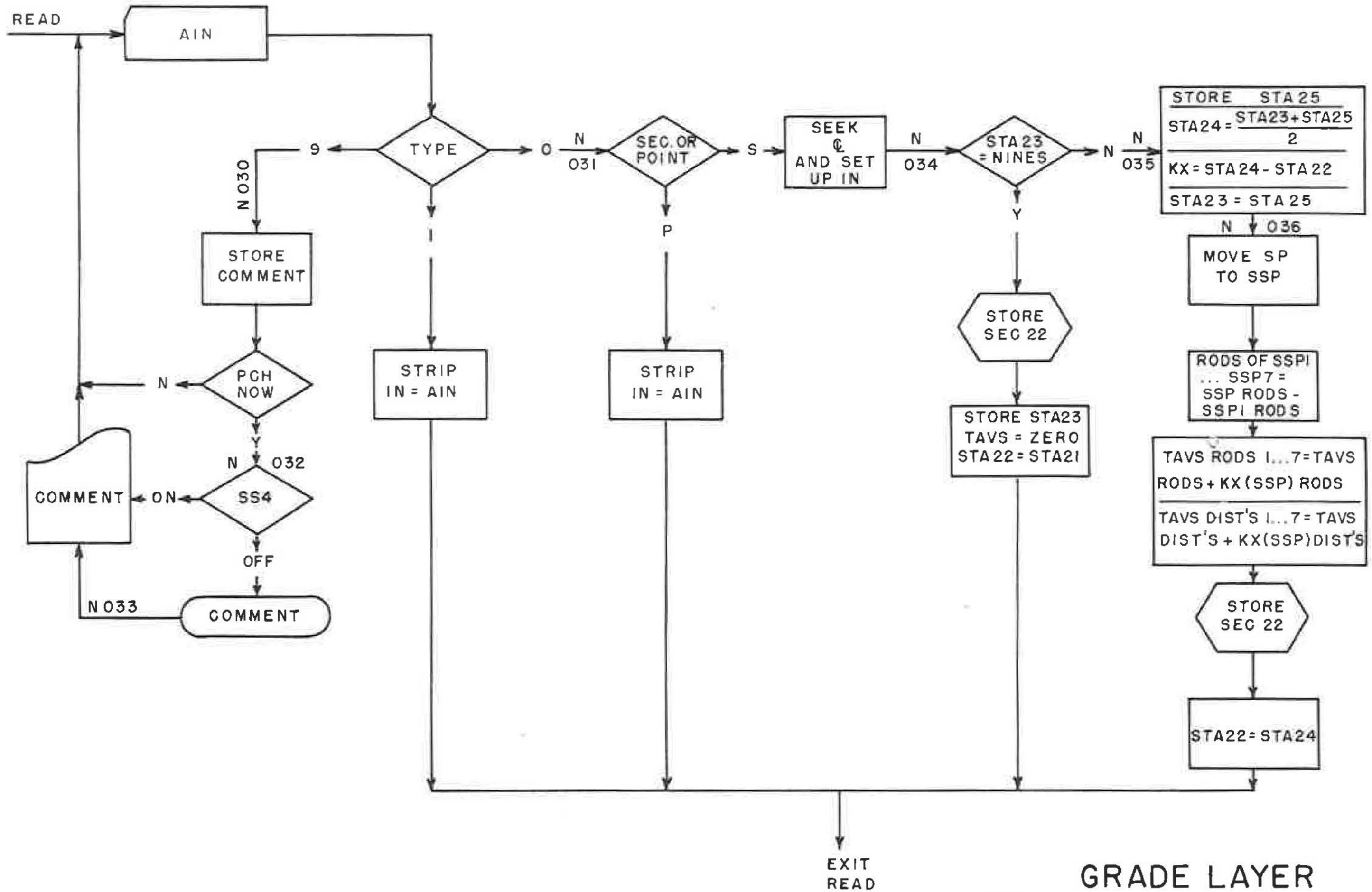


Sample Problem - Typical Listing

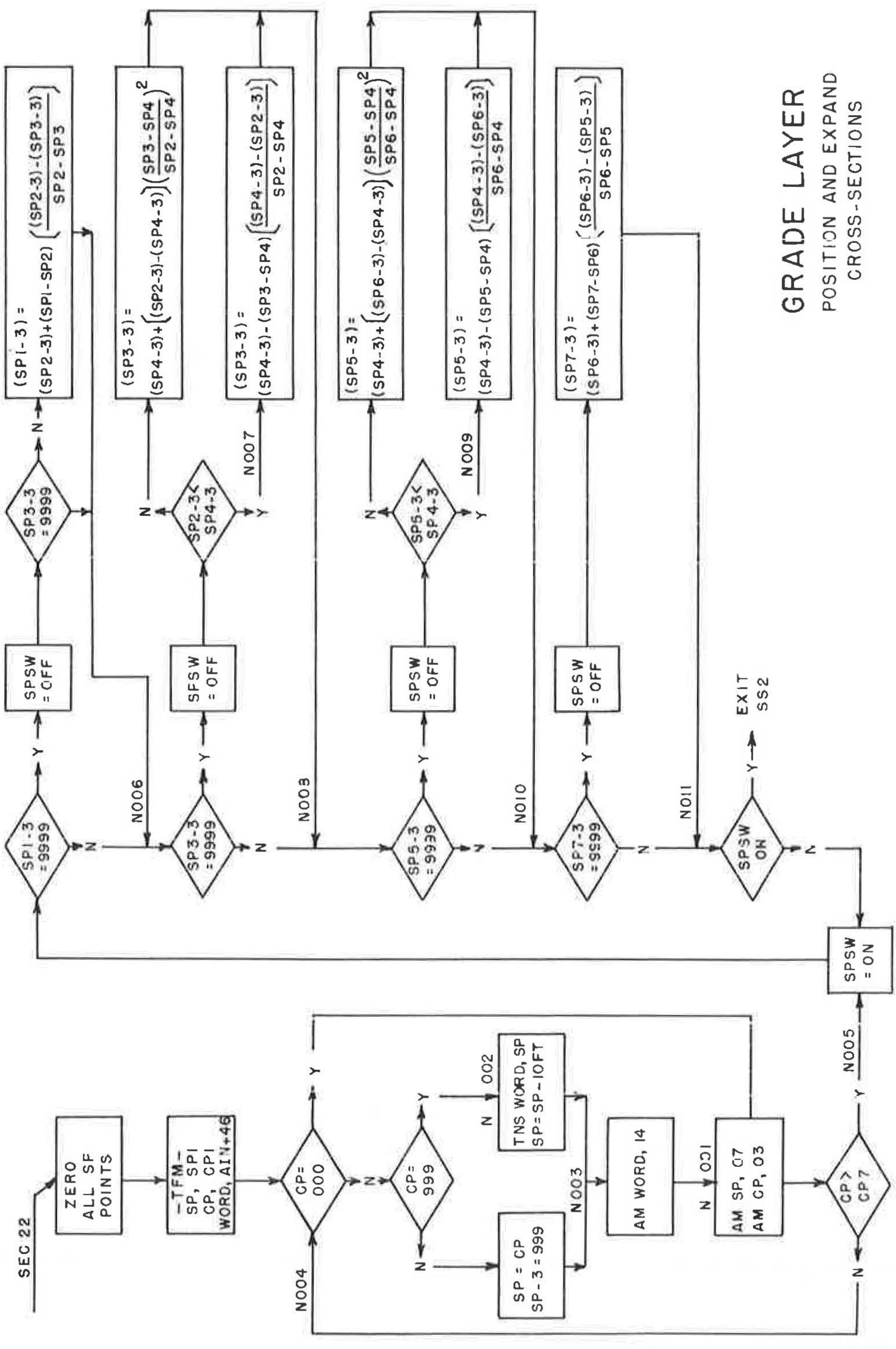
STA.	CR.GR.	ORD.	GR.DIF.
499+00	3368.46	0.14	00.00
499+25	3368.46	0.14	-00.04
499+50	3368.45	0.15	-00.01
499+67	3368.44	0.14	-00.07
499+75	3368.43	0.14	-00.08
500+00	3368.38	0.14	-00.10
500+25	3368.31	0.16	00.01
500+50	3368.23	0.14	00.04
500+75	3368.17	0.15	-00.07
501+00	3368.09	0.14	-00.00
501+25	3368.01	0.14	-00.26
501+50	3367.87	0.15	-00.00
501+75	3367.72	0.14	-00.08
502+00	3367.55	0.14	00.06
502+25	3367.40	0.15	-00.00
502+50	3367.25	0.14	-00.11
502+75	3367.07	0.14	-00.04
503+00	3366.88	0.14	-00.08
503+25	3366.67	0.15	-00.00
503+50	3366.46	0.14	00.01
503+75	3366.25	0.15	00.03
504+00	3366.05	0.14	00.04
504+25	3365.86	0.15	00.11
504+43	3365.74	0.15	-00.07
504+50	3365.69	0.14	00.16
504+75	3365.55	0.14	00.05
504+82	3365.51	0.14	-00.01
505+00	3365.42	0.14	-00.06
505+25	3365.28	0.16	-00.00
505+50	3365.13	0.14	-00.14
505+68	3365.00	0.15	00.01
506+00	3364.78	0.15	00.01
AVERAGE FILL		.145	
506+08	3364.74	0.17	00.06
506+25	3364.60	0.18	00.11
506+50	3364.42	0.18	00.15
506+75	3364.28	0.18	00.04
507+00	3364.15	0.18	-00.00
507+25	3364.02	0.17	-00.02
507+50	3363.88	0.19	00.07
507+75	3363.76	0.18	00.15
508+00	3363.68	0.19	00.06
508+25	3363.62	0.19	-00.00
508+50	3363.55	0.17	-00.12
508+60	3363.51	0.17	00.13
508+75	3363.47	0.18	-00.00
509+00	3363.41	0.17	-00.02
509+25	3363.34	0.17	-00.04
509+50	3363.26	0.17	-00.16
509+75	3363.14	0.19	00.04
510+00	3363.03	0.18	00.04
510+25	3362.93	0.18	-00.05
510+50	3362.82	0.17	-00.15
510+75	3362.67	0.18	-00.08
511+00	3362.50	0.17	-00.00
511+25	3362.33	0.17	-00.05
511+50	3362.15	0.20	00.16
511+75	3362.00	0.20	00.06
512+00	3361.88	0.19	-00.00
512+25	3361.75	0.17	-00.05
512+50	3361.61	0.17	00.02
512+75	3361.47	0.18	00.06
513+00	3361.35	0.19	-00.00
513+25	3361.23	0.17	-00.01
513+50	3361.11	0.20	00.03
513+75	3360.99	0.18	00.02
514+00	3360.88	0.18	00.00
AVERAGE FILL		.181	



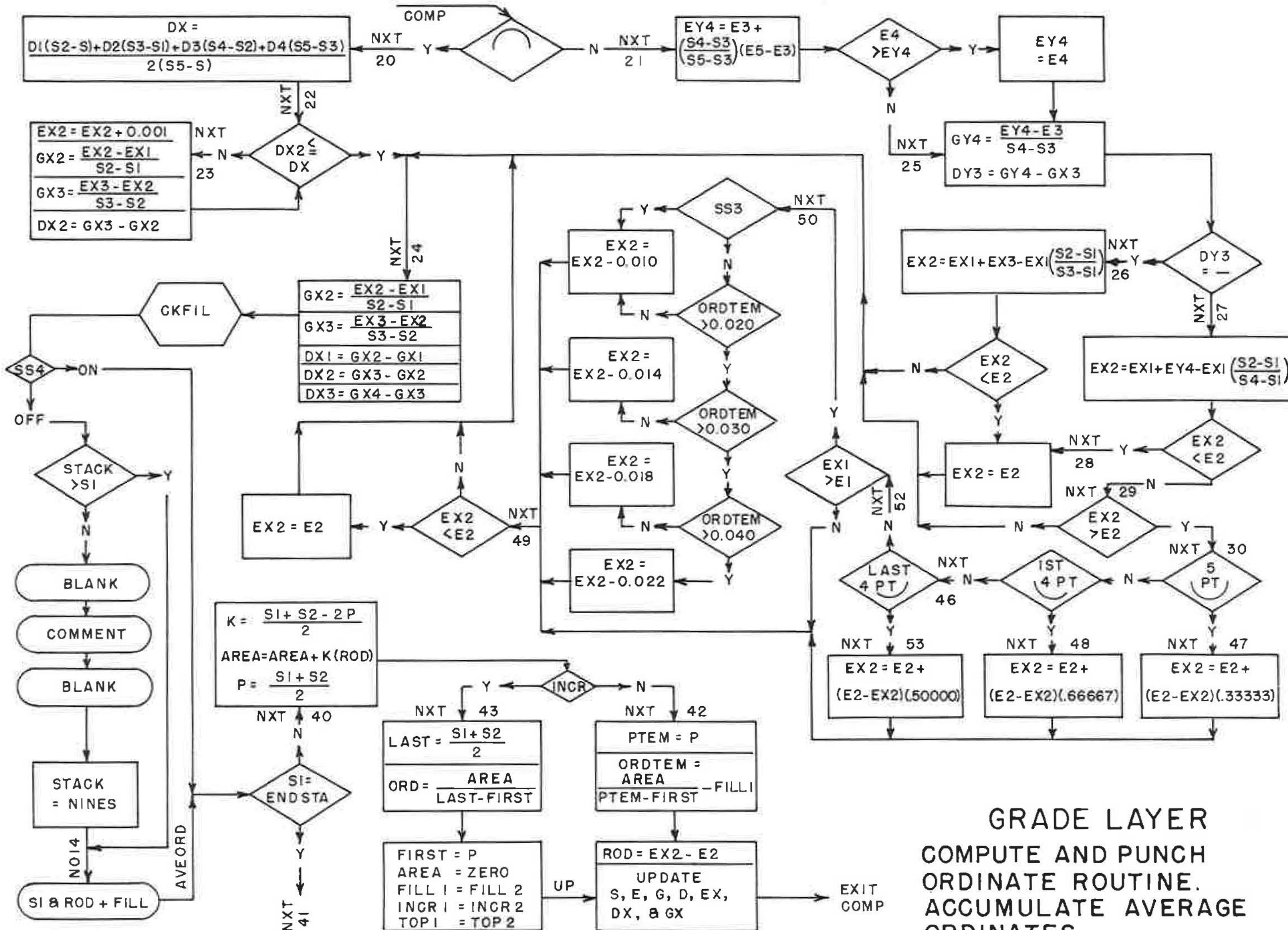
GRADE LAYER
INITIALIZATION AND BASIC COMPUTATIONS



GRADE LAYER
READ-PROCESS COMMENTS,
SET UP PROFILE POINTS,
AND ACCUMULATE AVERAGE
CROSS-SECTIONS.

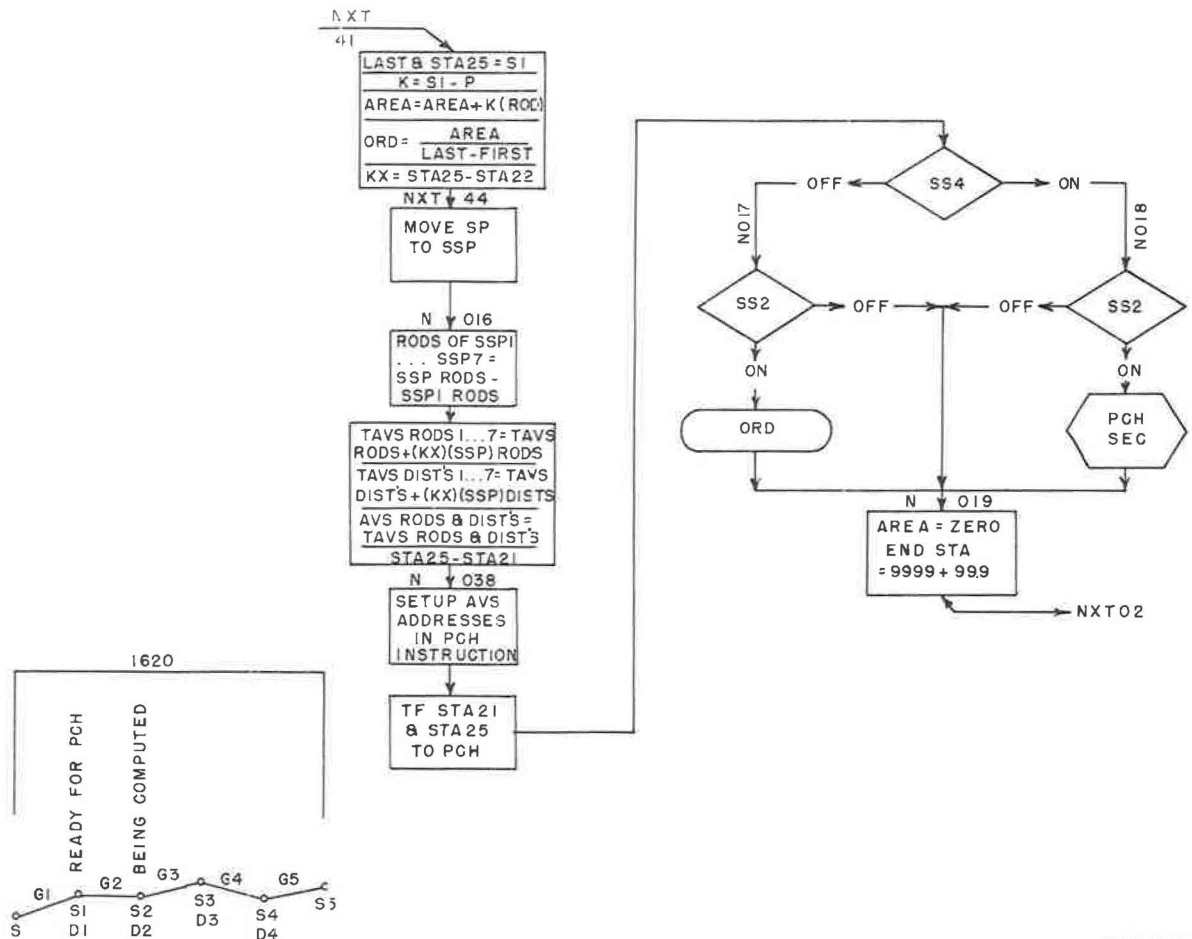


GRADE LAYER
POSITION AND EXPAND
CROSS-SECTIONS



GRADE LAYER

COMPUTE AND PUNCH ORDINATE ROUTINE. ACCUMULATE AVERAGE ORDINATES.



GRADE LAYER

ACCUMULATE FINAL AVERAGE CROSS-SECTION AND FINISH COMPUTATIONS

DATA PROCESSING INSTRUCTIONS

I Key punch cross sections and original profile from the field notes.

II Prepare control and comment cards.

III Sort all cards together by equation and station.

IV Make computer edit run.
Normal: Sense Switch 2 and Sense Switch 4 on.
If Sense Switch 2 is off, no X-sections will be punched.
Start program 05000.

ERRORS AND MESSAGES:
1. NO FILL ORDINATE CARD - Control card needed at the beginning of the data deck. Add this card to the front of the deck. Reload the read hopper and depress start.

2. FIVE CONSECUTIVE DATA CARDS REQUIRED - Depress start.

3. XXXX+XX SEQ ER - Depress start.

4. NO CL ON X-SECTION CARD - Invert card and depress start.

5. XXXX+XX FILL = X.XX - No halt involved.

6. XXXX+XX ERRATIC SECTION - No halt involved.

At the completion of the computer run, make a 407 listing using the "Plot X-Section" board. Return the listing and typed output to the programmer.

V Make all card corrections and insert necessary comment cards.

VI Make computer grade layer run.
Sense Switch 2 on.
Start program - 05000.

ERRORS AND MESSAGES:
1. NO FILL ORDINATE CARD - Control card needed at the beginning of the data deck. Add this card to the front of the deck. Reload read hopper and depress start.

2. FIVE CONSECUTIVE DATA CARDS REQUIRED - Call programmer.

3. XXXX+XX SEQ ER - Call programmer.

4. NO CL ON X-SECTION CARD - Call programmer.

5. XXXX+XX FILL = X.XX - Call programmer.

6. XXXX+XX ERRATIC SECTION - No halt.

Make four 80-80 407 listings and return them with the typed listings to the programmer.

VII Make visual inspection of the 4 0 7 listings and the typed output.

Send the field notes and the 40 7 listings to the Designer.

Asphalt Paving With Automatic Screed Control

MATHIAS BLUMER, dipl. Ing. ETH/SIA, Frutiger Söhne AG, Thun/Switzerland

In Switzerland, deviation from the correct gradient is determined by a leveling instrument. Another device called a "Planum" records the profile of the road from one leveled point to another. Because a great deal of time is necessary to obtain an accurate surface profile by this means, movable testing instruments will be used and pavement quality will be classified by means of a smoothness index.

From 1963 through 1965 the author's firm carried out an extensive program of testing bituminous pavers with a floating screed unit, equipped with automatic screed control. Some of these tests are described and illustrated. For instance, on highway N 1, "Grauholz," an equalizing course and a wearing course were laid on a bituminous base course. Although the deviations from true profile in the base amounted to 2 in., no attempt was made to patch up the hollows first manually. The average deviation from true grade was (a) base course, 0.38 in.; (b) binder course, 0.13 in.; and (c) wearing course, 0.10 in. The tests prove that owing to the electronic screed control it is possible to lay surfaces of outstanding smoothness, even when there are large depressions and elevations in the base.

When a bituminous mat is laid with a finisher using an electronic controller special conditions must be fulfilled for obtaining a surface free from waves. These conditions are specified in the paper.

High primary compaction through the screed is of great importance in smoothing out an uneven base by machine without previous manual patching. As demonstrated by a test described in the paper, the degree of primary compaction by the screed unit determines the effectiveness of reducing the depth of potholes.

•SMOOTHNESS is one of the most important features of a pavement. A road with a smooth surface is safer and more comfortable for driving than a rough one. Research and experience have shown that the initial smoothness of a pavement has a direct influence on its durability.

Any unevenness in the structure of a road causes vehicles to bump. Unevenness in longitudinal direction causes vertical acceleration

$$a_v = s^2 \cdot dLS/d\ell$$

Paper sponsored by Committee on Construction Practices—Flexible Pavement and presented at the 45th Annual Meeting.

where

a_v = vertical acceleration,
s = speed,
dLS = change of longitudinal slope, and
$d\ell$ = distance traversed.

The forces of negative acceleration reduce the adhesive weight and therefore reduce driving safety. There is a high correlation between acceleration and driving comfort. As the formula shows, the rate of change of slope with distance is of great importance, and short waves seriously impair comfort of the users, especially at high speeds.

Unevenness in the transverse direction causes additional horizontal acceleration

$$a_h = g \cdot dTS$$

where

a_h = horizontal acceleration,
g = gravity, and
dTS = change of transversal slope.

On curves, additional centrifugal force causes side-slip of vehicles. Changes of slope cause wheels to shimmy. Therefore it is important to maintain the designed slope.

RECORDING AND EVALUATING PAVEMENT ROUGHNESS

Swiss specifications for highway pavement surfaces can be summarized as follows:

1. The maximum variation from plan grade is ± 0.4 in.
2. When the pavement is tested with a 13-ft straightedge, the maximum permitted variance is $\frac{1}{8}$ in.

Deviation from the correct gradient is determined by the Planum which consists of a 13-ft long plank with a recording wheel that records the profile of the road from one leveled point to another. The distance traversed is recorded on a drum in the scale 1:20, the ordinates in the scale 1:1. However, much time and care are required to obtain an accurate surface profile by this means. In addition, the correlation between road profile and actual driving comfort is lower than the correlation between acceleration profile and driving comfort. For these reasons, movable highway testing instruments will be used in Switzerland, and pavement quality will be classified by means of a roughness index. Tests have been made with the AASHO profilometer and with the BPR roughness indicator.

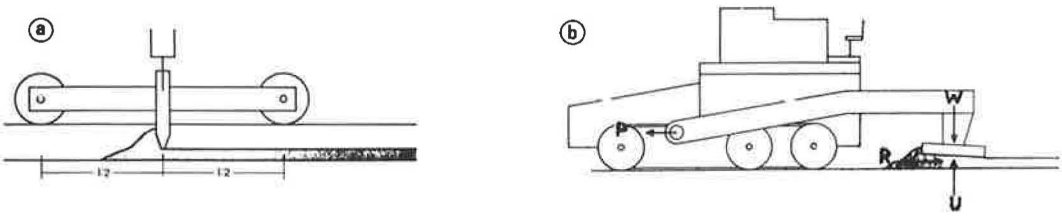
BITUMINOUS PAVERS

Fixed Screed Unit Paver

In European highway construction the same machines are often used for asphalt paving and for portland cement concrete paving. The equipment consists of two different units, the spreader and the finisher (Fig. 1a). The finisher consists mainly of a tractor, a small blade for striking off the material, and a screed for smoothing it. Blade and screed do not float on the mix; they are attached to the tractor. The finisher moves on rails or directly on a previously laid cement concrete curb (Fig. 2). The finisher is independent from the base, enabling it to lay a bituminous pavement which shows only slight deviations from true profile. However, every unevenness in the finisher's lane is directly transmitted to the surface in the scale 1:2, producing short, shallow waves which have perceptible effects on driving comfort (Fig. 3).

Paver With Floating Screed, Fitted With Electronic Screed Control

The main parts of the paver are a tractor unit and a screed unit. The screed unit is attached by long leveling arms, extending from the screed unit to a pivot point near the front of the tractor unit. Thus the screed floats on the mix as it is being spread.



$$U = \delta \cdot s^2 \cdot a \cdot \sin^2\alpha = W$$

Figure 1. (a) Paver with fixed screed; (b) paver with floating screed.

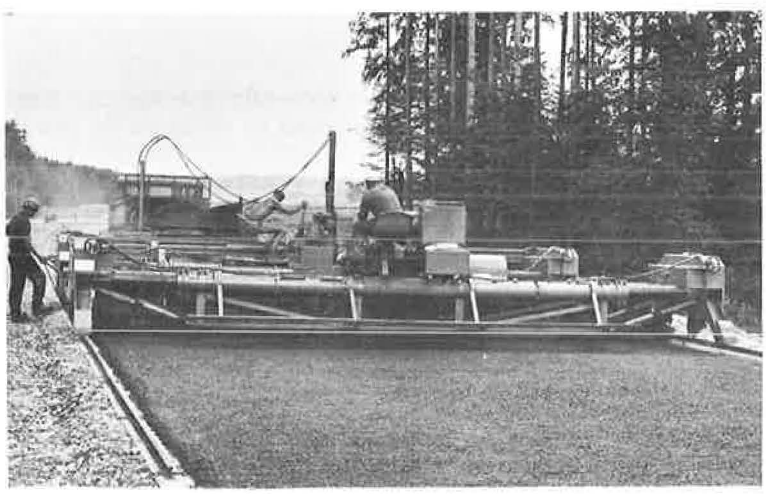


Figure 2. Paver with fixed screed, moving on rails.

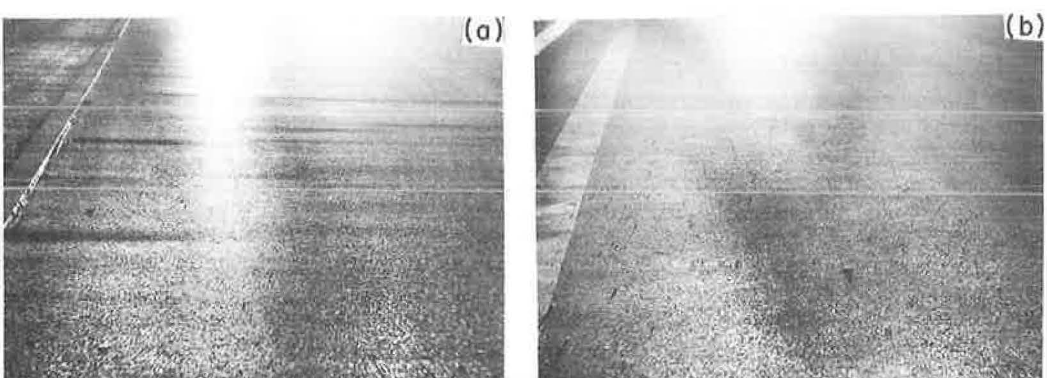


Figure 3. (a) Pavement laid with fixed screed paver; (b) pavement laid with floating screed paver.

Figure 1b shows the forces acting on the screed unit during the paving operations. As long as the paver is moving, pull, P, at the pivot point always exceeds horizontal resistance, R, in the screed plate. The screed constantly brings into balance or keeps in balance the vertical forces, vertical uplift, U, and weight, W, according to

$$U = s^2 \cdot \delta \cdot a \cdot \sin^2\alpha = W$$

$$\sin^2\alpha = W/a \cdot 1/s^2 \cdot f(t)$$

where

U = vertical uplift,
δ = density of mix,
s = paving speed,
α = angle of approach,
W = weight of screed unit,
a = area of screed, and
t = thickness.

At a given paving speed the paver has a tendency to lay a mat with a thickness that remains constant. A given tilt of the screed corresponds to each thickness. When the paver's track rollers pass over an elevation in the base, the screed plate is tilted upwards, and, as a result, the vertical uplift exceeds the weight and causes the screed plate to rise. While it rises U diminishes, until equality with W is restored and the vertical motion stops. Then the screed plate again moves in the horizontal direction only, in a path parallel to the direction of the pull. However, balance is not affected by the change of the angle of approach alone, but also by a change in the density of the mixture which occurs when the screed is pulled over an elevation or a depression in the base. When there is a hollow, the density diminishes and the screed lowers itself, but when there is an elevation the heightened density causes the screed to rise.

Irregularities in the base influence the smoothness of the surface by disturbing the balance of the forces acting on the screed. However, deviations are strongly reduced and do not appear immediately. A pavement which has been laid using a paver with a floating screed shows only a few slight waves, which have little influence on driving comfort (Fig. 2).

If the base on which the paver is moving does not correspond to the true profile, it is very difficult to lay a pavement conforming to standards. The operator has to change the tilt of the screed constantly in accordance with the thickness of the carpet. To do this at the right time and in the right proportion requires considerable skill.

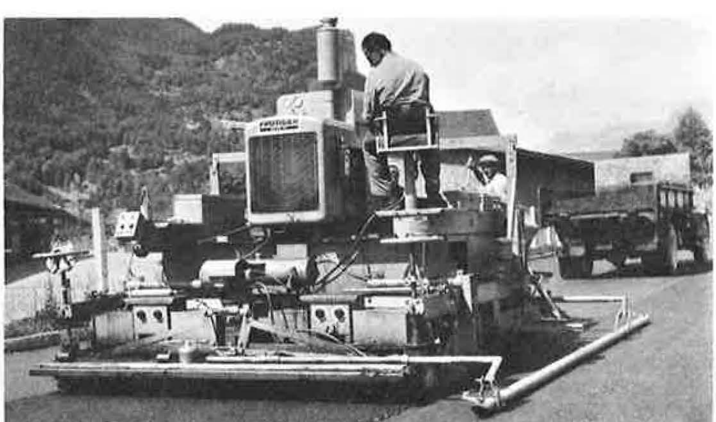


Figure 4. Floating screed paver unit with electronic screed control.

The latest achievement in bituminous asphalt paving is a paver with floating screed, equipped with an electromatic screed controller system (Fig. 4) which makes it possible to lay pavements with exceptional smoothness and practically no deviation from true grade. In this system the necessary corrections of the screed approach angle are made with servomotors which react instantly to electric impulses from the control box (faster than is possible with manual screed control). Various tests made with the paver fitted with electronic grade and slope control are described in the following section.

EXPERIMENTS WITH ELECTRONIC SCREED CONTROL SYSTEM

Smoothing Out Irregularities in Subgrade

Heterogeneity of the soil is an important problem in Swiss highway construction. Often the subsoil has not quite consolidated at the time the pavement is laid, and settlement must be expected after traffic has begun. In such cases a 4-in. asphalt-concrete (AC) base is laid and exposed to traffic for two or three years and then the final pavement ($1\frac{1}{2}$-in. binder course and $1\frac{1}{4}$-in. surface course) is laid.

In 1961-1962 highway N 1 "Grauholz" was laid with a bituminous base course by means of a conventional paver. Under traffic several hollows appeared which were to be corrected when the final pavement was laid during 1964-1965. Although the deviations from true profile in the base amounted to + 0.8 and - 2.7 in., no attempt was made to patch the hollows manually with a smoothing course mixture.

The following process was chosen:

1. A wire was stretched parallel to the determined grade on the left side for a reference line (Fig. 5).
2. The sensor glides along this reference line; any deviation from the correct height releases electronic impulses controlling the left side servomotor, which corrects the tilt of the screed.
3. The position for the correct slope can be set on the control panel (Fig. 6). A pendulum device senses every deviation and reports it to the control box. From there the right side servomotor, which raises or lowers the outer screed, is actuated.

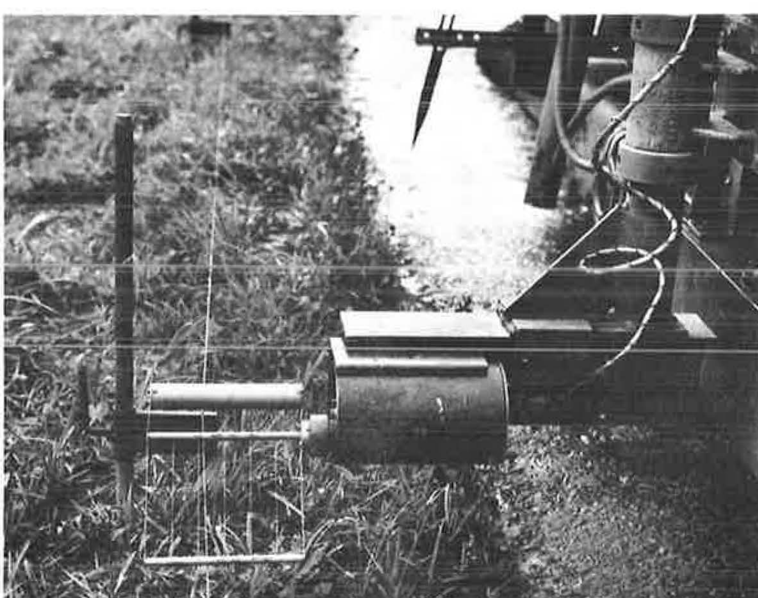


Figure 5. Grade sensor following stretched wire reference line.

Figure 6. Automatic screed control paver, showing control panel.

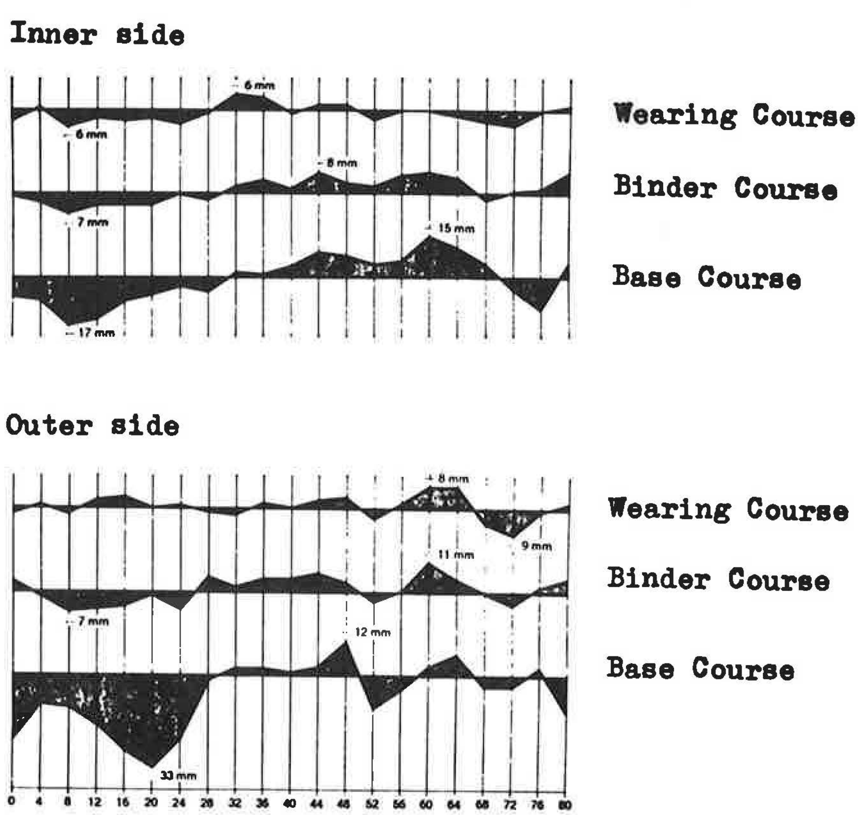


Figure 7. Deviation from true profile.

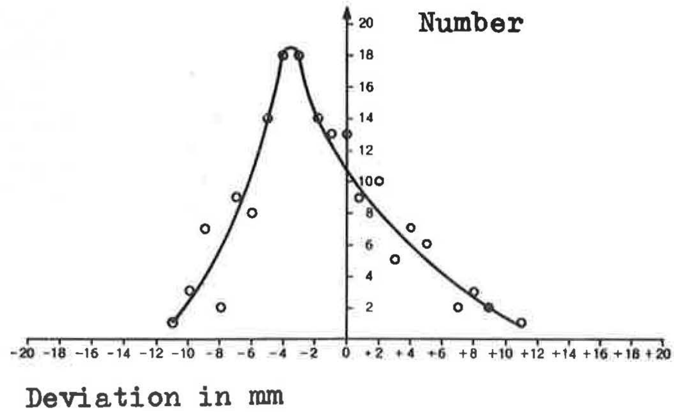


Figure 8. Frequency of deviations from true profile (168 leveled points).

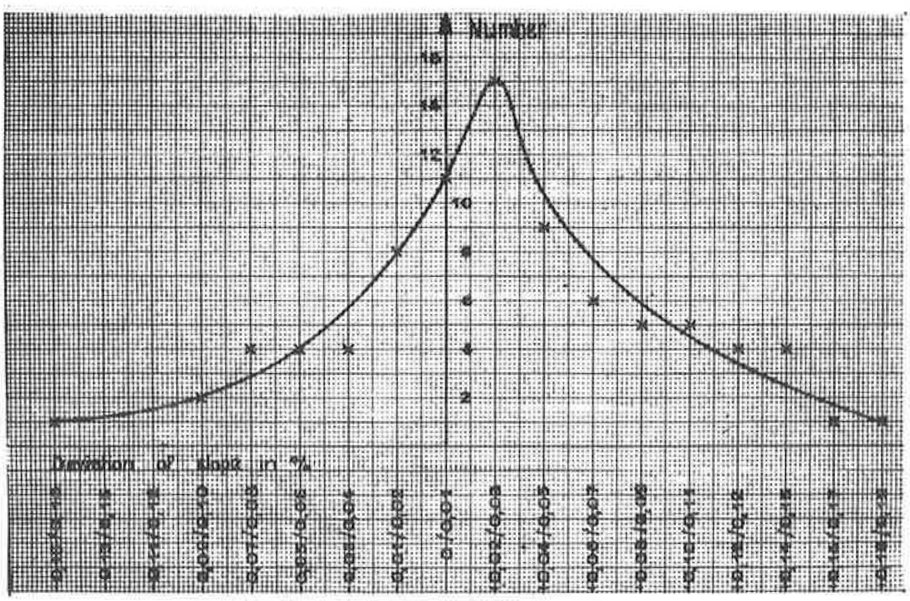


Figure 9. Frequency of slope deviations.

In two test sections of 330 ft each, the heights of the base, the binder and the wearing course were leveled and compared with heights on the plan. The results attained on section Nordrampe are shown in Figure 7.

For comparison purposes, we have calculated the profile variance, PV, which is defined as the average squared deviation from true grade.

Section	Base Course, $\sqrt{PV}$ (in.)	Binder Course, $\sqrt{PV}$ (in.)	Wearing Course, $\sqrt{PV}$ (in.)
Nordrampe			
Inner side	0.34	0.14	0.09
Outer side	0.38	0.13	0.10
Eyfeld			
Inner side	0.25	0.15	0.11
Outer side	0.41	0.14	0.10

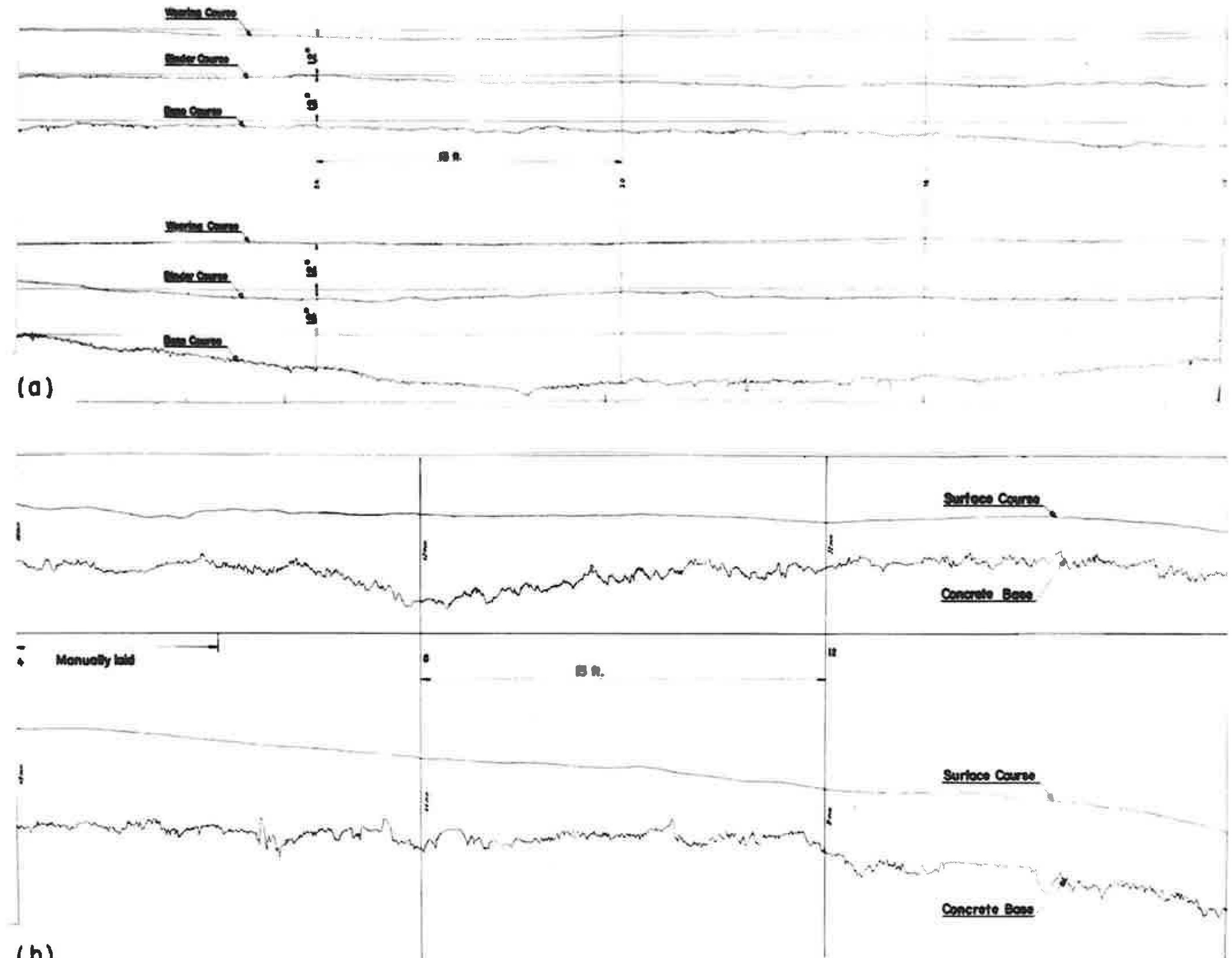


Figure 10. Evenness diagram made with Planum instrument: (a) typical pattern of highway N 1, Grauholz; (b) resurfacing on cement concrete of a bridgway.

The base course was laid with a conventional paver; for the binder and wearing courses a paver with electronic screed control was used.

Figure 8 shows the deviations of binder and surface from true profile at 168 leveled points; 166 points are within the allowed tolerance of ± 0.40 in., and only 2 exceeded this tolerance. Figure 9 shows the frequency of deviation from correct slope in 84 controlled cross-sections. Figure 10a shows the evenness diagram for a 330-ft section made with the Planum instrument.

These tests demonstrate that with electronic screed control it is possible to lay surfaces of exceptional evenness despite large depressions and elevations in the base. The deviations from true profile are well within the tolerances allowed both along the reference line and on the opposite side which is corrected with the pendulum.

The electronic screed control system permits establishment of a reference line on both sides of a lane, and makes it possible to transmit the heights by using two sensors. However, experience has shown that only one reference line on one side is necessary, because the pendulum does the transmitting to the other side.

When there is a change of slope in a transition to a curve, the slope must be indicated at the edge, and the operator has to make constant adjustments through the control panel. With this method exact results can be obtained even when the transitions are very short and extreme (Fig. 11).

Paving With the Traveling String-Line Procedure

The work is not always done with a stretched wire as reference line. Sometimes the height can be transmitted from the base or from the asphalt or concrete curbs (Fig. 12). To straighten out irregularities in the reference line, a 20-ft long ski is towed along. The electronic grade-control sensor receives the height values from the ski.

In one operation, a bordering strip of prefabricated concrete elements was at our disposal. On the evenness diagram the joints and every other unevenness were clearly visible. By means of the 20-ft ski, these irregularities could be completely wiped out. The smoothness of the pavement, after it had been laid, was even better than that of the reference line. All further layers were then built on this base.

In another operation (Fig. 10b), a bituminous resurfacing approximately 1¼ in. thick was laid on the very uneven cement concrete of a bridgeway. According to conventional methods the deep hollows in the cement concrete should first have been manually filled with bituminous mixture. But in this case the patching was omitted and the paving was done with the paver having a 20-ft long ski towed over the concrete. The evenness diagram shows that all the hollows under 20 ft long have vanished (Fig. 10b). The surface conforms to the specifications.

Smoothness of Wearing Course

It has been suggested that the adjustments in height achieved by use of the electronic controller might impair the smoothness of the mat. Some authorities think that only the lower courses, i.e., the bituminous base course and the equalizing course should be laid with electronic control, and the natural leveling quality of the finisher should be used for laying the wearing course. This procedure may be correct for roads on which several courses are laid consecutively. But in cases where mats must be laid in two separate phases, or for single-layer corrections of the profile, it is important to be able to make height corrections. In such cases the controller is necessary, even in laying the wearing course.

When a bituminous mat is laid with a finisher whose electronic controller is switched on, certain conditions must be fulfilled to obtain a surface that is absolutely free of waves. These conditions are partly related to the construction of the paver itself and partly related to the method of its operation.

1. The balance of forces in the paver unit is such that changes in the angle of approach or in the density of the mix can disturb the balance when the laying is performed at constant speed. Accordingly, one unevenness in the base can cause two in the covering mat: (a) when the tractor unit passes across the unevenness and (b) when the paver unit is directly over the unevenness (Fig. 13).

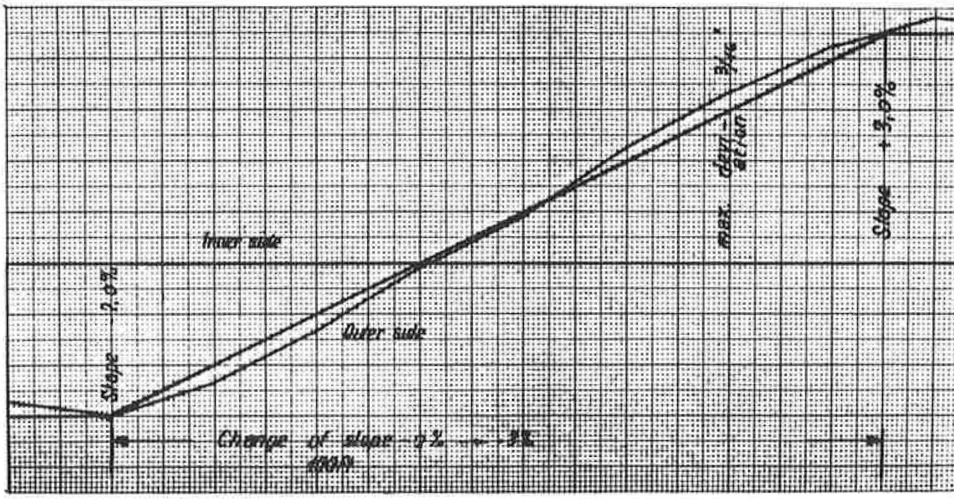


Figure 11. Deviation from true profile in a curve (change of slope - 2% to +3%).

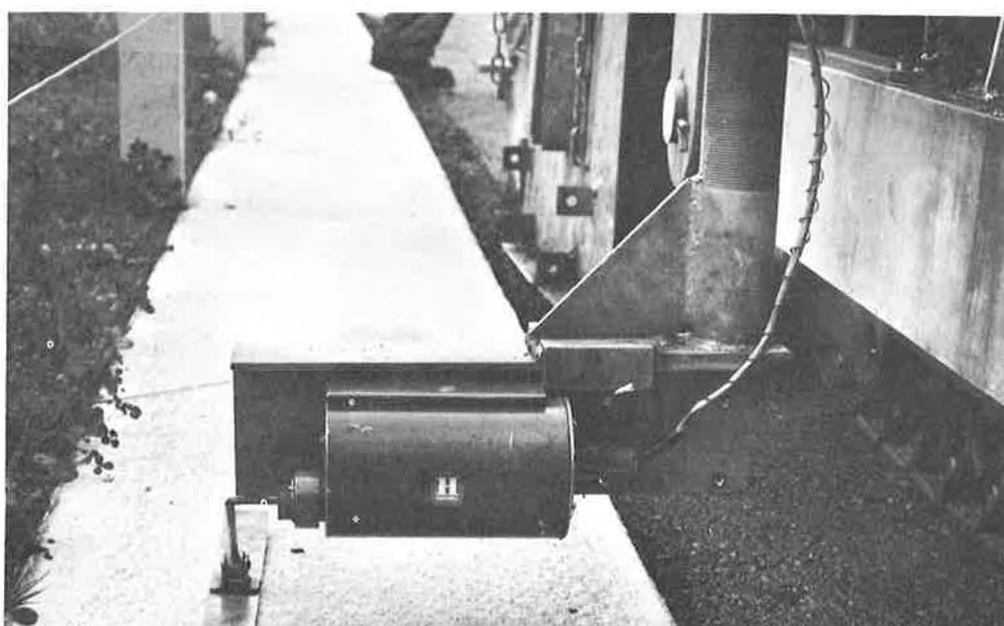


Figure 12. Follower shoe, concrete curb as reference line.

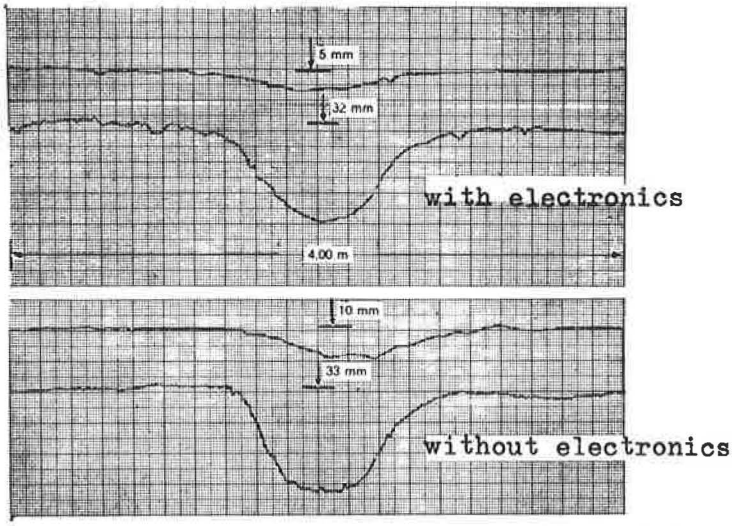


Figure 13. Evenness diagram of wearing course laid over pothole.

These deviations are corrected by the operation of the controller; they are registered by a sensor and transmitted to servomotors through electronic impulses. The motors change the angle of approach accordingly. In this operation the point at which the sensor is fastened to the leveling arm is crucial. When fastened to the forward pivot point, the sensor can transmit only the deviations of the tractor unit, and it is not influenced by a subsequent sinking of the paver unit caused by decreased density of the mix. However, if the sensor is fastened to the screed, it cannot register a movement of the forward pivot point. In this case the angle of approach is only altered when the screed moves upwards or downwards, which is comparatively late.

Our experience with different types of finishers has shown that some do not achieve as good results as others, and that machines with a sensor fastened to the paver unit react very quickly to the slightest vertical movement of the screed and therefore have a tendency to oversteer. Although this oscillation in the range of the zero position produces good conformity to the reference line, it is undesirable because it also produces shallow but short and regular waves which seriously impair the driving comfort. A sensor positioned near the pivot point is more advantageous, because its detection of the vertical movements of the screed is considerably diminished, and it only reacts to more important deviations. The mat is admittedly less true to the reference line, but at worst it will only have a long wave type of unevenness, over which it is possible to drive smoothly.

Obviously what is true for the position of the sensor applies to the position of the pendulum as well, because it controls the heights on the opposite side of the sensor. The pendulum should also be as far removed from the finisher screed as possible and be able to detect deviations as early as possible, so that corrections can be carried out gradually.

2. The finisher with a floating screed reacts sharply to differences in the mix laid before it. Therefore, the mix should be homogenous and of uniform temperature. Under no circumstances should it be unloaded directly from the truck to the base. It must be brought to the screed in such a way as to keep the height of the roll in front of the screed as constant as possible. The best results are achieved when the mistakes of the operator can be eliminated by a material depth control which automatically switches feed conveyors and spreading screws on and off.

3. The acting forces in the paver unit are dependent on the working speed, which should be as nearly constant as possible and adjusted to the amount of mix delivered per hour. Even when feeding, the finisher should not stop.

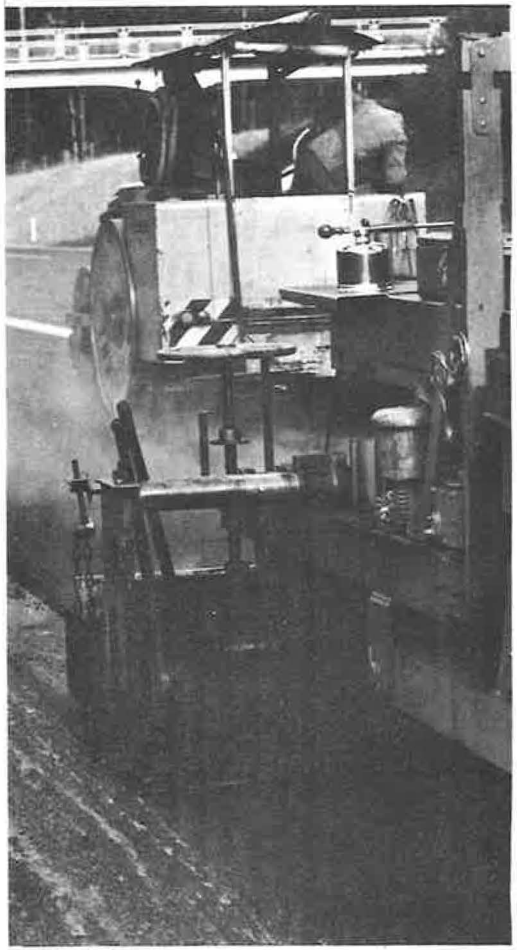


Figure 14. Compacting by vibrating screed and by steel-wheel roller.

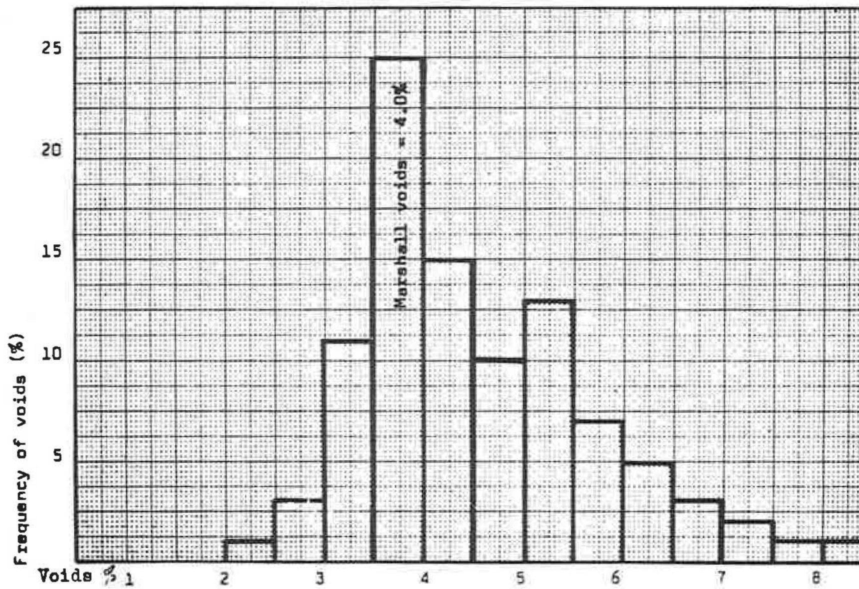


Figure 15. Void frequency of 100 samples cut from base course.

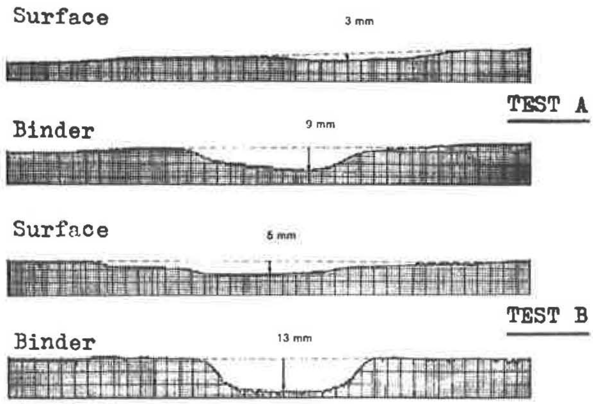


Figure 16. Evenness diagram of binder and surface course over pothole: test A, high vibrating intensity of paver screed; test B, low vibrating intensity of paver screed.

Figure 17. Two pavers equipped with electronic screed control, working in echelon, on Swiss highway N 1 Berne-Zurich.

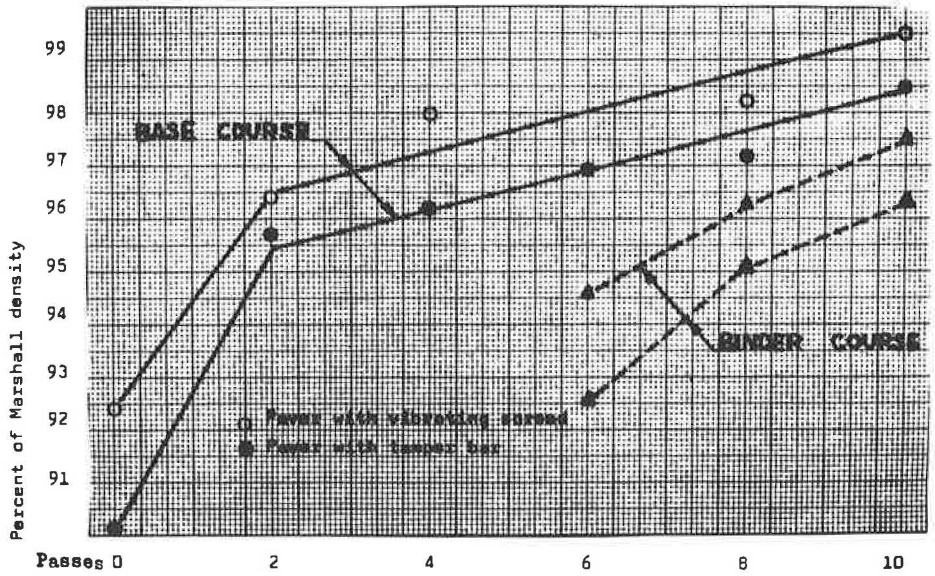


Figure 18. Density of mix, behind paver and after rolling.

4. Even high quality electronic controllers can suffer from disturbances, for instance, through changes of temperature. It is therefore essential that the height and transverse slope of the mat which has been laid be continuously checked behind the machine. The controller should be operated by the most reliable man of the crew. When a deviation is detected, work must be stopped and readjustments made.

5. Every paving operation must be prepared with great care. The leveling apparatus is useless, when the reference line is not true to profile.

CORRELATION OF SMOOTHNESS AND COMPACTING

The compacting process of bituminous mixtures goes through three stages: the paver's screed, the rollers, and traffic. Only a very slight degree of further consolidation by traffic is permissible, because otherwise settlement and rutting will be caused by wheel pressures. The finisher should achieve the highest possible compaction for the following reasons:

1. The precompacted mixture is more resistant to the horizontal forces when it is rolled. No rolling waves can form.

2. Breakdown rolling can take place directly behind the finisher, at a time when the mixture is still hot and can be well compacted (Fig. 14).

3. The screed permits absolutely homogenous compaction on the entire width of the spread. But with rolling, it cannot be determined whether all the areas on the surface have received the same amount of compaction by the roller.

Swiss specifications demand 6 rollers for compacting 1,000 tons of base mix per day. Results (Fig. 15) show that with high primary compaction by the screed compaction can be obtained using only three rollers.

High primary compacting through the screed is of even greater importance when smoothing out the uneven base by machine without prior manual patching. Each hollow in the subgrade causes a smaller hollow in the compacted mat. The higher the degree of primary compaction by the screed unit, the better is the reduction of the depth of hollows. This is conclusively proved by the following tests (Fig. 16):

Test A

Two layers of bituminous mat cover a pothole in a base of 2.4 in. Compacting was done with a vibrating paver screed (high-vibrating intensity), followed by ten passes with a steel-wheel roller.

Hollow in first (equalizing) course: 0.36 in. = 15 percent of original hollow.
Hollow in second (wearing) course: 0.12 in. = 5 percent.

Test B

Spreading and compacting are performed as previously described, but with low vibrating intensity of screed.

Hollow in first course: 0.52 in. = 22 percent of original hollow.
Hollow in second course: 0.20 in. = 9 percent of original hollow.

There are two different ways of obtaining a high compaction by screed. Some pavers use a tamper bar at a rate of 1,200 to 1,500 impacts a minute; others use a vibrating screed unit with a frequency of 3,600 vibrations per minute. For comparison purposes, two pavers were operated in echelon to lay a bituminous base course of $2\frac{1}{2}$ in. (Fig. 17). One was fitted with a tamper bar, the other with a vibrating screed. To compare the effectiveness of each compaction, the density of the mat was first measured immediately behind the paver and then after the 21-ton pneumatic tire roller had passed a few times (Fig. 18).

The results prove that compaction with a vibrating screed produces higher densities. The density is higher before rolling, i.e., immediately behind the finisher, and remains higher until after the last rolling. Results of a second test with a binder course were similar. Both tests demonstrate that high primary compaction is equivalent to high final compaction.

New Mexico's Semiannual Condition Surveys Effect Changes in Geometric and Structural Designs

CHARLES W. JOHNSON, Engineering Director, New Mexico State Highway Commission

•POST-CONDITION inspection trips are made in New Mexico semiannually to gather data on changes occurring with usage, and to determine results of design variations on recently constructed projects. Conditions are evaluated at identical locations from year to year.

The idea of making continuing surveys on new construction work developed from routine inspections of experimental projects F-051-1(8) and I-010-1(8)6, on which comparative studies of various combinations of treated and untreated bases and subbases were being conducted. It was believed that information gathered from completed projects would be equally as valuable as the experimental projects for future design determinations. The post-condition surveys started in 1960, and generally have continued with the same key personnel from the New Mexico State Highway Department and the U. S. Bureau of Public Roads.

Comparisons are made yearly at identical locations of rutting, U. S. Bureau of Public Roads roughometer readings, and cracking patterns. At selected locations Benkelman beam deflections are measured and test samples lifted to supplement routine information. W. L. Eager, regional materials engineer for the U. S. Bureau of Public Roads, documented all inspections with written reports and photographs pertinent to developing conditions. He used measurements made to calculate the PSI value by the Illinois formula (1). Comparison of readings on the same project from year to year gives a more significant measure of deterioration than the condition of the project at any one time. It may indicate the need for corrective measures such as overlays, plant mix, seals, or slope flattening before there is pronounced distress. Also, these measurements are used as the basis for special recognition to contractors and engineers doing especially good work.

A problem has evolved concerning the number of new construction projects added to the list each year. Inspection trips are made semiannually and cover approximately one-half of the state each trip. We do not feel justified in allotting more than a week for each trip. The number of projects under surveillance has already passed the two hundred mark. About one hundred projects a week are all that can be adequately covered. It will be necessary in the future to be more selective and to drop some projects that past surveys indicate are duplications or informationally nonproductive.

PAVEMENT CRACKING

Because cracking is so prevalent in flexible pavements, highway engineers are prone to accept it as a normal development. Transverse, shrinkage, or aging cracks usually appear from 2 to 4 yr after construction in New Mexico. Transverse cracking has occasionally occurred during the first winter, for instance on the following two projects: FI-143(5), located in the southwestern part of the state, and F-053-1(6), located in the northern part of the state. On both projects a basalt gravel was used in the hot mix. Laboratory tests indicated an almost unbelievably high volumetric change in the mix due to temperature changes. It was calculated by the laboratory that within the normal expected temperature ranges, lineal volumetric change for 1 mi would vary up to 7 ft.

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Figure 1. Pavement constructed with crushed limestone, showing little cracking 3 yr after placement.

In contrast, a 12-mi project constructed in 1958 was found on a post-condition survey to be in excellent condition, without any cracking, 7 yr after construction. The coarse aggregate used in the hot mix was crushed limestone; it indicated a very low volumetric change due to temperature. Although cracking is widespread in New Mexico, this project indicates that through careful control of material and placement, cracking can be greatly alleviated. Comparison of pictures taken at the same location (Alamogordo Orogrande) 4 yr apart shows little change (Figs. 1 and 2).

Joint cracking at a point adjacent to a preceding hot-mix lay is probably due to internal stresses, and develops frequently in the Southwest. Past practice in New Mexico was to lay the shoulder more or less vertical. Present practice is to lay the longitudinal joints on a slope and leave the slope uncompacted until the adjacent lane is placed. With this procedure a more homogeneous mass is produced, instead of a potential cleavage plane (Fig. 3).

It appears that so-called shrinkage cracks are caused by aggregate volumetric changes due to temperature rather than by aging of the asphalt. Whatever the cause, cracking has developed through internal stresses within the asphaltic pavement for this type of cracking.

Reference has been made to cracks caused by internal stresses within the asphaltic pavement. There are other types of cracking; one is cracking occurring from repeated wheel loads. Traffic is generally channelized, and rutting is the first indication of distress. It may be slight during the first year, but if it continues to progress, cracking will eventually appear. During our post-condition surveys, hundreds of rutting measurements were taken. The critical point appears to be from $^3/_8$ to $^1/_2$ in. in depth.

Once rupture of the surface has developed due to rutting, the condition becomes very serious as channelizing of surface water feeds moisture into supporting foundation materials which, under traffic, become oversaturated. Complete failure is imminent.

Figure 2. Pavement constructed with crushed limestone, showing little cracking 7 yr after placement.

Figure 3. Longitudinal shoulder joints being laid on slope which is compacted after adjacent lane has been placed.

Excessive rutting indicates instability either from an underdesign of the total pavement or from construction deficiencies. Common contributing factors may be insufficient compaction in the pavement, base, subbase or subgrade; materials used in the pavement structure may not have the inherent stability needed for durability or to support the applied loads. Some rutting is to be expected, and most pavements stabilize before they become critical.

Safety experts contend that one eye is dominant in driving patterns. If the left eye prevails, the driver tends to hold close to the center stripe. If the right eye dominates, the driver tends to hold close to the shoulder stripe. My personal observations lead me to believe that this is true.

Rutting on New Mexico highways develops in the travel lane, and seldom in the passing lane, when the shoulders are striped. We believe that a wider traffic lane would disperse wheel loading over a greater area and reduce the tendency toward critical rutting. Certainly, wider traffic lanes would enhance safety and driving comfort. It is also logical to believe that the pavement's life would be prolonged.

We striped several projects in New Mexico, using 12-ft passing lanes, and 14-ft driving lanes where the pavement extends shoulder-to-shoulder. The cost is no greater, as the stripe uses a part of the shoulder. These projects will be observed closely, and measurements taken during post-condition surveys.

The post-condition surveys indicate that most of the cracks, and the most damaging, occur through external forces to the mat. Often overlooked by highway engineers as inconsequential is early shoulder cracking outside the traffic lanes. The condition is general in embankment areas with relatively steep slopes. A good example of progressive deterioration occurred on I-010-1(8)6, Road Forks East, a project built with experimental features and closely watched. In Report No. 1 (state) dated November 5, 1956, station 400+00 reads: "A few longitudinal cracks were also found about one foot from the left edge of the driving surface."

Report No. 3 (Eager) dated August 16, 1960, B-condition of project, reads: "The project is in excellent condition. Transverse cracks are less in evidence than at previous inspection, February 25, 1960. Rutting (in outer wheelpath of traffic lane) is more pronounced. Longitudinal cracks, particularly in paved shoulders and in passing lanes, are pronounced in fill sections across light fill sections across flats (station 326+15. 4 to 800+00)."

The first cracks to appear in the paved shoulders are probably the result of soils movement in the subgrade. In the Southwest there are alternating cycles of moisture and extreme drought which in turn cause a swelling and shrinkage of the soils beneath the pavement. The susceptibility of the soils to moisture varies over a wide range; consequently, cracking patterns vary accordingly. When cracks begin to open up they permit entry of surface water and the condition becomes progressively worse (Fig. 4).

It is quite noticeable that the worst conditions are more pronounced in fills with relatively steeper side slopes. The westbound lane adjacent to the project under discussion was constructed in 1943 with much flatter slopes. There is no indication of similar distress, although the embankment was constructed of the same type of borrow and traverses the same terrain.

GEOMETRICS

Early in 1965 the New Mexico State Highway Department changed the standard design of slopes from a mininum of 4-to-1 to a minimum of 6-to-1 for a distance of 9 ft beyond the paved shoulder (Figs. 5 and 6).

Although our new geometric standards were originally discussed on the basis of additional support for the shoulders, there are a number of other advantages that may be even more valid.

From my viewpoint, the new geometrics are only the first step in constructing safer highways. They provide for an additional 9 ft on each side of the shoulder as a possible recovery area. This is not enough. The recovery area needs to be from right-of-way fence to right-of-way fence. General Motors has proven that anything less than a 6-to-1 slope is unsafe when a car is out of control. It is time highway engineers faced the facts.

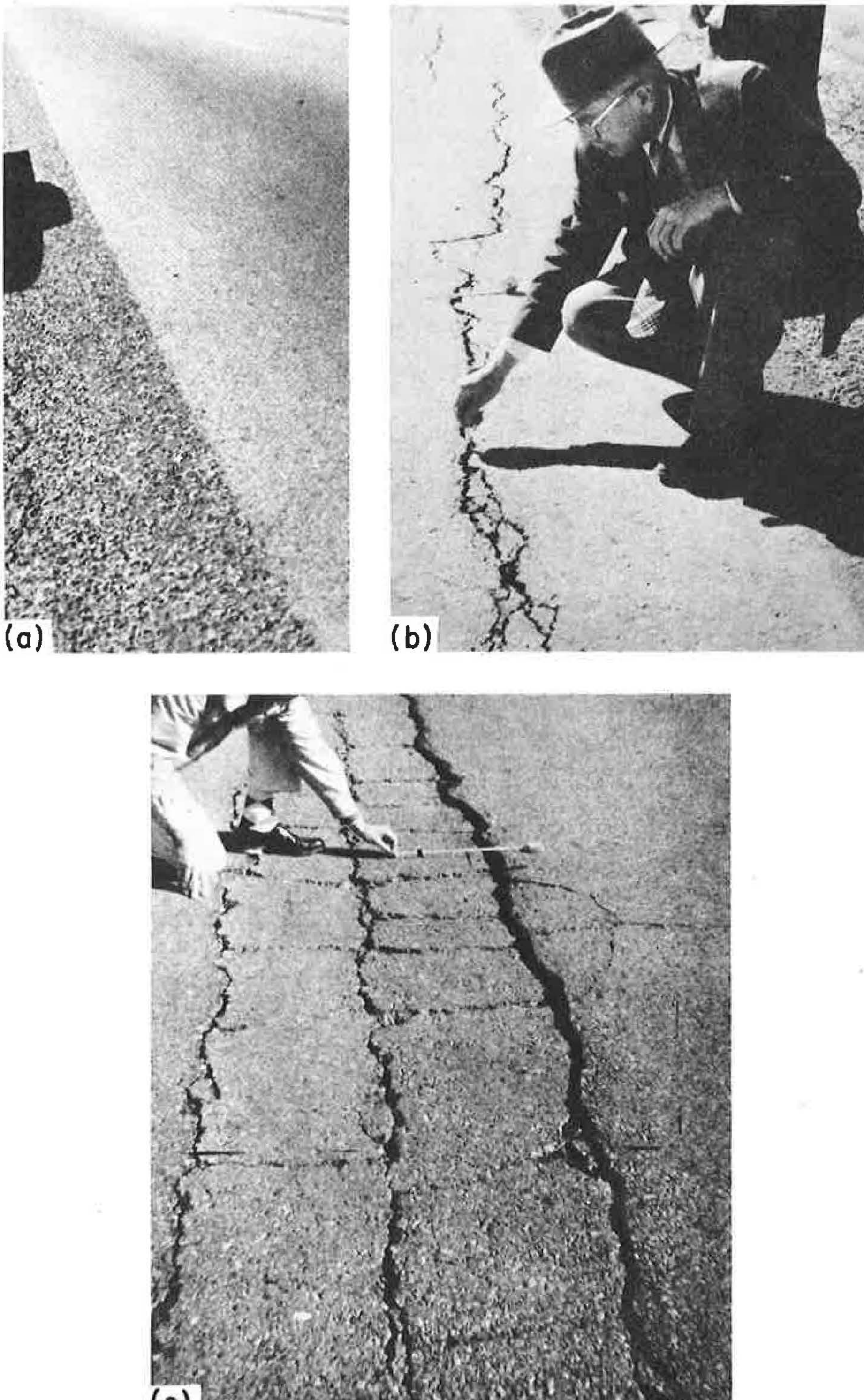

Figure 4. Typical progressive deterioration from shoulder cracking: (a) early shoulder cracking; (b) progressive cracking into driving lane; and (c) ultimate failure.

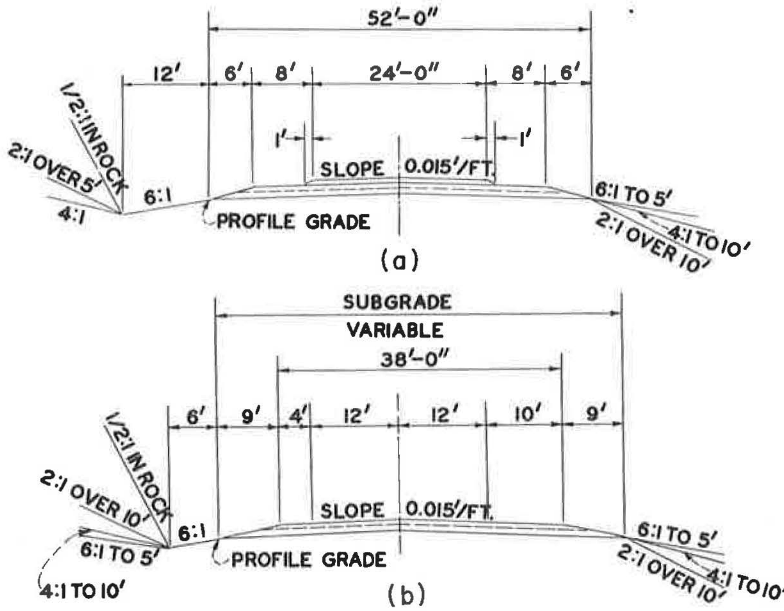

Figure 5. (a) Old and (b) new typical sections.

Not much can be done to prevent head-on collisions except to provide greater maneuverability. In rural areas, by far the greater portion of vehicles in accidents terminate off the pavement structure. In the process they strike some obstacle such as a curb, post, guardrail, or sign. These safety devices cause the accidents. It does not make sense.

When New Mexico obtained approval of the new geometric design, we planned to move the obstacles out to the 9-ft hinge point, but were unable to do so except on an experimental basis because the AASHO sign manual requires that these obstacles be placed within 2 ft of the shoulders (2). The WASHO Construction Committee, during our recent meeting at Salt Lake City, voted unanimously to request the WASHO Executive Committee to take the necessary action to permit moving the obstacles out.

We have read and heard much recently about safety, beautification, billboards, and junkyards. The most esthetic highway possible is one that gives the motorist a feeling of comfort and safety, with flat slopes and a minimum number of obstructions within the recovery area. Why are signs erected by highway departments within the rights-of-way more esthetic than billboards off the rights-of-way?

The flatter slopes are more conducive to better ground cover of natural vegetation and facilitate mowing and snow removal.

In the process of extending the 6-to-1 slope out to the 9-ft hinge point, normally crushed rock or gravel is used. The value of this material in controlling erosion became known to me as I was inspecting paved shoulders showing some distress, on New Mexico Project I-040-2(24), West of Gallup. Erosion stopped on the slope at the edge of the gravel placement (Fig. 7). The section greatly reduces the need for shoulder curbs and rundowns, which are potential accident obstacles. Also, I believe that gravel in side ditches would control erosion more effectively than check dams on grades up to 3 percent.

STRUCTURAL DESIGN

The conditions on some projects emphasized the need for, and brought about, some revisions in our structural design practices.

Although some cracking occurred on projects where untreated base course was placed directly under the asphaltic surface course, cracking patterns developed earlier

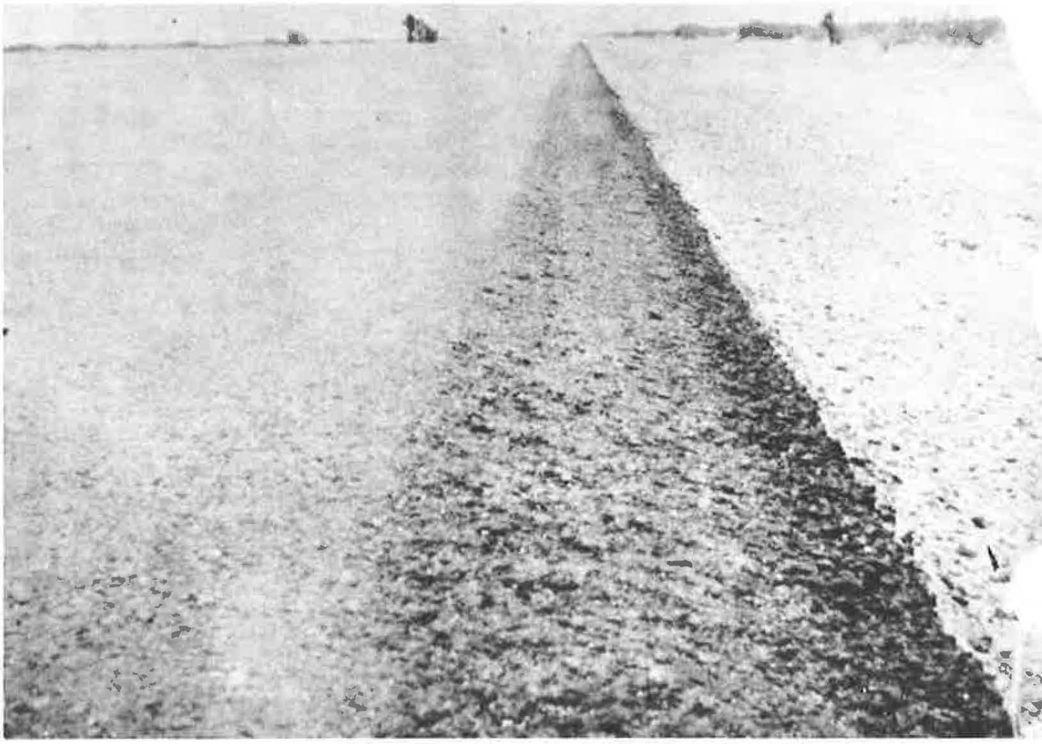

Figure 6. New design shoulder slope, June 21, 1965.

Figure 7. Four-to-one slope; slope erosion stops at edge of gravel placement.

and reached more serious proportions on those projects where the asphaltic concrete was placed directly over a cement-treated base course. To alleviate the problem caused by the transmittal of reflective patterns from the cement-treated base course through the bituminous pavement, the practice of placing cement-treated base course directly under the mat has been discontinued almost entirely and two alternate designs have been used. The first alternate design, which has been referred to as the upside-down design, consists of placing a cushioning layer of high-quality base course between the cement-treated base material and the asphalt concrete mat. Structurally, the upside-down design appears to be equal to and perhaps superior to the normal practice. As indicated by recent semiannual conditions surveys, the reflective cracking problem has been greatly reduced on those projects where the intermediate layer of cushion material has been used. The other alternate design consists of placing asphalt-stabilized base course material directly under the mat in lieu of the cement-treated material. Our limited experience has shown that the asphalt-stabilized base performs equally well; consequently, cracking problems of any appreciable degree have not been encountered on recent projects constructed using either design.

The aggregates available for highway construction in certain areas are instrumental in the design determination inasmuch as not all aggregates are equally suitable for stabilizing with cement or with asphalt. The choice of the stabilizing material, and consequently the design to be used is, therefore, influenced by the nature of the aggregate to be stabilized. Also, any type of design which results in shrinkage cracks over a plastic subgrade soil is highly undesirable, since the cracking patterns allow surface moisture to leak into the subgrade, resulting in reduced support, pumping, and swell.

Another problem noted during the course of the semiannual inspections was the development of alligator cracking patterns and deformation of the roadway structure in certain areas due to the inadequate stabilization and structural failure of the underlying courses. Benkelman beam tests performed in conjunction with post-condition inspections showed high readings in extensively cracked areas. As a result, the previous practice of using untreated base materials on high-volume roads has been discontinued, and it is the present policy to use stabilized bases on these types of highways. Benkelman beam tests taken on projects which were constructed with stabilized bases have shown readings well within the acceptable tolerances. Travis Cole, materials engineer for the New Mexico State Highway Department, recently made a comprehensive revision of our structural design criteria and procedures for flexible pavements. The new structural design criteria and procedures are based on the AASHO Road Test findings, and are given in the New Mexico Highway Department Bulletin No. 101, Structural Design Guide for Flexible Pavements.

Adoption of the new design criteria has permitted more versatility in structural design selection because more realistic structural strength coefficients are assigned the materials used in the construction of the flexible pavement structure. The inclusion of a regional factor, which permits an adjustment in thickness determination based on climatic and environmental conditions, is a realistic approach.

Rutting was generally attributed to the instability of the asphalt concrete mat. To provide more internal stability in the asphalt concrete surfacing, the requirement for the percent of fractured faces in the asphalt concrete aggregate was increased to a minimum of 60 percent of two mechanically fractured faces, and plasticity index requirements which presently call for sandy, nonplastic material have resulted in a cleaner aggregate with a corresponding increase in the stability of the bituminous surfacing.

Comparison of pavement condition on identical projects on a yearly basis also revealed aging of the asphalt concrete surface course in certain areas. The aging process was retarded on projects which had received an early chip-seal treatment. Chip sealing did not prove satisfactory due to bleeding; it also chipped windshields and caused a general inconvenience to the traveling public, especially on roads carrying a high volume of traffic. Because of this, New Mexico eliminated sealing on most of the asphalt surface courses.

Recently a ½-in. plant mix seal coat was used on numerous projects with satisfactory results. The specifications require a minimum of 75 percent of the material retained on the No. 4 screen to have at least two fractured faces. The gradation requirements are as follows:

Sieve	Percent Passing
3/8 in.	100
No. 4	30-50
No. 10	5-25
No. 40	0-12
No. 200	0- 6

The key to a successful seal with this mix is the asphaltic content. New Mexico uses 6 1/2 to 7 percent of 60 to 70 or 85 to 100 asphalt cement by weight of the total mix. It is necessary to lay the mix at lower temperatures than the usual asphaltic concrete mixes to retain a thick film over the aggregate particle. The asphalt flows gradually down to the surface of the mat and completes the seal. Warm weather and traffic are necessary to accomplish the best results.

For some unknown reason, surfaces constructed with this design and tested with the U. S. Bureau of Public Roads roughometer show a lower reading (by about 20 in. per mile of deviation) than the normal hot-mix surface. The mix when properly laid has a dense terra cotta effect and provides an excellent nonskid surface. Allowance is made for the 1/2-in. layer in the final thickness determinations (Fig. 8).

The post-condition surveys constitute a continuing research project, and permit adoption of improved practices. A vast pool of factual data has been accumulated. The questions are multiple and the answers are elusive, but through the process of analyzing the changes which are occurring we believe that better serviceability can be attained.

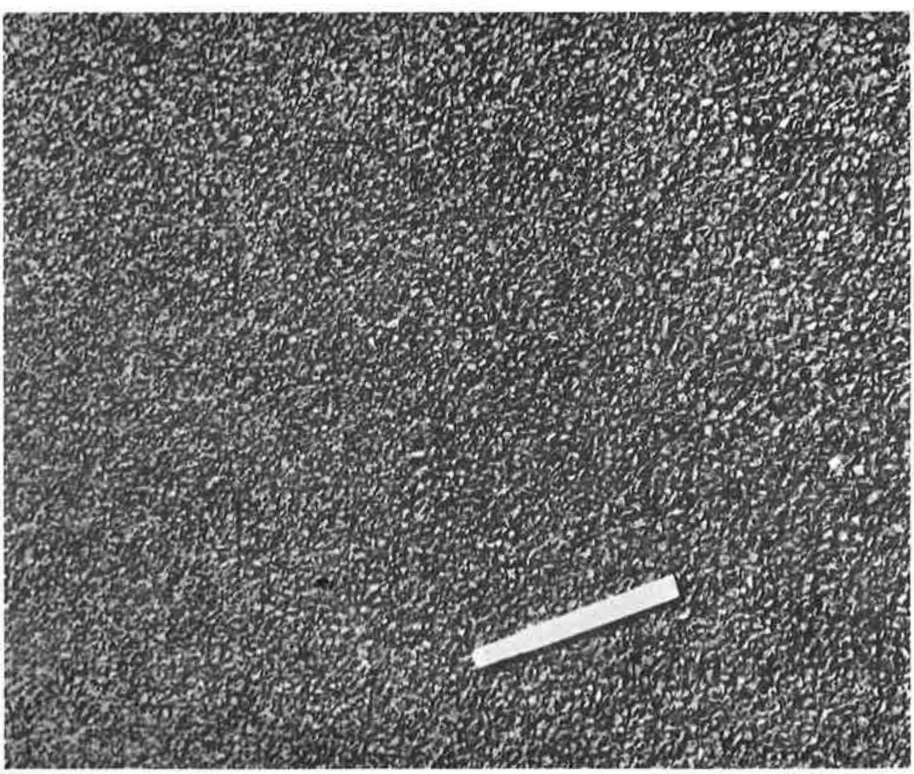

Figure 8. PI mix seal; Arizona state line-road forks.

ACKNOWLEDGMENTS

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REFERENCES

1. Chastain, W. E., Sr., and Burke, John E. Experience with a BPR-Type Roadometer in Illinois. Highway Research Board Bull. 328, pp. 52-58, 1962.
2. AASHO. Uniform Traffic Control Devices for Streets and Highways. p. 22, 1961.