CHAPTER 8

SURFACE AND OTHER CONSIDERATIONS

8.1 Rideability Specifications, December 17, 1987.

8.2 A Selection of Measuring Equipment Used to Monitor and Enforce Rideability Specifications, Technical Paper 88-03, May 24, 1988.

8.3 TA 5040.17, Skid Accident Reduction Programs, December 23, 1980.

8.4 TA 5140.10, Texturing and Skid Resistance of Concrete Pavement and Bridge Decks, September 18, 1979.

8.5 TA 5040.31, Open-Graded Asphalt Friction Course, December 26, 1990.

8.6 Automatic Profile Index Computation, February 21, 1991.
- Analysis and Recommendations Concerning Profilograph Measurements in South Dakota, November 1990.

8.7 Measurements, Specifications, and Achievement of Smoothness for Pavement Construction, NCHRP No. 167, 1990.

8.8 A Half Century with the California Profilograph, Report Number FHWA-AZ-SP9102, February 1992.

Memorandum

U.S. Department of Transportation
Federal Highway Administration

Washington, D.C. 20590

Subject: Rideability Specifications

Date: DEC 17 1987

From: Director, Office of Highway Operations

Reply to Attn of: HHO-12

To: Regional Federal Highway Administrators
Direct Federal Program Administrator

Smoothness has become the primary measure by which the traveling public determines and evaluates the quality of both newly constructed and rehabilitated pavements. In the spring of 1987, the Pavement Division assisted the Rideability Task Force of the American Association of State Highway and Transportation Officials (AASHTO) Subcommittee on Construction by developing and compiling a survey on rideability specifications being used by the States. We have attached a copy of the draft survey summary report for your use.

Based on the survey results, the task force proposed a revised PCC rideability specification and a new AC rideability specification at the Subcommittees mid-year meeting. These specifications were balloted on in the fall of this year and received a two-thirds majority approval. They were subsequently approved by both the Standing Committee on Highways and the Executive Committee of AASHTO at their December meeting and will be incorporated into the new 1988 AASHTO Guide Specifications for Highway Construction when published. Copies of the specifications as balloted are attached for your information.

Key elements of both proposed guide specifications include:

1. The measurement device is limited to only the California-type profilograph.

2. Methods for the evaluation of both profiles and bumps are specified as California Test 526.

3. The acceptance level for the profile index was revised from 12 to 10 inches per mile using the California type profilograph with a 0.2 inch blanking band.

4. The minimum day's paving length for which a profile is taken is established as 0.1 mile, consistent with the evaluation length.

5. The size of the "must grind" bump was increased to 0.4 inch in 25 feet.

6. Two pay adjustment schedules are provided: 1) establishes disincentives only on a sliding scale to a maximum reduction of 10 percent at 15 ipm. 2) an optional schedule, establishes both incentives and the disincentives on a sliding scale of a maximum reduction of 10 percent at 15 ipm and a maximum increase of 5 percent at 3 ipm.

We believe that adopting rideability specifications pays significant dividends to all elements of the highway industry in not only providing smoother pavement surfaces, but providing a higher quality product. In addition, it has been demonstrated through the AASHTO design equations that there is a significant direct relation between initial pavement smoothness and design life. Therefore, we strongly recommend that you and your staff work with your respective divisions and States in developing and implementing rideability specifications for both Portland Cement Concrete and Asphalt Concrete. The specifications should generally follow the new AASHTO Guide Specifications. The Pavement Division is available to provide assistance at your request.

David S. Gendell

401.03 Asphalt Concrete

Surface Test. Method #1. The surface will be tested with a 10-foot s-traightedge at locations selected by the Engineer. The variation of the surface from the testing edge of the straightedge between any two contacts, longitudinal or transverse with the surface shall not exceed inch (3/16 to 1/8 suggested). Irregularities exceeding the specified tolerance shall be corrected by and at the expense of the Contractor by removing the defective work and replacing it with new material or by an overlay (not patching), or by grinding/cold milling as directed by the Engineer. Following correction, the area shall be retested to verify compliance with the specified tolerances.

Profilograph Surface Test. Method #2. The smoothness of the pavement will be determined by using a profilograph over each designated lane. The surface of mainline pavement where the design speed will be 40 miles per hour (MPH) or higher will be tested and shall be corrected by the Contractor to a smoothness as follows.

If the final surface course is a friction course or other special purpose pavement layer, this specification, including corrective actions and pay adjustments, shall be applied to the pavement layer placed prior to the final surface course. The Contractor shall place the final surface course so the profile index of the final surface course is less than or equal to the profile index of the preceding pavement layer.

Equipment - The profile index will be determined using a California type profilograph furnished and operated by the Department. The profilogram is recorded on a scale of 1 inch, or full scale, vertically. Motive power may be manual or by a propulsion unit attached to the assembly. The profilograph will be moved longitudinally along the pavement at a speed no greater than 3 MPH to minimize bounce. The results of the profilograph tests will be evaluated as outlined in California Test 526.

Surface Test - The Contractor shall furnish paving equipment and employ methods that produce a riding surface having a profile index of 10 inches per mile or less, except as provided for in subsequent paragraphs. Initial profiles up to 15 inches per mile may be accepted with applicable Price Adjustments. The profile will terminate 15 feet from each bridge approach pavement or existing pavement that is joined by the new pavement.

Pavement profiles will be taken 3 feet from and parallel to each edge of pavement for pavement placed at a 12-foot width or less. When pavement is placed at a greater width than 12 feet, the profile will be taken 3 feet from and parallel to each edge and from the approximate location of each planned lane marking. Additional profiles may be taken only to define the limits of an out-of-tolerance surface variation.

During the initial paving operations, either when starting up or after a long shut-down period, the pavement surface will be tested with the profilograph as soon as the final rolling has been completed. Initial testing will be used by the Contractor and the Engineer to evaluate the paving methods and equipment.

If the initial pavement smoothness, paving methods, and paving equipment are acceptable to the Engineer, the Contractor may proceed with the paving operation. After initial testing, profiles of each day's paving will be run prior to continuing paving operations on prior to opening the pavement to public traffic.

A daily average profile index will be determined for each day's paving. A day's paving is defined as a minimum of 0.1-mile of full-width pavement placed in a day. If less than 0.1-mile is paved, the day's production will be grouped with the next day's production. If an average profile index of 15 inches per mile is exceeded in any daily paving operation, the paving operation will be suspended and will not be allowed to resume until the Contractor takes corrective action. In the event that paving operations are suspended as a result of the average profile index exceeding 15 inches per mile, subsequent paving operations will be tested in accordance with the initial testing procedures.

For determining pavement section where corrective work or pay adjustments will be necessary, the pavement will be evaluate in 0.1 mile sections using the profilogram. Within each 0.1-mile section, all areas represented by high points having deviations in excess of 0.4 inches in 25 feet or less shall be corrected by the Contractor. After correcting individual deviations in excess of 0.4 inches in 25 feet, corrective action shall be made to reduce the profile index to 10 inches per mile or less.

In addition, any 0.1 mile section having an initial profile index in excess of 15 inches per mile shall be corrected to reduce the profile index to 10 inches per mile or less.

On those sections where corrections are made the pavement will be tested to verify that corrections have produced a profile index of 10 inches per mile or less.

Corrective actions shall be made at the Contractors expense. All corrective work shall be completed prior to determining the pavement thickness. Corrections made by cold milling, by diamond grinding, by overlaying, or by removing and replacing, shall be as directed by the Engineer in accordance with the following:

(1) Cold Milling/Grinding

Cold Milling/grinding shall be performed by the Contractor until the required surface tolerances are achieved. Cold milling/grinding shall be performed so a uniform cross-section is produced. All milled areas shall be neat and of uniform surface appearance.

(2) Overlaying

Asphaltic concrete pavement overlays shall meet all the requirements specified in the Contract. The overlay lift shall extend the full width of the underlying pavement surface and have a finished compacte thickness sufficient to correct the roughness and produce a final surface meeting specified surface tolerances.

If the overlay does not meet the longitudinal smoothness requirement, a second overlay will not be allowed. The repairs to an overlay not meeting smoothness requirements shall be made by the Contractor as directed by the Engineer

(3) Removing and Replacing

Corrections made by removal shall be replaced by asphalt concrete pavement materials meeting the requirements specified in the contract.

Price Adjustments - When the profile index does not exceed 10 inches per mile per 0.1 mile section, payment will be made at the contract unit price for the completed surface course. When the profile index exceeds 10 inches per mile per 0.1 mile section but does not exceed 15 inches per mile per 0.1 mile section, the Contractor may elect to accept a contract unit Price Adjustment in lieu of reducing the profile index. Contract unit Price Adjustments will be made in accordance with the following schedule.

Profile Index Inches per mile per 0.1-mile section	Contract Unit Price Adjustment Percent of pavement unit bid price
Less than (10)	100
Over 10 to 11	98
Over 11 to 12	96
Over 12 to 13	94
Over 13 to 14	92
Over 14 to 15	90
Over 15	Corrective work required

This unit bid Price Adjustment will apply to the total theoretical tonnage representing the total thickness of the asphaltic pavement structure of the 0.1-mile-long section for the lane width represented by the profilogram.

The above Price Adjustment schedule will apply to pavement sections where corrective work has been completed.

Pay adjustments with incentives. Method #3. When the profile index is greater than 7 inches per mile but does not exceed 10 inches per mile per 0.1 mile section, pavement will be made at the contract unit price for the completed surface course. When the profile index exceeds 10 inches per mile per 1.0 mile section but does not exceed 15 inches per mile per 0.1-mile section, the Contractor may elect to accept a contract unit Price Adjustment in lieu of reducing the profile index. When the profile index is less than or equal to 7 inches per mile, the contractor will receive an incentive payment.

Contract unit price adjustments will be made in accordance with the following schedule.

Profile Index Inches per mile per 0.1-mile section	Contract Unit Price Adjustment Percent of pavement unit bid price
3 or less	105
Over 3 to 4	104
Over 4 to 5	103
Over 5 to 6	102
Over 6 to 7	101
Over 7 to 10	100
Over 10 to 11	98
Over 11 to 12	96
Over 12 to 13	94
Over 13 to 14	92
Over 14 to 15	90
Over 15	Corrective work required

Pay adjustments for incentives will only be based on the initial measured profile index, prior to any corrective work. The Price Adjustment schedule for 100 percent pay or pay reductions apply to pavement sections where corrective work has been completed.

This unit bid Price Adjustment will apply to the total theoretical tonnage representing the total thickness of the asphaltic pavement structure of the 0.1-mile-long section for the lane width represented by the profilogram.

501.03 Portland Cement Concrete

Surface Test. Method #1. The surface will be tested using a 10-foot straightedge at locations selected by the Engineer. The variation of the surface from the testing edge of the straightedge between any two contacts, longitudinal or transverse with the surface, shall not exceed 3/16 inch. Irregularities exceeding the specified tolerances shall be corrected by and at the expense or the Contractor with an approved profiling device or by other means as directed by the Engineer. Following correction the area will be retested to verify compliance with the specified tolerances.

Profilograph Surface Test. Method #2. The smoothness of the pavement will be determined by using a profilograph over each designated lane. The surface finish of mainline pavement where the design speed will be 40 miles per hour (MPH) or higher shall be tested and corrected to a smoothness as follows:

Equipment - The profile index will be determined using a California type profilograph finished and operated by the Department. The profilogram is recorded on a scale of 1 inch, or full scale, vertically. Motive power may be manual or by a propulsion unit attached to the assembly. The profilograph will be moved longitudinally along the pavement at a speed no greater than 3 MPH to minimize bounce. The results of the profilograph tests will be evaluated as outlined in California Test 526.

Surface Test - The Contractor shall furnish paving equipment and employ methods that produce a riding surface having a profile index of 10 inches per mile or less, except as provided for in subsequent paragraphs. Initial profiles up to 15 inches per mile may be accepted with applicable Price Adjustments. The profile will terminate 15 feet from each bridge approach pavement or existing pavement that is joined by the new pavement.

Pavement profiles will be taken 3 feet from and parallel to each edge of pavement for pavement placed at a 12-foot width or less. When pavement is placed at a greater width than 12 feet, the profile will be taken 3 feet from and parallel to each edge and from the approximate location of each planned longitudinal joint. Additional profiles may be taken only to define the limits of an out-of-tolerance surface variation.

During the initial paving operations, either when starting up or after a long shut-down period, the pavement surface will be tested with the profilograph as soon as the concrete has cured sufficiently to allow testing. Membrane curing damaged during the testing operation shall be repaired by the Contractor as directed by the Engineer. Initial testing will be used to aid the Contractor and the Engineer to evaluate the paving methods and equipment.

If the initial pavement smoothness, paving methods, and paving equipment are acceptable to the Engineer, the Contractor may proceed with the paving operation. After initial testing, profiles of each day's paving will be run prior to continuing paving operations.

A daily average profile index will be determined for each day's paving. A day's paving is defined as a minimum of 0.1-mile of full-width pavement placed in a day. If less than 0.1-mile is paved, the day's production will be grouped

The unit bid adjusted price will be computed using the planned thickness of portland cement concrete pavement. This unit bid adjusted price will apply to the total area of the 0.1-mile section for the lane width represented by the profilogram.

The above Price Adjustment Schedule will apply to pavement sections where corrective work has been completed.

Pay adjustments with incentives. Method #3. When the profile index is greater than 7 inches per mile but does not exceed 10 inches per mile per 0.1-mile section, payment will be made at the Contract unit price for the completed pavement. When the profile index exceeds 10 inches per mile per 0.1-mile section but does not exceed 15 inches per mile per 0.1-mile section, the Contractor may elect to accept a contract unit adjusted price in lieu of reducing the profile index. When the profile index in less than or equal to 7 inches per mile, the Contractor is entitled to an incentive payment. Contract unit Price Adjustments will be made in accordance with the following schedule in those cases when the Contractor is entitled to incentive payments or elects to accept contract unit Price Adjustments in lieu of reducing the profile index.

Index Profile Inches per mile per 0.1-mile section	Price Adjustment Percent of pavement unit bid price
3 or less	105
Over 3 to 4	104
Over 4 to 5	103
Over 5 to 6	102
Over 6 to 7	101
Over 7 to 10	100
Over 10 to 11	98
Over 11 to 12	96
Over 12 to 13	94
Over 13 to 14	92
Over 14 to 15	90
Over 15	Corrective work required

Pay Adjustments for incentives will only be based on the initial measured profile index, prior to any corrective work. The Price Adjustment schedule for 100 percent pay or pay reductions apply to pavement sections where corrective work has been completed.

The unit bid adjusted price will be computed using the planned thickness of portland cement concrete pavement. This unit bid will apply adjusted price to the total area of the 0.1-mile section for the lane width represented by the profilogram.

TECHNICAL PAPER 88-03

A SELECTION OF MEASURING EQUIPMENT USED TO MONITOR AND ENFORCE RIDEABILITY SPECIFICATIONS

I. INTRODUCTION —

There has been a large amount of information provided concerning the use and benefits of rideability specifications recently. This paper is to help FHWA engineers in the field become more familiar with the various types of equipment available to enforce these specifications. Consequently, the following information is provided to give the engineers an overview of the equipment characteristics, operational and calibration methods, and costs. We have tried to provide this information for the most commonly used devices and the inclusion or omission of any particular device should not be construed as an endorsement or disapproval.

Comparability of measurements from the different pieces of equipment is always a question. There have been a number of research studies done, and several underway to address this question. So far, it is sufficient to say that correlations have been developed between several of the following equipment types. Many of these studies contain restrictions and/or limitations which should be understood before using the correlations.

II. PROFILOGRAPHS —

Traditional California Type Profilograph — characterized by an aluminum truss frame 25 feet long. At each end is mounted a wheel assembly consisting of six caster wheels. In the middle of the frame a profile wheel is mounted directly under and connected to a graphic recorder. See figure 1. The truss breaks down into three major components plus the graphic recorder. This allows easy transportation in a standard sized pickup truck or van. Assembly requires two persons about 15 minutes. The cost of the traditional California Type Profilogragh ranges from about $12,000 to over $25,000 for a model which includes a microprocessor for data reduction. This type profilograph is manufactured by James Cox & Sons, Inc., Mc Cracken Concrete Pipe Co., Mc Beth Engineering Corp., and Thompson-Quill Assoc.

Ames California Type Profilograph — this device is characterized by an aluminum box beam 25 feet long as the reference plane. Mounted at each end of the beam is a wheel assembly the same as the traditional California type device, with six caster wheels. The Ames profilograph has both a bicycle wheel and the graph recorder assembly mounted at the rear of the device. This wheel drives the graph recorder and is not involved in recording the vertical deviations. The Ames device uses a six-inch caster wheel mounted mid beam as the profile measuring wheel. See figure 2. As with the traditional California type this device also brakes down into three major components plus the graph recorder assembly for easy transportation. The type of profilograph costs about $7,000 and is manufactured by Ames Profilograph.

Rainhart Profilograph — this device is characterized by an aluminum frame which has four triangular subframes mounted equidistant along it. Each of the

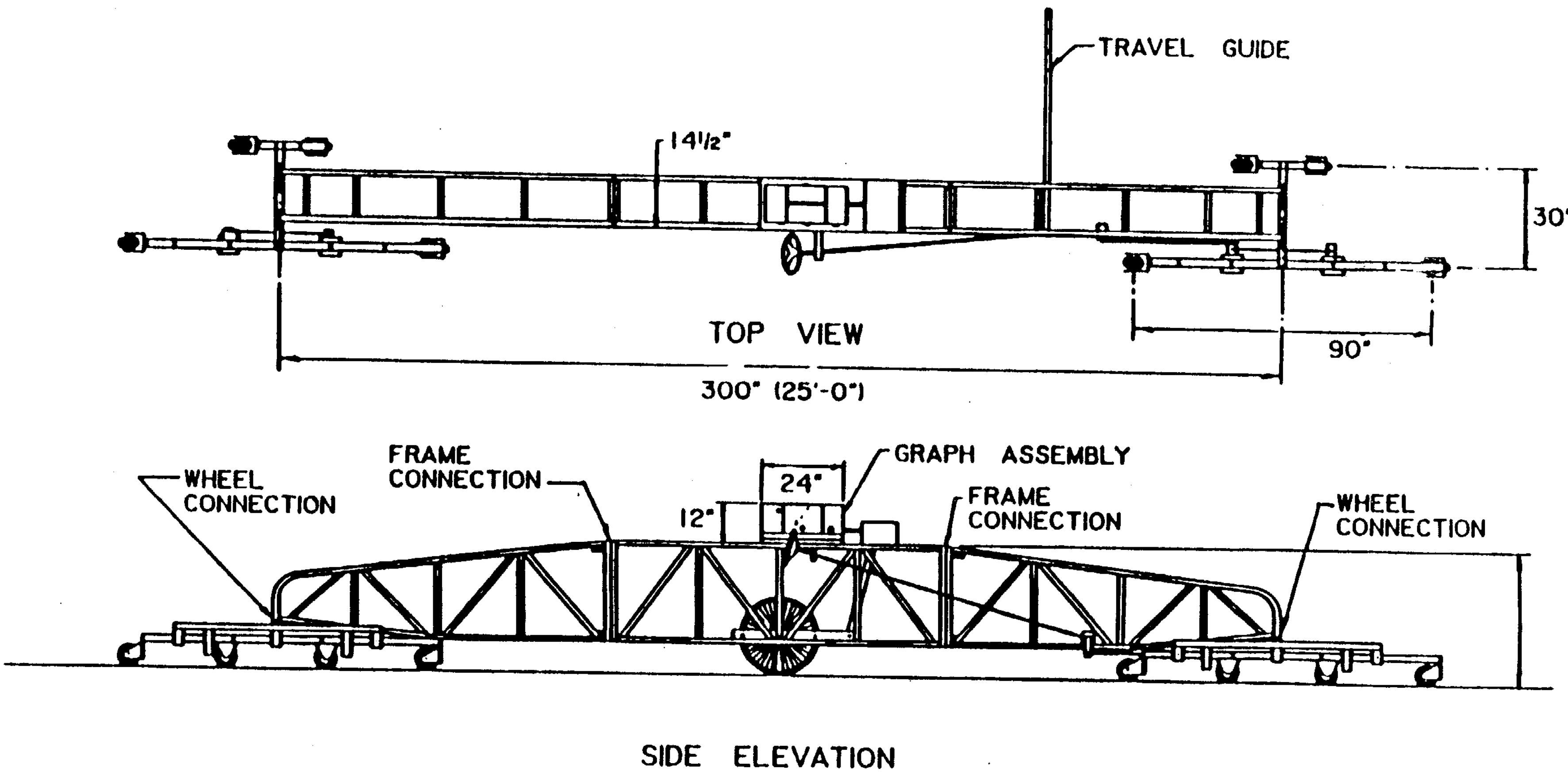
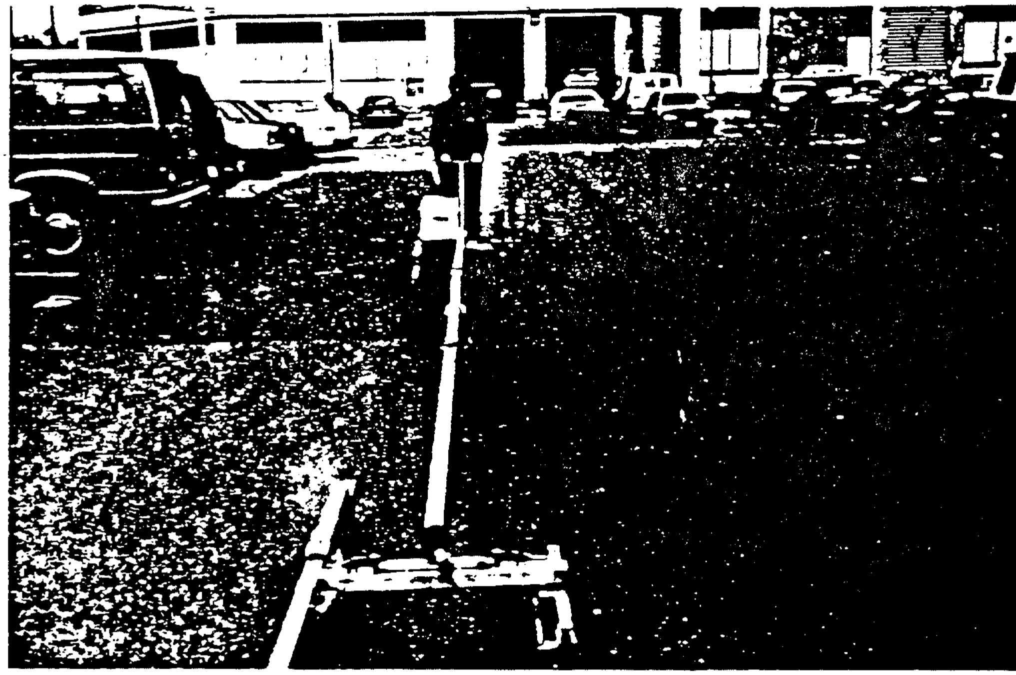
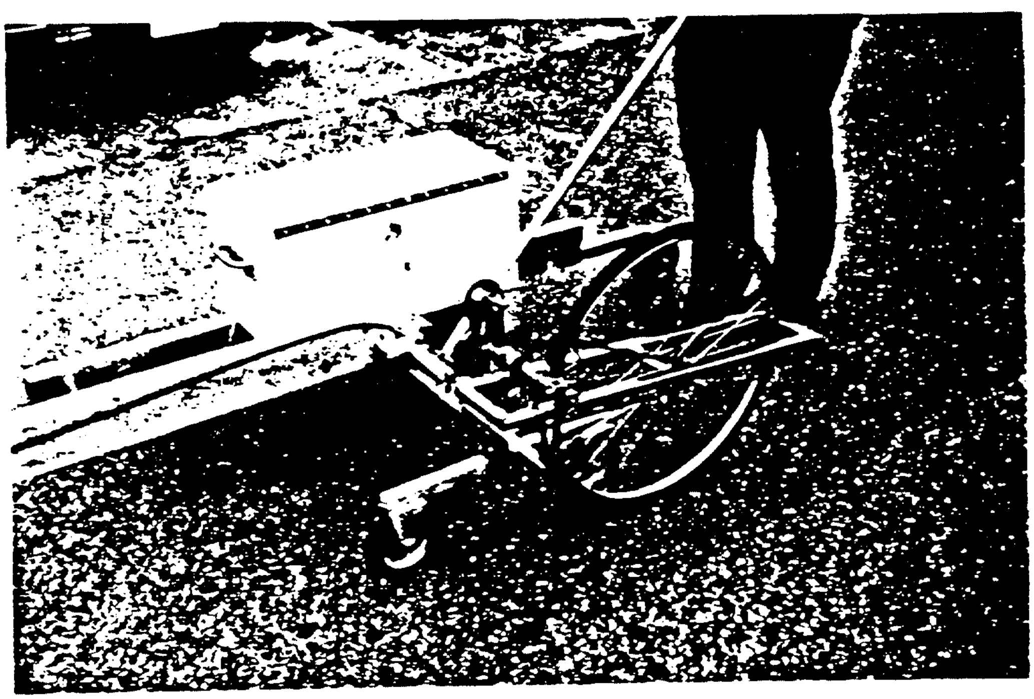


Figure 1. Schematic of California type profilograph.

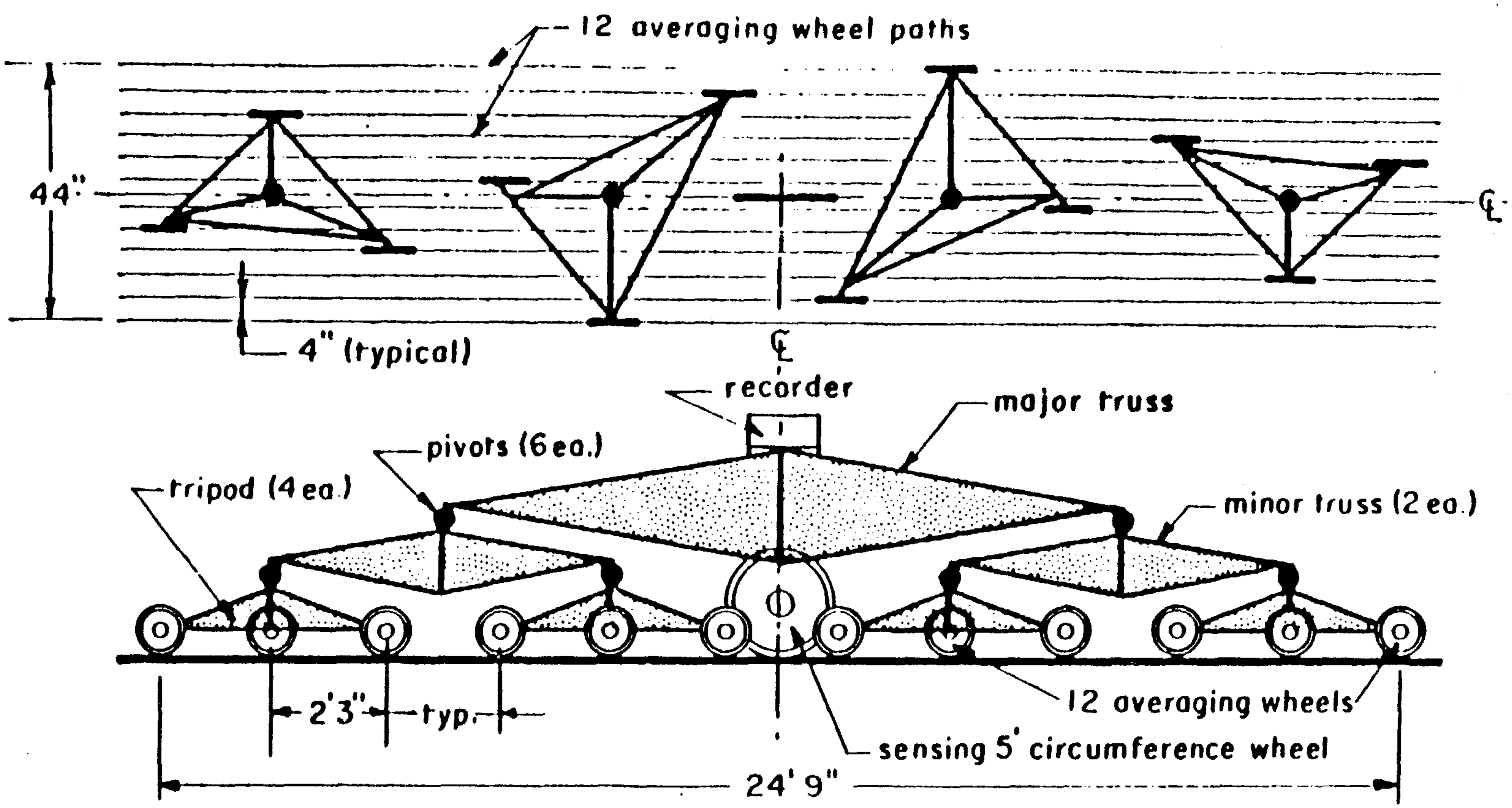


Front View Ames California Profilograph



View of recording box
Ames California Profilograph

Figure 2



Any averaging wheel lifted 1" = tripod apex up $\frac{1"}{3}$ = minor truss ℄ up $\frac{1"}{6}$ = major truss ℄ (recorder) up only $\frac{1"}{12}$ (0.083")

Figure 3 Schematic of Rainhart Profilograph subframes has three wheels mounted such that no two wheels follow the same path. The Rainhart device has a profile wheel mounted directly under and connected to a graphic recorder in the middle of the frame. See figure 3. The Rainhart Profilograph does not break down into subassemblies for transportation as do the other profilographs. Instead, the Rainhart provides an auxiliary wheel assembly which locks down making the device into a trailer for ready transportation. This device is manufactured by the Rainhart Company and costs about $11,000.

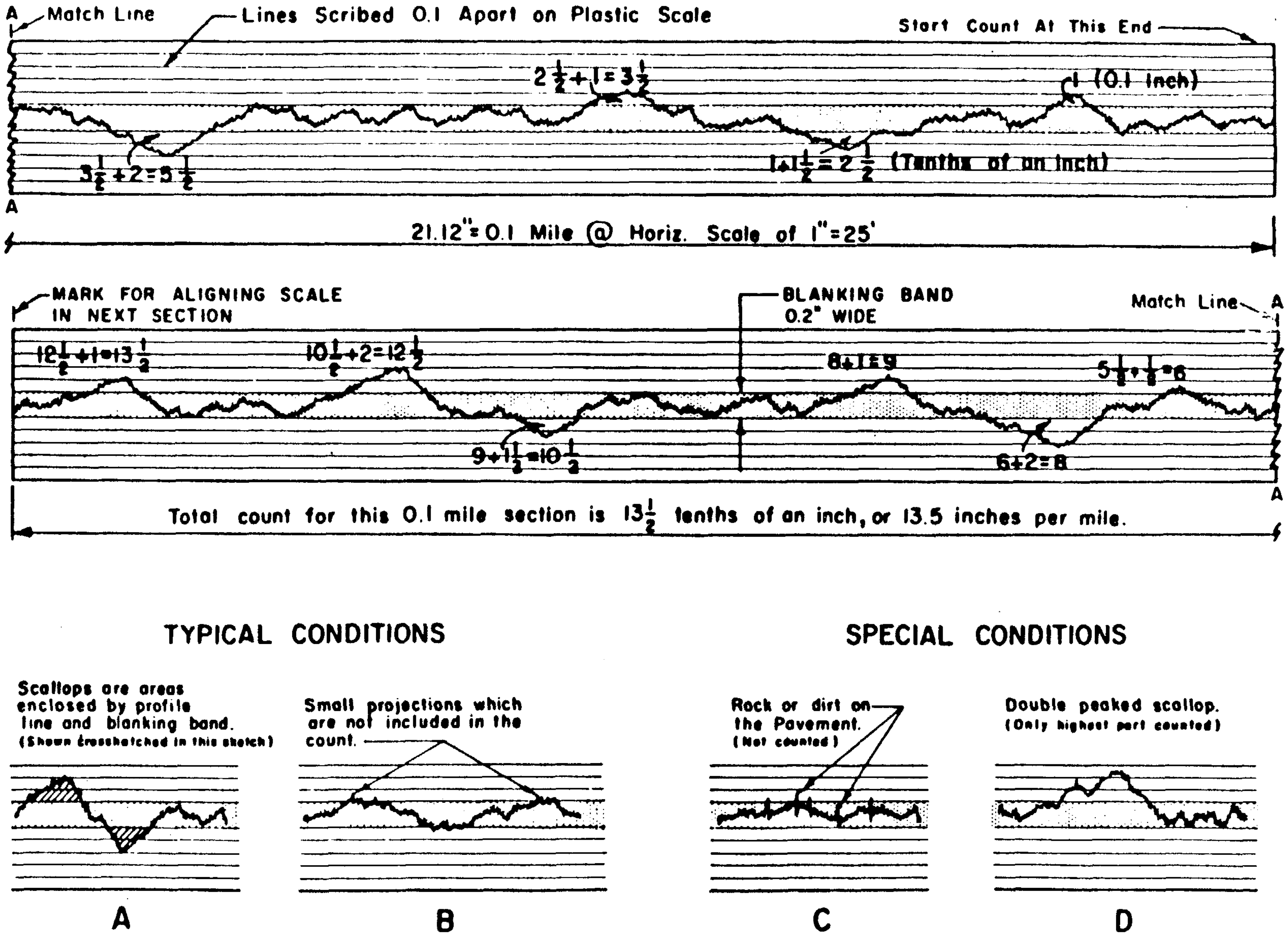
The calibration, operation, and methodology for reducing the data is generally the same for all the profilographs, we will address the details generically.

Calibration of the profilograph is relatively simple because they are simple machines where little can go wrong. There are two basic calibrations. One is the horizontal distance and the other is the vertical deviations. The horizontal distance is measured by the bicycle tire rolling along the pavement. This movement is transferred to the graph recorder by a standard bicycle chain. The only adjustments are in tire pressure, tire trueness, and a some models allow adjustments to the gearing at the graph recorder. To calibrate horizontally, a known distance should be measured with the profilograph. If the measured distance does not correspond to the known distance, adjustments should be made. Vertical deviations in the pavement are measured by the wheel mounted in the center of the profilograph raising and lowering. This movement is transferred to the pen in the graph recorder by cables. There is usually little adjustment available to the cable mechanism. To calibrate vertically, blocks of known height are placed under the recording wheel and the pen in the graph recorder checked to ensure a like height is recorded.

A two person crew is required to measure a profile of the roadway. One person pushes and steers the profilograph while the other marks events on the graph recorder and ensures the profile is being properly recorded. Because these devices are only able to operate at low speeds, i.e., 2-3 miles per hour, extreme care must be exercised while operating in traffic. Some form of traffic control must be an element of the normal operation if the roadway is open to traffic. Most specifications require that the profilograph measure the roughness in the wheel paths, with the measurements of the two wheel paths being averaged for the lane. Events, such as mileposts, structures etc. and the direction of travel should be recorded on the graph.

The data reduction process or trace evaluation is an activity which requires training in order to ensure repeatability. A special blanking band is placed over the trace so that a maximum amount of trace is covered by the 0.2 (0.1 recommended for the Rainhart profilograph) inch opaque band. A nearly equal amount of deviations will show above and below the opaque band when the blanking band is positioned correctly. See figure 4. The height of each deviation is determined to the nearest 0.05 inch. The value is recorded above each deviation. Multi-peaked deviations are considered only once with the highest peak measured. Deviations with no width (spikes) represent chunks of mortar, rocks, texture or wheel bounce and are not counted. The sum of the deviation heights is counted and divided by the distance. The profile index is expressed in inches per mile. Profile Index = total count (inches)/length of profile (miles).

EXAMPLE SHOWING METHOD OF DERIVING PROFILE INDEX FROM PROFILOGRAMS



ıre 4

Studies have shown that the traditional California type and the Ames device produce a tracing which is virtually identical. Because of the different wheel configuration on the Rainhart, it produces a different looking tracing when compared to the other two devices. The results of the California type profilographs cannot be directly correlated to that of the Rainhart device.

The American Society of Testing and Materials (ASTM) is in the process of developing a standard test method of measuring pavement roughness using a profilograph. We will provide copies of the standard when available.

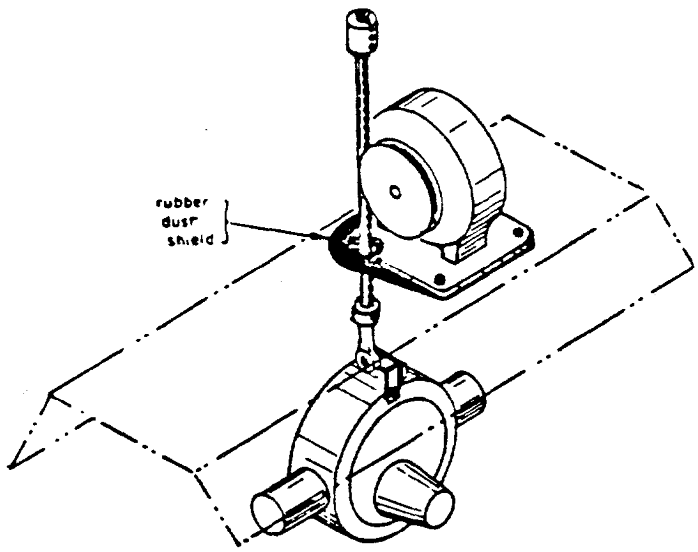
III. RESPONSE TYPE ROAD ROUGHNESS MEASURING (RTRRM) DEVICES —

Response type road roughness measuring devices operate at highway speeds. These devices are mounted in a vehicle (trailer) and measure the response (bounce) of the vehicle to the road roughness. It is therefore not a true measurement of roughness. Included in this category of equipment are devices that measure the relative axle-body motion and devices that measure the acceleration of the axle or the vehicle body.

Mays Ride Meter — this device determines the roughness of the roadway by measuring the displacement between the axle housing and the body of the test vehicle. The method actually measures the relative motion of a sprung mass system in response to traveled surface roughness where the mass is supported by automotive type suspension and tires. Their are other types of response devices, but the Mays Ride Meter is by far the most commonly used today.

The major components of the Mays Ride Meter is the rotary transducer, the pavement condition recorder, and the distance measuring instrument. The rotary transducer converts the axle/body movement to an electrical signal. The distance measuring instrument is an electronic odometer. The pavement condition recorder is a microprocessor which accepts input from the rotary transducer, the distance measuring instrument, and a keyboard processes the various signals into an output. This output is commonly in the form of accumulated inches of relative motion over a distance.

The Mays Ride Meter can be mounted in either a standard passenger car or a trailer. Experience has shown that many characteristics in a passenger car can affect the roughness reading. Consequently, it is now common to mount the Mays Ride Meter components in a trailer. See figure 5. However, there are still items within the trailer which can cause potential inaccuracy. These items include: shock absorbers, tire roundness, tire balance, tire pressure, condition of springs, loose wheel bearings, play in tow/sway bar assembly, and wind. It has also been found that the roughness readings are temperature sensitive, therefore readings should not be made at temperature extremes. The trailer must receive preventative maintenance on a regular basis. The tires must be trued and balanced. Wheel bearing should be checked and periodically repacked. Tire pressure should be checked several times a day and maintained at a constant hot pressure. It is also very important to assure that the shock absorbers be maintained in good condition and replaced only with a standard type. Also new shock absorbers should be subjected to a break in period and should be warmed up daily for a prescribed distance. Another factor affecting



Detail of Mays Ride Meter

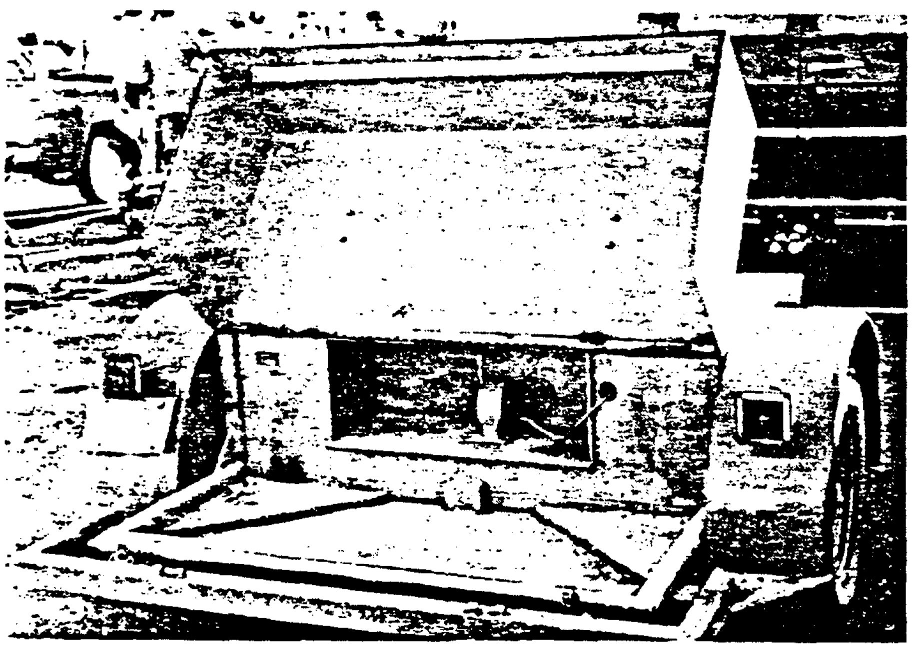


Figure 5 Trailer Mounted Mays Ride Meter

repeatability is the speed at which the test is run. It is very important to maintain the vehicle at a constant speed. A speed of 50 miles per hour is commonly used in rural areas, but the device can be operated at slower speeds. Calibration tests must be done at all speeds at which roughness measurements will be recorded.

Most users of the Mays Ride Meter use extensive calibration procedures to increase the potential for repeatability of each Ride Meter and the comparability between two different Ride Meters. One method of calibration (Highway Performance Monitoring System Field Manual) consists of both a periodic control check to assure the components of the device are operating properly and an annual calibration based on a number of control sections. These control sections should be approximately 0.2 mile in length and be relatively straight and level with a low ADT. The low ADT is important because to use in-service control sections the assumption has to be made that the roughness measurements do not significantly change during a years time. It is extremely important to recalibrate the device anytime a component is replaced or repaired. Further information is available in Appendix J of the HPMS Manual.

The cost of the May Ride Meter Trailer and associated strip chart recorder manufactured by the Rainhart Company is about $8,000. The price of the Pavement Condition Recorder is about $10,000. The operation requires a two man crew and costs about $40.00 per hour including the cost of the tow vehicle.

ASTM Standard Test Method E1082-85, Measurement of Vehicular Response to Traveled Surface Roughness is available to more fully discuss the details of the equipment and its operation.

IV. ROAD ROUGHNESS PROFILING DEVICES —

Profiling devices measure and record the longitudinal profile in one or both wheel tracks. In the United States the inertial type profiling devices are used. Devices in this category of equipment include the K.J. Law profilometer, the FHWA PRORUT system and the South Dakota profiling device.

Inertial Profilometer — these devices are capable of measuring and recording road surface profiles at speeds between 10 and 55 miles per hour. The devices utilize the inertial reference concept which was developed in the early 1960's at the General Motors Corporation Research Laboratories.

The profilometer measures and computes the longitudinal profile of the pavement through the creation of an inertial reference by using accelerometers placed on the body of the measuring vehicle. Relative displacement between the accelerometers and the pavement surface is measured with a non-contact light or acoustic measuring system mounted with the accelerometer on the vehicle body.

Operation requires a two person crew, one as a vehicle driver and the other as a system operator. The entire system is mounted in a full size van. See figures 6 and 7. The profile computer, data recording and other system components are all contained in the vehicle. The profilometer contains non-

Figure 6 K.J. Law 690 DNC Profilometer

Figure 7 FHWA PRORUT System

contact sensors for measuring road surface profile. The accelerometers establish the reference plane for the profilometer system's measurement by measuring the vertical accelerations of the vehicle body. The distance traveled by the system is measured with a distance encoder. This is usually a pulse type distance measuring device which is mounted to the front wheel of the vehicle.

The profile signal processing is performed by a digital computer which is mounted in the vehicle. Profile computations are performed in real time as the vehicle is driven down the road. Interface between the user and the profilometer system is provided through a system terminal and printer.

Vehicle response simulation programs for roughness index calculations are available with the profilometer system. The selected roughness index is normalized to read inches per mile and is printed out on the system printer. The roughness indices are simulations of standardized response type devices performed by the profilometer computer system and computed from the measured and recorded pavement profile data. The simulations can be used to calibrate response type equipment like the Mays Ride Meter or the roughness indices values can be used as the measured roughness statistic.

The cost of a Non-Contact Inertial Profilometer varies with the level of precision. The K.J. Law profilometer is the most precise device and is commercially available from K.J. Law Engineers for $250,000 to $300,000. The FHWA PRORUT system measures the profile in both wheel tracks and the average rut depth. To date, one prototype of the PRORUT has been constructed for a cost of $100,000 to $150,000. The South Dakota device us inexpensive ultrasonic sensors which are less precise. It measures the profile in one wheel path and the average rut depth. The South Dakota device is estimated to cost $50,000.

ASTM Standard Test Method E950-83, Measuring the Longitudinal Profile of Vehicular Traveled Surface with an Inertial Profilometer is available to more fully discuss the details of the equipment and its operation.

Additional information is available from Bruce E. Matzke, HHO-12, at 366-1342.

HHO-12/5-24-88

PARTIAL LISTING OF

EQUIPMENT MANUFACTURERS

Profilographs:

James Cox & Sons
PO Box 674
Colfax, California 95713

McCraken Concrete Pipe Company
PO Box 1708
Sioux City, Iowa 51102-1708

Rainhart Company
PO Box 4533
Austin, Texas 78765

Ames Profilograph
200 Rockwell Avenue
Ames, Iowa 50010

Response Type Road Roughness Measuring Devices:

Mays Ride Meter
Rainhart Company
PO Box 4533
Austin, Texas 78765

Road Roughness Profiling Devices:

K.J. Law Engineers, Inc.
23660 Research Drive
Farmington Hills, Michigan 48024

Mr. David Huff
South Dakota Department of Transportation
700 Broadway Avenue East
Pierre, South Dakota 57501-2586

U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION

SUBJECT
SKID ACCIDENT REDUCTION PROGRAM

FHWA TECHNICAL ADVISORY
T 5040.17
December 23, 1980

1. PURPOSE. To provide guidance for State and local highway agencies in conducting skid accident reduction programs.

2. BACKGROUND

a. This Technical Advisory provides a general overview of factors that should be considered as elements of any Skid Accident Reduction Program. This Technical Advisory supports current Federal Highway Administration (FHWA) policy and will be revised as appropriate to reflect changes in policy as they occur.

b. The existing requirements for skid resistance pavements are contained in several documents including Highway Safety Program Standard No. 12, Highway Design Construction and Maintenance (23 CFR 1204.4), Federal Highway Program Manual (FHPM) 6-2-4-7, Skid Measurement Guidelines for the Skid Accident Reduction Program. Other sources of technical advice are cited in the appropriate sections of this Technical Advisory.

c. Highway Safety Program Standard 12 (HSPS No. 12) states that every State shall have a program of highway design, construction, and maintenance to improve highway safety. This program shall provide that "there are standards for pavement design and construction with specific provisions for high skid resistance qualities." The HSPS No. 12 also requires that each State have a "program for resurfacing or other surface treatment with emphasis on correction of locations or sections of streets and highways with low skid resistance and high or potentially high accident rates susceptible to reduction by providing improved surfaces." In discharging the responsibilities of FHWA, the Division Administrator

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should determine the acceptability of specification requirements and construction practices for placing, consolidating, and finishing both asphalt concrete and portland cement concrete pavements. Such determinations will rely on the highway agency to research, evaluate, and document the performance of the various aggregates, mix designs, and construction practices used.

d. Even though the use of studded tires is beyond the control of most highway agencies, their use can cause significant wear on the pavement surface texture. For example, grooves sawed in concrete pavements have worn completely down in as short a time as 2 years. States are encouraged to ban or restrict the use of studded tires.

e. Legislative actions in recent years support a general duty of any highway agency to "... maintain the roadway in a reasonably safe condition. This would involve, in essence inspection, anticipation of defects, and conformity with generally accepted standards and practices."* The practical result is that highway agencies should have an organized system to identify and correct hazardous locations in a cost-effective manner, as well as a comprehensive pavement management program to design, construct, and maintain highways in conformance with reasonable standards. Such a systematic process is the best way to execute the highway agency's duty to maintain a reasonably safe roadway.

3. <u>SKID ACCIDENT REDUCTION PROGRAM</u>. Each highway agency is encouraged to develop and manage a skid accident reduction program to reflect the individual needs and conditions within the State. The purpose of a skid accident reduction program is to minimize wet weather skidding accidents through: identifying and correcting sections of roadway with high or potentially high skid accident incidence; ensuring that new surfaces have adequate, durable skid resistance properties; and utilizing resources available for accident reduction in a cost-effective manner. A program comprised of at least the following three basic activities, if faithfully implemented, should enable the highway agency to comply with HSPS No. 12.

* Engineering and Government Liability, David C. Oliver, FHWA, an unpublished paper presented to the American Road and Transportation Builders Association Local Officials Meeting, St. Louis, Missouri, August 23, 1978.

a. The evaluation of pavement design, construction, and maintenance practices through its pavement management program to ensure that only pavements with good skid resistance characteristics are used.

b. The detection and correction of locations with a high incidence of wet weather accidents utilizing (1) the State and local accident record systems, and (2) countermeasures for locations with high wet weather incidences, to ensure that existing highways are maintained in a safe condition.

c. The analysis of skid resistance characteristics of selected roadway sections to:

(1) ensure that the pavements being constructed are providing adequate skid resistance,

(2) develop an overview of the skid resistance properties of highway systems,

(3) provide up-to-date information for the pavement management process, and

(4) provide data for use in developing safety improvement projects and the implementation of cost-effective treatments at appropriate locations.

4. PAVEMENT DESIGN, CONSTRUCTION, AND MAINTENANCE

a. Pavement Design

(1) Current pavement design practices should be evaluated to ensure that skid resistance properties are durable and suitable for the needs of traffic. Consideration of skid resistance levels, texture, aggregate availability, traffic volume, traffic speed, type of facility, rainfall, construction and maintenance practices, and accident experience are basic elements in such evaluations. Evaluations should document the compliance with the requirement for skid resistant surfaces and provide basic data for use in choosing corrective actions for locations with high wet weather accident rates.

(2) One principal result of the evaluations is the development of a performance history for each particular pavement used by each highway agency. The performance of the existing pavement designs

should be monitored and new designs should be evaluated to ensure that only skid resistant pavement surfaces are used. Information should be gathered as to the durability of a mix and the loss of skid resistance under traffic.

(3) The level of skid resistance needed for a particular roadway depends primarily on the traffic volume, traffic speed, type of facility, and climate with additional consideration warranted at special locations such as steep hills, curves, intersections, and other sites which experience high demands for pavement-tire friction. It is desirable to have one or more "skid resistant mixes" which have durable and higher than usual frictional properties for use in these special areas.

(4) A pavement surface may provide adequate skid resistance at low speeds, yet be inadequate for high speed conditions. Pavement surfaces, therefore, should be designed on the basis of properties at expected operating speeds.

(5) The American Association of State Highway and Transportation Officials (AASHTO) Guidelines for Skid Resistant Pavement Design, 1976, provide detailed information on the design of surfaces for both flexible and rigid pavements. The major considerations follow:

(a) Flexible Pavements

1 The skid resistance evaluation of bituminous pavements should include a determination that the aggregate used in the top layer of future pavements is capable of providing adequate skid resistance properties when incorporated in the particular mix and that the mix should be capable of providing sufficient stability to ensure the durability of the skid resistance.

2 A bituminous pavement surface should contain nonpolishing aggregates. It is essential for good skid resistance that a mix design be used which allows good exposure of the aggregates. This

requires that the pavement surface mixture be designed to provide as much coarse aggregate at the tire-pavement interface as possible.

3 The open graded asphalt friction course (OGAFC), with a large proportion of one size aggregate, provides excellent coarse texture and exposes a large area of coarse aggregate. Guidance for this mix can be obtained from FHWA Technical Advisory T 5040.13, Open-Graded Asphalt Friction Courses, January 11, 1980.

(b) Rigid Pavements

1 The evaluation of portland cement concrete (PCC) pavements should include a determination that the finishing procedures, mix design, and aggregates provide the initial texture and necessary surface durability to sustain adequate skid resistance.

2 In PCC pavements, the initial and early life skid resistance properties depend primarily on the fine aggregates for microtexture and on the finishing operation for macrotexture. Specifications for texturing concrete pavements should be carefully selected and enforced to ensure a macrotexture pattern appropriate to the type of facility.

3 Regardless of the finishing or texturing method used, adequate durable skid resistance characteristics cannot be attained unless the fine aggregate has suitable wear and polish resistance characteristics. Research by the Portland Cement Association indicates that the siliceous particle content of the fine aggregate should be greater than 25 percent.

4 If pavement evaluation studies indicate that the coarse aggregates will be exposed by the surface wear and have a significant effect on skid resistance of pavement, it too should have a suitable polish resistance characteristic.

5 Metal tines, preceded by burlap or another type of drag finish, are recommended as being the most practical and dependable method of providing texture in PCC surfaces. Additional guidance can be obtained from FHWA Technical Advisory T 5140.10, Texturing and Skid Resistance of Concrete Pavements and Bridge Decks, September 18, 1979.

b. Pavement Construction

(1) Highway agencies are encouraged to adopt a policy of "prequalifying" aggregates to be used in surface courses. Prequalifying is a method by which aggregates can be classified according to their friction, texture, wear, and polish characteristics. Classifications should reflect performance related to traffic volume, operating speed, percent trucks, climate, geometric design, and other appropriate factors. Design procedures should be established to ensure that aggregates can be selected for each project which are suitable to the needs of traffic.

(2) Prequalification may be accomplished by one of the following, or a combination of both:

(a) A systematic rating of all fixed sources of aggregates (e.g., a commercial quarry which obtains aggregate from the same location for many years). Ratings should be based on standardized laboratory tests such as the American Society for Testing and Materials (ASTM) D 3319, Recommended Practices for Accelerated Polishing of Aggregates Using the British Wheel, or ASTM D 3042 Test for Insoluble Residue in Carbonate Aggregates, combined with data obtained from skid resistance tests of pavements in service. Other tests may be added or substituted if shown to predict pavement performance.

(b) An evaluation and in-service history of the geologic or petrographic types of aggregates commonly used. Thus, when a new aggregate source is proposed, it can be accepted with minimum testing if an in-service history has been established for that type of aggregate.

(3) Based on prequalification of aggregates, construction plans and specifications should define the friction quality of aggregate which will be acceptable. The following steps should be followed to assure acceptability of the as-constructed pavement surface course:

(a) After the contractor has identified the particular aggregates and asphalt to be used on a project, it is recommended that a mix design be performed with the actual ingredients being used. Aggregates should be checked to determine if they are from prequalified sources or are an acceptable petrographic type.

(b) Macrotexture and void content are important considerations in asphalt mixes. Since asphalts are often blended from several sources of crude oil that vary in temperature-viscosity characteristics, the mixing temperature should be determined for each project after establishing the characteristics of the selected asphalt. Allowable tolerances for asphalt content, mixing temperatures, and gradation should be established for each asphalt mix.

(c) Job control of asphalt mixes should be designed to ensure that desired skid resistance properties are obtained. It should be recognized that small changes in aggregate gradation or asphalt content may significantly affect the macrotexture of finished surfaces.

(4) The frictional properties of pavement surface types should be randomly tested within 6 months after opening to traffic to verify that the anticipated characteristics are present. Evaluation tests should involve direct measures such as the skid tester (ASTM E 274), or an acceptable alternative, but may use surrogate measures such as those which evaluate texture (for example, ASTM E 303, Standard Method for Measuring Surface Frictional Properties Using the

British Pendulum Tester; and sand patch tests as described in the American Concrete Paving Association Technical Bulletin No. 19, Guidelines for Texturing Portland Cement Concrete Highway Pavements, Measurement of Texture Depth by the Sand Patch Method).

(5) In cases where the skid resistance properties of a pavement are found to be questionable or inadequate, appropriate warning signs should be placed immediately as an interim measure. A complete evaluation and any remedial action needed should be effected as soon as possible.

c. Pavement Maintenance. The same procedures and quality standards used in construction should be used in the maintenance operations.

5. WET WEATHER ACCIDENT LOCATION STUDIES. The purpose of this type of study is to identify locations with high incidence of wet weather accidents, determine corrective measures, and take appropriate actions in a timely and systematic manner. This activity should be conducted as part of the highway agency's safety improvement program and should make effective use of the agency's accident data file. Items to be considered for retrieval from the accident and traffic records are total accidents (rate), wet weather accidents (rate), and the wet/dry ratio.

a. Identification of Wet Weather Accident Sites

(1) Accident records, which are developed in compliance with Highway Safety Program Standard No. 9, Identification and Surveillance of Accident Locations, should be searched at least annually to identify sites which have a high incidence of wet weather accidents. It is essential to have a standardized highway location reference system for correlating data from different sources. Accident rates at a site will be of greatest value if:

(a) the traffic volume is relatively high (i.e., approximately 1,500 vehicles per day or greater),

(b) the period of accident data is at least two years, and

(c) rainfall data are available for the same period as the accident data.

(2) Rainfall patterns for the years in which skid resistance and accident data were compiled should be acquired for each area in the highway agency's jurisdiction. A suggested method is presented in Appendix A.

(3) There are several methods in use by highway agencies to evaluate wet weather accident locations. One such method is the Wet Safety Factor (WSF), which is presented in Appendix A.

b. Field Review. A list of all sites ranked in order of WSF or another appropriate measure should be prepared as the basic list of candidate sites for remedial treatments. The selected locations should then be skid tested and reviewed by a team representing various disciplines such as highway materials, design, construction, maintenance, traffic and safety. See Appendix B for skid testing procedures. The review team should determine probable reasons for the high incidence of accidents and recommend corrective actions. Once the review team has recommended appropriate corrective treatments, a priority list of projects can be prepared based on benefits and expected costs.

c. Priority Program. An assessment should be made of the benefits relative to the cost of providing remedial treatments for high priority projects. A number of highway agencies have their own methods for conducting benefit cost analyses of alternative remedial treatments. Some of these remedial methods are tied into traffic engineering or pavement management programs. A specific program for evaluating the benefits and cost of alternative treatments is presented in reference 1, Appendix C.

d. Evaluation

(1) Evaluation of completed projects as required in Highway Safety Program Standard No. 9 and FHPM 8-2-3, Highway Safety Improvement Program, should be well documented and should include a representative sample of completed projects. A sampling plan should be established, using accepted statistical methods, to evaluate projects with a range of such variables as classes of roadways, traffic volumes, types of countermeasures, pavements used, and other pertinent factors. On hazard elimination

projects, these data should be correlated with accidents and traffic exposure and other pertinent factors in before/after analysis. See reference 2 in Appendix C.

(2) The evaluation of completed safety projects should be a continuing process to ascertain the long-term performance of corrective actions such as skid resistant overlays. The evaluations should address at least:

(a) the overall effectiveness of the program in reducing accident rates at the corrected sites,

(b) the adequacy of the various materials, designs, or methods used, and

(c) recommendations for changes in the program, practices, or needed research and development.

(3) As a secondary benefit, the evaluation process should provide input to an overall pavement management process.

6. <u>PAVEMENT SKID RESISTANCE TESTING PROGRAM</u>

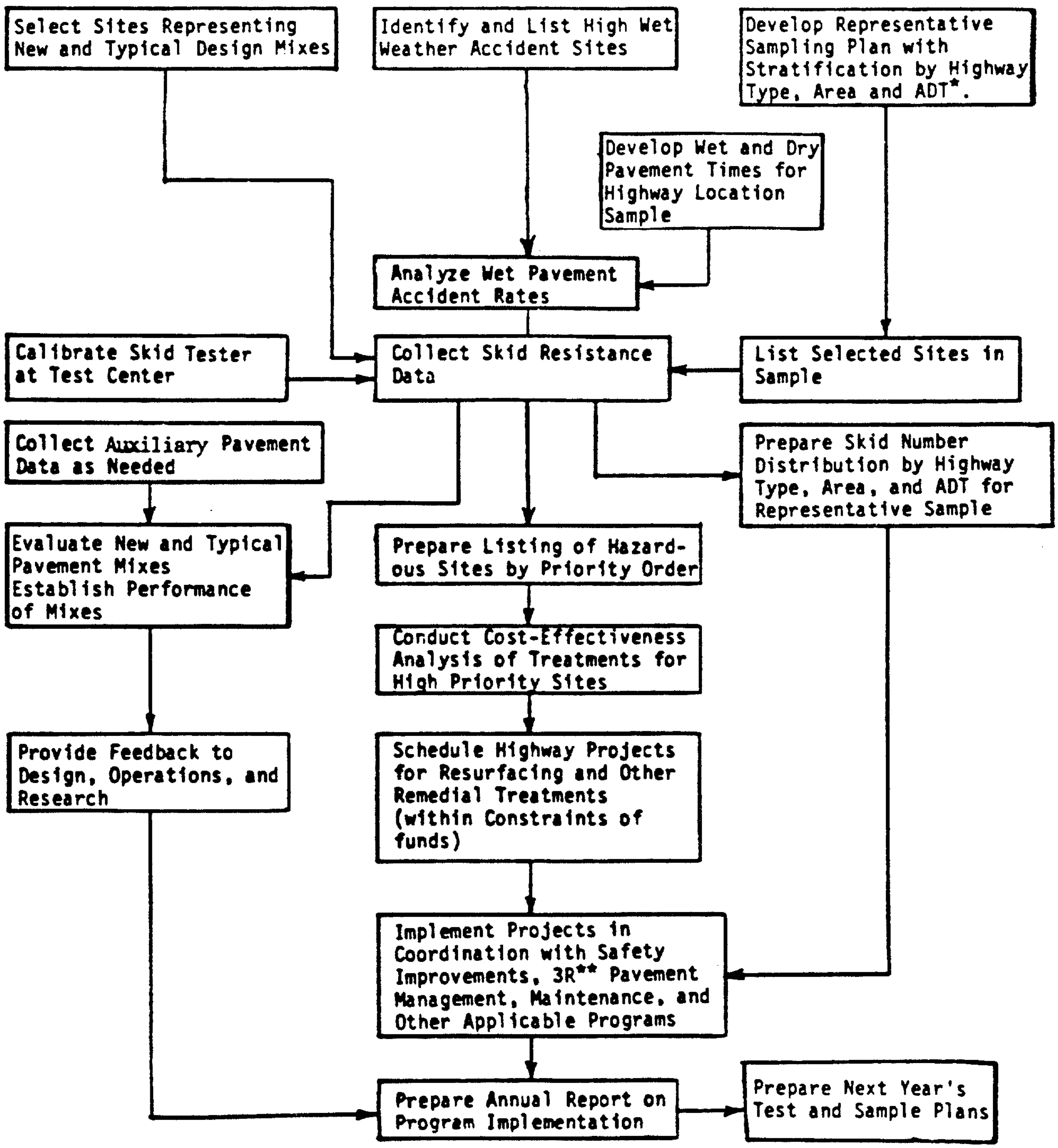
a. <u>General Description of Program.</u> The actual testing of pavement friction provides basic data for use in the three activities introduced in paragraph 3. Figure 1 graphically presents the interrelation between these activities. The upper portion of Figure 1 provides an overview of data to be collected to serve the safety, construction, and maintenance functions of highway organizations concerned with the skidding properties of pavement surfaces. The lower portion of Figure 1 indicates the various uses of the skid testing data, along with weather and accident data. Some of these data are evidence of the durability of particular surfaces, while other data provide a general overview of the skid resistance characteristics of the highway system.

(1) Skid resistance testing should be organized to support the following activities:

(a) Pavement evaluation studies in which measurements of the skid resistance of test sections are made to determine the skid characteristics of typical mix designs. Sufficient numbers of measurements should be

Figure 1

MODEL SKID ACCIDENT REDUCTION PLAN



*ADT: Average Daily Traffic

**3R: Resurfacing, Restoration and Rehabilitation made to determine the level of pavement friction, wear rates, and speed gradient of the pavement under various traffic exposures. These test sections should include the new projects to be tested as described in paragraph 4b(4).

(b) Evaluation of friction characteristics at locations which have a high incidence of wet weather accidents.

(c) System status for which measurements of the skid resistance of a representative sample of roads are made to develop the general levels of pavement friction on all roads in the highway agency's jurisdiction.

(2) Accurate location of sites or road sections requires the use of a standardized reference system. Often each element of the State which collects highway data uses its own reference system. For example, police accident reports may locate accidents by distance to a landmark, pavement records may be kept by project number and geometric features may be identified by station. A unified reference system has many benefits, especially in pulling together technical data for identifying and analyzing locations with a high incidence of wet weather accidents.

(3) Pavement evaluation study sites and wet weather accident sites should be identified by the element within the highway agency responsible for those programs. The skid testing can then become a routine matter for the element charged with operation of the skid test equipment.

(4) A total skid inventory of all roads and streets in a highway system has proven to be impractical and is not necessary to carry out an effective skid accident reduction program. Roads and streets which are used primarily by vehicles traveling at low speeds are not highly susceptible to skid accidents and accordingly can be eliminated from routine sampling of highway sites. For urban areas, this means that most city arterials would be sampled but residential streets and roadways with low speed limits would not. Nearly all rural highway sections could be sampled, since such roads are liable to high-speed use.

(5) Another practical consideration in determining which roads should be sampled is traffic volume. In urban areas, most roads with high speeds have moderate to high traffic volumes whereas this is not the case for rural highways. Relatively few rural roads are used by more than 1,000 vehicles per day. On a cost-effectiveness basis, such roads can seldom justify resurfacing on the basis of safety considerations alone; therefore there is little benefit in routine sampling of low-volume rural roads.

(6) Highway sections within the constraints of higher speeds and volumes need not be tested every year, since few roads vary substantially in skid resistance in any two or three-year period. Beyond this period, however, roads may lose significant skid resistance and may pose a serious danger to users. Using these criteria as part of a sampling plan will permit most if not all highway agencies to make maximum use of skid resistance data without increasing the amount of skid testing undertaken.

(7) Skid resistance measurements should be made with a calibrated locked-wheel skid tester using the ASTM E 274 method and supplemental procedures described in Appendix B or an acceptable alternative method. Locations such as intersections and sharp curves which are not easily measured with the locked-wheel skid tester at the standard speed of 40 miles per hour should be tested at a lower speed. Such tests should be supplemented with texture measurements to permit extrapolation of available skid resistance to operating speeds. Alternative methods of measuring pavement friction properties may be used provided they correlate well with the locked-wheel skid tester.

(8) In analyzing the skid numbers obtained, the time of year the measurements were taken has to be considered. Several States have published the results of their analyses and have developed methods for correcting skid number measurements taken during various periods and for different pavement surface types. See references 5 and 6 in Appendix C.

b. Specific Data From Sample Sites. In conjunction with skid resistance measurements, pavement wet time and accident records are desirable for each roadway section in the sample. The highway location system should be used for correlating data from different sources. An example of specific data which is desirable at each sample site is given in Appendix D.

c. Sites With Low Skid Resistance. When sites with low skid resistance are identified during the testing of system status, these sites should be analyzed for corrective action. This can be done through a pavement management program, a high hazard elimination program, or other efforts. If the high hazard elimination program is used, the analysis should be in accordance with FHPM 8-2-3.

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Appendixes

EVALUATION OF WET PAVEMENT TIME AND ACCIDENT DATA

A.1 The quantity of rainfall (inches) recorded by weather stations may be used to calculate the percentage of pavement wet time. Wet pavement time (WPT) may be estimated from total annual rainfall in inches (AR) as follows:*

$$WPT = 3.45 \ln(AR) - 5.07$$

Dry pavement time may be estimated by subtracting the amount of wet time and ice and snow periods from the total time in the period analyzed. Data from rainfall stations maintained by the National Oceanic and Atmospheric Administration's Weather Service may be used for wet and dry pavement time estimates for various areas within a State.

Isohyetal maps may be used to develop site wet pavement times. If ice and snow cover pavements for a significant portion of the time, a map for dry time should be prepared as well. Figure A-1 provides an example of a wet time map drawn from isohyetal charts.

A.2 Wet Safety Factor (WSF)

There are a number of ways to evaluate the relative safety of the subject location, one of which is the wet safety factor (WSF) approach.** For each wet weather accident location, a WSF may be developed. This factor is expressed as follows:

$$WSF = (DA)(PWT)/(WA)(PDT)$$

where: DA = number of dry weather accidents
WA = number of wet weather accidents
PDT = percent of dry pavement time
PWT = percent of wet pavement time

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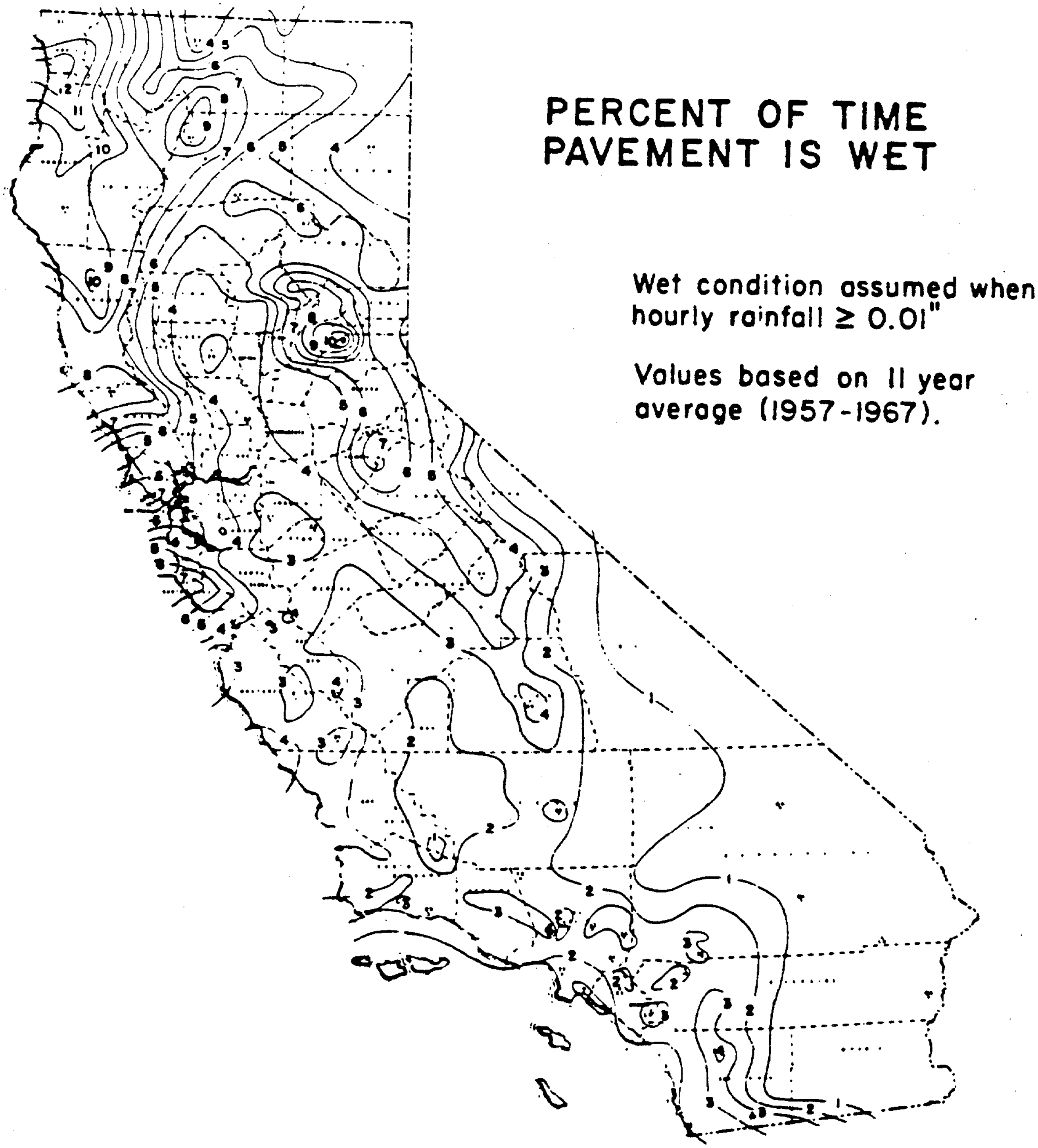
* This equation is based on a relationship developed by K.D. Hankins in "The Use of Rainfall Characteristics in Developing Methods for Reducing Wet Weather Accidents in Texas," Texas State Department of Highways and Public Transportation Study No. 135-4, July 1975.

** The WSF is a generalized form of an index referred to as the "skid trap ratio" and recommended for use in NCHRP Report 37, Tentative Skid-Resistance Requirements for Main Rural Highways," by H. W. Kummer and W. E. Meyer, Highway Research Board, Washington, D.C., 1967.

Figure A-1



[1]"A Method to Determine the Exposure of Vehicles to Wet Pavements," by James I. Karr, California Business and Transportation Agency, Department of Public Works, Division of Highways, January 1972.

This factor is the reciprocal of the risk of having a wet pavement accident relative to having a dry pavement accident. On a specific roadway section, each of these variables must be developed for the same time period; otherwise, traffic exposure must be taken into account. Criteria may be developed for further consideration of pavement sections. A WSF less than 0.67 suggests a wet weather problem. This criteria is based upon the conservative estimate of the overall likelihood of a wet weather accident being 1 1/2 as great as a dry pavement accident. This estimate assumes that wet weather accidents at the site or road section under consideration are attributable entirely to a skidding problem. A low WSF in most cases is due to poor skid resistance. However, traffic engineering evaluations may reveal deficiencies in sight distance, road markings, inadequate drainage, etc. Auxiliary information obtained during the test program should provide indications of the safety problems.

SKID MEASUREMENT SYSTEM DESCRIPTION AND OPERATING PROCEDURES

B.1 DESCRIPTIONS OF SKID MEASUREMENT SYSTEM

The requirements of American Society for Testing and Materials (ASTM) E 274 states "The method utilizes a measurement representing the steady state friction force on a locked test wheel as it is dragged over a wetted pavement surface under constant load and at constant speed while its major plane is parallel to its direction of motion and perpendicular to the pavement."

Although this specification may be met by a system involving only one wheel attached to a towing vehicle and although a few such systems are in use, the vast majority of skid measurement systems in use and expected to be in use in the near future consist of a towing vehicle and two-wheel trailer. On many systems either wheel may be locked during testing, but most commonly, the left is used.

The ASTM considers testing the left wheel track to be "normal." However, a differential in friction levels between the left and right wheel track may exist. When testing a site where a differential may exist, especially a high wet weather accident site, all lanes and wheel tracks should be tested. If a two-wheel trailer system is used, it is desirable to have the capability of testing with either wheel.

A skid measurement system must have a transducer associated with each test wheel which senses a force equal or directly related to the force developed between the sliding wheel and the pavement during test, electronic signal conditioning equipment to receive the transducer output signal and modify it as required, and suitable analog and/or digital readout equipment to record either the magnitude of the developed force or the calculated value of the resulting skid number (SN).

The system must include a facility for the transport of a supply of water--usually 200 to 500 gallons--and the necessary apparatus to deliver a specified amount of water--4.0 gallons per minute per wetted inch of pavement at 40 miles per hour within specified limits in front of the test wheel.

Finally, the system must include provision for measuring (and preferably for recording) the speed at which the test is conducted.

B.2 FIELD OPERATING PROCEDURES

B.2.1 Field Force Verification

It is generally impractical to perform force plate calibrations at frequent intervals while the measurement system is in the field. Facilities should, however, be available to permit the operator to ascertain that significant changes have not occurred in the force measurement subsystem since the most recent force plate calibration.

If the measurement system uses a torque transducer and is adaptable to mounting a torque arm, the verification can be accomplished within a reasonable time and effort. This device, consisting of an arm capable of being bolted to the test wheel in a horizontal position and of supporting known weights located at specified distances from the center of the test wheel, may be used to test the torque transducer to predetermined values of torque. Typically, the test wheel of the inventory system is raised off the ground, the torque arm is attached to the test wheel and held in a horizontal position, the brake of test wheel locked, and a series of known weights are suspended on the torque arm. This procedure will induce a series of known strains on the transducer, resulting in a series of output signals through the signal conditioning equipment. The magnitude of these signals should then be compared to the magnitude of signals produced through use of the same technique immediately after the most recent force plate calibration. Adjustment of signal conditioning equipment gain setting may be made to offset small force measurement subsystem variations which could occur.

Verification should be repeated periodically.

B.2.2 Test Tire and Wheel Preparation, Control of Tire Pressure

Tire Specification

Unless otherwise specified, all tests shall be performed with tires meeting the requirements of ASTM E 501, Standard Tire for Pavement Skid Resistance Tests, and all pertinent sections of that specification as well as ASTM E 274 should be observed in their use.

Tire Mounting and Break-in Procedure

The tire should be mounted on a Tire and Rim Association 6JJ rim. The rim should have been examined to determine that it has suffered no damage or misalignment in prior use. After mounting, and before break-in, the tire and wheel should be balanced. The tire should be subjected to a break-in of 200 miles use before being used for testing. This break-in may be accomplished by using the tire on the skid trailer wheel which is not used for testing. If the tire must be remounted before test use, it should be rebalanced after remounting.

Tire Warm-Up Procedure

The test tire should be inflated to 24 + 0.5 pounds per square inch measured at ambient temperature. After tire pressure measurement and adjustment, the tire should be subjected to a 5-mile warm up, travelling at conventional highway speeds, before tests are performed. The 5-mile warm-up should be repeated on any occasion when the measurement system is parked for a period of 15 minutes or more.

Tire Wear and Replacement Procedure

The standard pavement test tire has a series of visual wear guide sipes (small circular holes) cast into each of the outer ribs of the tire. The test tire should be withdrawn from testing use when wear has progressed to a point at which the wear guide sipes are no longer visible. During routine testing, test tires should be examined at least twice daily (and more frequently as tire nears unacceptable wear level) to determine that wear has not progressed beyond acceptable limits.

Additionally, after any series of tests on pavements having very high skid numbers (in excess of SN=70) or in the event of a deliberate or inadvertent dry skid, the test tire should be examined for the development of a flat spot. If a significant flat spot or spots develop on a test tire, it should be withdrawn from test use due to the tendency of the test wheel to seek out and return to such a flat spot in subsequent lockups.

B.2.3 Watering Subsystem Procedures

Daily Procedures

Prior to the beginning of each day's activity, the crew should perform at least the following functions with respect to the water subsystem:

1. Determine that the water nozzle (nozzles) when in the testing position assumes the proper angle with respect to the pavement (ASTM E 274 requires an angle of 25 + 5 degrees).

2. If the measurement system has provision for raising and lowering the nozzle between tests, determine that the mechanism is working properly and that the nozzle assumes a fully lowered position during the test sequence.

3. Determine that the nozzle, when in the test position, will discharge water directly in front of and centered on the test wheel.

4. Examine the nozzle outlet orifice to determine that it is free from damage or distortion.

The above inspections should be repeated during a day's testing in the event of operation on very rough highways (or in the event of any off-highway travel) which may have caused damage to the nozzle or adversely affected its orientation.

Water Trace Width Check

Periodically the crew should make a measurement of the water trace width as a gross measure of overall water subsystem performance. This may be accomplished by driving the measurement system over a pavement at a selected convenient speed (the same speed should be used on all occasions), initiating water flow without locking the test wheel brakes, and measuring the width of the resulting water trace on the pavement. The trace width measurement should be made as quickly as possible after passage of the inventory system (preferably within 30 seconds). This would require that one member of the crew drive and operate the measurement system while the other member is positioned off the side of the pavement at the location at which the measurement is to be made. Best results are achieved if this procedure is performed on a relatively smooth pavement surface (low macrotexture).

B.2.4 Instrumentation Calibration Verification

Provision should be made to allow for verification of the signal conditioning instrumentation calibration (to account for the effects of zero and gain drifts).

General Requirements for Calibration Signal

The minimum acceptable facility for verification of conditioning instrumentation is a calibration signal subsystem. The calibration signal should be provided from such a source and in such a manner that there is little likelihood of variation in the calibration signal itself. This assurance then permits the operator to make adjustments in the measurement subsystem gain to offset the frequent small deviations which occur due to changes in ambient temperature and other operating parameters.

Force Measurement Calibration Signal

The most straightforward technique for providing a force measurement calibration signal is to make provisions for switching a high quality shunting resistor of known value in parallel with one arm of the force transducer strain gauge bridge. This induces an imbalance in the bridge equivalent to the application of a known force to the transducer. The resultant signal is sufficient to verify, or provide means of adjustment for, all elements of the force measurement system forward of the transducer itself.

Frequency of Use

Instrumentation calibration verification through use of calibration signals should be accomplished at the beginning of each day's operation after equipment warm up, at intervals of no more than 2 hours when the system is in continuous use, and upon the renewal of operation throughout the day after any period during which the signal conditioning equipment has been turned off or the unit has been allowed to stand without use for 30 minutes or more.

B.2.5 Check List

A check list should be available to the crew and should be used prior to the beginning of daily operations and on any occasion during the day when testing is suspended for 30 minutes or more or when instrumentation has been turned off. The check list varies from system to system due to differences between the systems, but should provide for at least the following checks:

1. all power subsystems on and providing proper levels of power
2. all signal conditioning subsystems on for adequate time to reach stable operation (typically 10 to 30 minutes)
3. all recording systems on and functioning properly
4. instrument calibration (described above) performed
5. tire pressure checked and adjusted if necessary
6. test tire checked for wear
7. water nozzles checked for position and condition
8. water tank adequately filled
9. fuel supply adequate
10. safety chains and all other connections between trailer and towing vehicle properly connected, positioned, and protected if necessary
11. trailer jacks (if available) in retracted position
12. all auxiliary equipment (air-compressors, lights, etc.) functioning properly

B.3 USE OF STATIC AND DYNAMIC CALIBRATION PROCEDURES

B.3.1 Purpose of Field Test Center

At the present time the highest order of calibration and evaluation available for a State skid measurement system is that provided through the Field Test Center established under contract by the Federal Highway Administration (FHWA). Arrangements to receive the services of the Field Test Center may be initiated by a State through submittal of a request for such services to the local FHWA division office.

B.3.2 Criteria for When to Use the Field Test Center

Each measurement system should be submitted for calibration and evaluation at the Center as soon as possible after its introduction into service. It should be resubmitted for calibration and evaluation whenever:

1. significant repair or modification has been accomplished by the owning agency which might reasonably be expected to affect test results, or

2. whenever it has experienced sufficient use such that normal wear in the various subsystems might be expected to have affected their operation.

The second consideration suggests that each measurement system should be resubmitted at least every 2 years.

B.3.3 Calibration Services Provided by Field Test Center

The static and dynamic calibration services provided by the Field Test Center include the following:

1. Horizontal and Vertical Force Calibration. This provides for evaluation of the accuracy, linearity and hysteresis of the measurement system force transducers and signal conditioning equipment through use of an air bearing force plate maintained by the Center, and periodically calibrated by the National Bureau of Standards.

2. Flow Rate Evaluation and Adjustment if Required. This includes determination that the water delivery subsystem of the measurement system provides a quantity of water (dependent upon trace width) in front of the test tire which meets ASTM E 274 requirements at speeds between 20 and 60 miles per hour.

3. Static Evaluation of Water Distribution. This provides an evaluation of the uniformity with which the total water flow is distributed across the trace width and adjustment, if necessary, to assure that the water is in fact delivered uniformly and in line with the test tire.

4. Force Plate or Load Cells. The visitors force plate used for routine checks of the force measurement subsystem can be calibrated while at the Center.

5. Speed Calibration. The speed measurement (and recording if available) subsystem is evaluated, calibrated and, where necessary and possible, adjusted to produce accurate speed measurement values over the range of 20 to 60 miles per hour.

6. Tire Pressure Gauge Calibration. This provides assurance that tire pressures in the test wheels and in the speedmeasuring fifth wheel (if used) can be accurately measured and set.

7. Dynamic Correlation. Two such correlations are conducted: The first with the measurement system in the "as arrived" condition and the second after all of the foregoing evaluations have been conducted and indicated adjustments accomplished. The first correlation results in the development of mathematical relationships between the measurement system and the Area Reference Skid Measurement System that permit data collected by the measurement system, prior to its visit to the Center, to be adjusted to a common base provided by the use of the Area Reference System. The second correlation permits the development of similar relationships which may be used to relate the results of subsequent testing to the Area Reference System base. The data from the second correlation also provide an estimate of the system measurement variance.

B.4 MAINTAINING SYSTEM INTEGRITY BETWEEN FIELD TEST CENTER CALIBRATIONS

Two basic types of procedures are available for determining that significant changes have not occurred in the measurement system since its most recent evaluation and calibration at the Center. These involve techniques for evaluating important subsystem performance and techniques for evaluating performance of the total system.

B.4.1 Techniques to Evaluate Subsystem Performance

As a minimum, the owner of each measurement system should maintain and periodically make use of facilities for evaluating the force, water, and speed measurement subsystem of the inventory system.

Evaluation of Force Subsystem

The force subsystem should be evaluated through use of a force plate. An air-bearing force plate is recommended since its action is such as to essentially eliminate the effect of friction in the plate itself. If an air-bearing force plate is not available, any of several commercial mechanical force plates may be used. If a mechanical device is used, precautions should be taken to assure that all moving parts (particularly load application screws and spherical or roller bearings) are well lubricated and that the lubricant is periodically removed and replaced.

To conduct an evaluation, the test wheel of the measurement system should be centered on the force plate, the test wheel brake locked, and known frictional forces introduced to the tire-force plate interface through appropriate motion of the force plate. Frictional forces should be both increased and decreased in a stepwise manner to allow for detection of possible hysteresis effects. The indicated force readout values for the system should then be plotted against known force input values. The resulting plotted calibration line should be evaluated for nonlinearity and hysteresis characteristics. Also actual readout values for known force inputs should be compared with those readout values determined from tests conducted with the same equipment after the most recent Center evaluation.

Evaluation of Water Subsystem

The most effective evaluation of the water subsystem to discern variations in performance is that of flow. Flow rate may be evaluated by raising the rear wheels of the towing vehicle, running the vehicle at an indicated speed of 40 miles per hour (or any other desired speed), collecting the water pumped through the system and out the nozzle during a measured time period, and calculating the flow rate in gallons. This procedure should be repeated at two or more speeds to evaluate linearity of the water delivery subsystem with test speed.

The Pennsylvania State University has developed a water rate flow tank which is circular in cross section and of such size that it fits easily into a standard manhole. The tank has a threaded opening in the bottom for drainage and a stop-plug with a long handle which permits the plug to be removed and replaced from the top of the tank after it is hanging in the manhole. It also has a scale calibrated in gallons on the inside of the tank. This tank may be suspended in a standard manhole, the measurement system positioned so that the nozzle will discharge directly into the tank, the rear wheel of the towing vehicle raised, and total flow measured at any desired speed. The only additional equipment required is a stopwatch.

Evaluation of Speed Measurement Subsystem

The speed measurement subsystem should be evaluated by operating the measurement system at various test speeds over a measured mile course. If the basic speed measure is done through the use of the tow vehicle speedometer or through a tachometer-generator driven by the tow vehicle or by a fifth wheel, then the vehicle should be driven over the measured mile course at a selected speed and the time of transit measured with a stopwatch. The actual speed, calculated from the distance and the elapsed time, is then compared to the indicated speed.

If speed measurement is based upon a pulse generator driven by a fifth wheel, the accuracy of the speed measurement is directly dependent upon the accuracy of the fifth wheel for distance measurement. To evaluate this subsystem, the fifth wheel tire pressure is adjusted until the distance indicated agrees with the known distance traversed (the assumption being made here is that the electronic package which converts the pulses to velocity is functioning properly).

If tapeswitch event detectors, placed 200 feet apart, and an interval timer (+0.01 second resolution) are available to measure the time required by the inventory system to travel 200 feet, a very accurate speed measurement is obtained to check against the indicated value.

Time Between Subsystem Evaluations

The force, water and speed measurement subsystems of the measurement system should be checked by the methods described above at intervals no greater than 3 months.

B.4.2 Techniques to Evaluate Total System Performance

Use of Measurement System Sample Variance as Performance Measure

A portion of the information furnished, as a result of an evaluation at the Center, is the pooled sample standard deviation of the measurement system for repeated test at three test speeds on five special test surfaces. If the sample standard deviation at the desired speed is squared, the resulting value, SD_t^2 is an estimate of the skid measurement system variance. Subsequent to the Center evaluation, the crew should periodically select a pavement location having a skid number of approximately 30 to 40 and run 20 repeat tests at the desired speed over the same location. From the results of these latter tests, a new estimate, SD_E^2, can be calculated. If the ratio SD_E^2/SD_t^2 does not exceed 2.0, the chances are 19 in 20 that the system standard deviation has not doubled over that established during its visit to the Center. (If the system has not been to a Center to obtain an estimate of SD_t^2, its crew should select a pavement location having a skid number of approximately 30 to 40, run repeat tests at each desired speed over the same location, and calculate the sample standard deviation at each such speed.)

As an alternative, the above procedure could be performed making only 10 repeat tests on the selected pavement. In this case, the ratio of SD_E^2/SD_t^2 should not exceed 2.2 The chances are then four in five that the system standard deviation has not doubled over that previously established.

The above procedure should be performed at time intervals no greater than 3 months.

Short Term Checks of System Performance

The agency operating the measurement system should select several pavements located close to the site at which the system is normally garaged and perform repeated tests on the surfaces at quite frequent intervals, preferably weekly. Measured values of skid resistance on these surfaces will obviously change as the surfaces change from traffic wear, environmental, and/or seasonal variations. However, these changes should occur in an orderly and predictable fashion and any abrupt change would be an indication of possible erratic performance of the measurement system. A continually updated record of the results of such tests should be maintained and examined after each updating for evidence of such erratic performance.

REFERENCES

The following is a selected list of references which may be helpful in implementing the program described in this Technical Advisory. This list is not intended to be a bibliography of all documents available in this field:

*1. Effectiveness of Alternative Skid Reduction Measures, Benefit Cost Model, Report No. FHWA-RD-79-12, Volume II, November 1978, Federal Highway Administration.

*2. Accident Research Manual, FHWA-RD-80-016, February 1980, Federal Highway Administration.

*3. Evaluation of Minor Improvements (Part 8), Grooved Pavement (Supplemental Report) CA-DOT-TR-2152-11-75-01, September 1975, R. N. Smith and L. E. Elliott, Office of Traffic, California Department of Transportation.

4. Evaluation of Minor Improvements (Part 9), Open Graded Asphalt Concrete Overlays, January 1972, James I. Karr, Office of Traffic, California Department of Transportation.

5. Variations in Skid Resistance Over Time, FHWA-VA-80-33, February 1980, S. N. Runkle, David C. Mahone, Virginia Highways and Transportation Research Council.

6. Seasonal Variations in the Skid Resistance Pavements in Kentucky, Research Report 532, November 1979, James L. Burchett, Roland L. Rizenbergs, Kentucky Department of Transportation.

* These studies are available through the National Technical Information Service, 5285 Port Royal Road, Springfield, Virginia 22161

SPECIFIC DATA TO BE REPORTED FOR SAMPLE SITES

The following data should be collected in testing sample locations:

D.1 Skid numbers (SN) should be taken for major classes of roads stratified by traffic volume and geographical location.

D.2 Auxiliary data which should be included in order to establish distribution of skid numbers may include the following:

(a) Location of site or roadway section

(b) Responsible jurisdictional unit and route number or other designator

(c) Functional classification of road (e.g., two-lane, four-lane divided without full control of access, etc.)

(d) Surface type (e.g., bituminous, open-graded, concrete, tine finish, etc.)

(e) Average annual daily traffic (use traffic count data if available)

(f) Length of roadway section

(g) Lane where skid measurements are made

(h) Date of skid measurements

(i) Number of tests made in section

(j) Average SN

(k) Range of SN measurements

(l) Presence of atypical geometric or feature

(m) Evidence of skidding (e.g., skid marks, scarred posts, etc.)

U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION

SUBJECT	FHWA TECHNICAL ADVISORY
Texturing and Skid Resistance of Concrete Pavements and Bridge Decks	T 5140.10 September 18, 1979

Par. 1. Purpose
2. Cancellation
3. Background
4. Recommendations

1. PURPOSE. To issue guidance for providing an adequate level of wet pavement skid resistance on portland cement concrete (PCC) pavement surfaces and plain or latex modified concrete bridge deck surfaces.

2. CANCELLATION. This issuance supersedes FHWA Notice N5080.59 dated September 10, 1976, "Texturing of Concrete Pavements and Bridge Decks."

3. BACKGROUND

a. If a pavement or bridge deck surface is to provide adequate skid resistance for high speed traffic, two important, and to some extent independent, requirements must be satisfied. First, the pavement surface must provide for adequate adhesion between the tire and the pavement under wet weather conditions. Second, the pavement surface must provide sufficient surface texture and drainage potential to prevent the buildup of water pressure at the tire-pavement interface. On PCC riding surfaces, this drainage potential can be provided initially by the texturing of the concrete surface. The adhesion component of skid resistance is dependent on the wear and polish resistance characteristics of the aggregates.

b. A concrete finishing procedure that will provide an adequate and durable skid resistant surface texture is needed on all pavement and bridge deck surfaces.

c. While a skid resistant surface texture is needed on all pavement surfaces, the need may be especially critical on bridge decks because of the limited recovery area available to out-of-control vehicles.

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HNG-32

4. RECOMMENDATIONS

a. A burlap drag finish should not be used as the sole means of providing surface texture on projects with design speeds of 40 miles per hour (65 km per hour) or greater.

b. A broom or artificial grass drag finish, while often producing good initial surface texture, may not be durable unless a deep texture is produced by heavy bristles and high pressures. When a broom, artificial grass drag, or similar finish is specified for use on projects with design speeds of 40 miles per hour (65 km per hour) or greater, specifications that will insure a deep, durable texture must be used. Also, it should be conclusively demonstrated that safe, durable surfaces can be consistently produced by finishes before they are utilized extensively on high speed highways.

c. Metal tines, when preceded by a burlap or other type of drag finish, are recommended as being the most practical and dependable method of providing positive texture in PCC surfaces. The use of other procedures and equipment that will provide a similar grooved pavement surface is also encouraged. An Attachment to this advisory provides a summary of some of the more important research findings and recommendations relative to the texturing of PCC pavement surfaces.

d. The use of a deep surface texture on bridge decks may warrant increased concrete cover over the top layer of reinforcement steel beyond the 2.5 inches (63mm) required for minimum design cover and construction tolerance. The use of heavy transverse textures may also increase the concentration of deicing salts along the curb line unless proper provisions are made to drain these areas. The last 12 inches (300mm) of deck next to the curb should be left untextured to facilitate drainage.

e. Special techniques may be required in order to produce a durable grooved finish on latex modified and other low water-cement ratio bridge deck surfaces. The use of flexible tines on such surfaces frequently fails to produce a groove depth sufficient for reasonable durability. In at least one State, satisfactory grooves have been produced in these dense surfaces through the use of a roller consisting of sharpened flat washers welded at intervals onto the outside of a weighted section of pipe. Other techniques may also prove to be successful. As is the case with pavement surfaces, it is normally desirable that the plastic grooving application be preceded by a burlap or other drag finish or a light broom finish.

f. Regardless of the finishing or texturing method used, adequate durable skid resistance characteristics cannot be attained unless the fine aggregate has suitable wear and polish resistance characteristics. Research conducted by the Portland Cement Association indicates that the siliceous particle content of the fine aggregate should not be less than 25 percent. If past experience indicates that the coarse aggregates will be exposed by surface wear and have a significant effect on the skid resistance of the pavement, they, too, should have suitable polish resistance characteristics. Crushed material will normally provide higher skid resistance than uncrushed gravels.

RDMorgan

R. D. Morgan
Associate Administrator for
Engineering and Traffic Operations

Attachment

TEXTURING OF PCC PAVEMENT SURFACES

1. Transverse grooving will assist in providing a pavement surface with good durable pavement skid resistance characteristics at high speeds, will reduce splash and spray and headlight glare from wet roadway surfaces, and will continue to facilitate surface drainage until the depth of the wheelpath ruts exceeds the depth of the grooves. Longitudinal grooving assists vehicle control at curves and sites involving lateral movements. Both types of grooving effectively reduce the hydroplaning potential. The longitudinal grooving of existing pavements, while not necessarily producing an improvement in skid number, has been found to be an effective means of reducing accidents at sites having high, wet weather accident rates.

   Although longitudinal grooving may be preferable under some circumstances, and particularly when dealing with existing pavements, transverse grooving is considered to be superior to longitudinal grooving for general use on new construction because of the improved pavement drainage provided. Also, with the increased use of smaller, lighter cars and radial tires, complaints of vehicle handling problems on longitudinal grooved pavements seem to be on the increase.

   From the standpoint of compatibility between tire tread designs and pavement texture, it is desirable that, to the extent practical, longitudinal grooving patterns be standardized. The use of 0.095-inch (2.4mm) wide grooves spaced on 3/4-inch (19mm) centers is recommended. There is insufficient information available on which to base recommendations regarding the optimum texture for a milled PCC pavement riding surface. Some extremely harsh milled surfaces may be unacceptable for use as a riding surface.

2. The use of tines 0.03 inches (0.8mm) thick, 0.08 inches (2.0mm) wide and 4 to 6 inches (100 to 150mm) in length has resulted in good, durable surfaces when the grooves

were constructed to the maximum depth practical. This maximum practical depth, which will vary depending on the concrete mix design and other factors, is normally in the 1/8-inch (3mm) to 3/16-inch (5mm) range. Average transverse groove spacings of approximately 1/2 to 3/4 inch (13-20mm) are recommended. Groove spacings of less than 1/2 inch (13mm) may not have adequate durability. An increase in average groove spacing beyond 3/4 inch (20mm) cannot be expected to increase durability by a significant amount and may lead to noise problems. Relatively uniform groove spacings of 1 inch and 1 1/2 inches (25 and 38mm) have in some instances been considered unacceptable because of the concentration of tire-pavement noise into an objectionable frequency band. Experience with average groove spacings exceeding 1 1/2 inches (38mm) is too limited to facilitate the drawing of any conclusions. The use of wider, deeper grooves can be expected to increase the absolute noise level by a significant amount. Randomization of the groove spacing, which may result to some extent from the flexibility of the tines, is considered desirable from the noise standpoint.

3. The optimum length and angle of the tines will be dependent on the mix design, the weather, the finishing operations, and the stiffness of the tines and may be expected to vary somewhat from project to project.

   The timing of the plastic texturing operation is critical. If performed too early, the grooves may close back up. If performed too late, the groove depth will be reduced. The optimum time for performing the plastic grooving operation will be dependent on many variables which are subject to rapid fluctuations.

   When a tine finish is utilized, care should be exercised to avoid overlaps in the texturing operations. The overlaps will result in weak areas which will wear faster than the normal texture. For the best initial surface texture, a tine finish should be preceded by a burlap or artificial grass drag finish.

Because of the increased surface area, deeply textured surfaces will require a heavier than normal application of curing compound.

A tire tread depth gauge may be used to measure groove depths both in hardened and plastic concrete. When measuring the depth of fresh grooves formed by the tining of plastic concrete, it must be recognized that an artificially high reading may be obtained that will not be representative of the actual groove depths that will exist after a few months' exposure to traffic. To insure that the tined grooves will be durable and effective, it is essential that they be made as deep as practical during construction.

U.S. Department
of Transportation
**Federal Highway
Administration**

Technical Advisory

Subject

OPEN GRADED FRICTION COURSES

Classification Code	Date
T 5040.31	December 26. 1990

Par. 1. Purpose
2. Cancellation
3. Background
4. Recommendations

1. PURPOSE. To provide technical guidance on the use of open graded friction courses (OGFC), also known as plant mix seal courses, to develop good friction characteristics for pavement surfaces.

2. CANCELLATION. Technical Advisory T 5040.13, Open-Graded Asphalt Friction Courses, dated January 11, 1980, is canceled.

3. BACKGROUND

a. Open graded friction courses constructed with high quality, polish resistant aggregates have an outstanding capacity for providing and maintaining good frictional characteristics over the operating range of speeds on high speed highways. Their macrotexture facilitates drainage of water from the tire/pavement interface, improving tire contact with the pavement and reducing the potential for hydroplaning.

b. Open graded friction courses have generally provided good performance for 7 to 10 years under a range of traffic conditions. When failures have occurred, many were resolved by making minor refinements to the mix design and construction procedures to adjust for local conditions.

c. When compared to other high type surfaces, open graded friction courses have demonstrated the following advantages:

(1) provide and maintain good high speed, frictional qualities (the frictional characteristics are relatively constant over the normal range of operating speeds);

(2) reduce the potential for hydroplaning;

(3) reduce the amount of splash and spray;

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(4) are generally quieter, often providing a 3 to 5 decibel reduction in tire noise;

(5) improve the wet weather, night visibility of painted pavement markings; and

(6) conserve high quality, polish resistant aggregates, which may be scarce in some areas, because they are placed only as a surface layer, up to 3/4 inch thick.

d. Open graded friction courses exhibit the following limitations:

(1) increase the potential for stripping of the surface and underlying pavement (they do not seal the underlying pavement against moisture intrusion);

(2) require special snow and ice control methods and generally remain icy longer;

(3) require special patching and rehabilitation techniques;

(4) do not add structural value to the pavement (their performance is governed by the condition of underlying pavement); and

(5) may ravel and shove when used at intersections, locations with heavy turning movements, ramp terminals, curbed sections and other adverse geometric locations.

4. RECOMMENDATIONS. In selecting an OGFC, a number of factors should be considered, such as the environmental conditions, alignment, accident rates and the frictional properties of the State's standard surface mixes. Some locations or pavements may not be appropriate for an OGFC and therefore proper project selection must be considered. For an OGFC to perform as intended, it must be properly designed, constructed, and maintained.

a. An OGFC should only be placed on structurally sound pavements that have minimal cracks, ruts, bleeding and depressions. Pavement cracks are as likely to reflect through an OGFC as with any other thin asphalt course. The high air voids content in an OGFC will allow water to drain into it and attempt to flow laterally. Ruts in the underlying pavement may inhibit lateral flow and cause water to pond in the ruts, promoting separation of the OGFC from the underlying pavement. An OGFC placed on a bleeding pavement may lose its drainage characteristics (close up) due to the migration of the free asphalt from the underlying pavement.

b. The underlying pavement should be sealed with a 50 percent diluted asphalt emulsion, applied at a rate of 0.05 to 0.10 gallons per square yard. An OGFC will increase the amount of time that the underlying pavement will be wet. If the underlying pavement has a high air voids content, stripping potential is increased.

c. Specifications should require the coarse aggregate to be polish resistant and 100 percent crushed material. Carbonate aggregates should not be used. Certain slags and light weight aggregates have demonstrated satisfactory performance. The frictional qualities of an OGFC are affected by the microtexture of the coarse aggregate. It is poor practice to construct a premium friction course and then have its effectiveness lost due to polishing.

d. An OGFC should be designed in accordance with the mix design procedures included as the Attachment to this Technical Advisory. The basic steps in this procedure determine asphalt content, mixing temperature, air voids, and moisture damage susceptibility.

(1) An OGFC generally has a higher asphalt content than a dense graded mix and uses an equal or harder grade of asphalt. A very heavy asphalt film on the aggregate is essential for longevity. The film helps to resist stripping and oxidation of the asphalt cement. Typical dense graded mixes achieve a 4-6 micron average film thickness, where as an OGFC requires a 8-11 micron average film thickness. The OGFC has a black shiny appearance and appears to have excessive asphalt when compared to a dense graded mix. It is critical that no reduction in asphalt content be made based on the appearance of the OGFC. Excessive drain down of asphalt in the haul trucks can usually be corrected by lowering the mixing temperature or correcting deficiencies in the mixing and handling procedures. The combined handling and hauling of the mix should be limited to 40 miles or 1 hour.

(2) To ensure that a heavy asphalt cement film is actually obtained, the mixing temperature should correspond to the asphalt viscosity in the range of 700 to 900 centistokes from the temperature-viscosity curve for the asphalt cement. Higher mixing temperatures can cause the asphalt cement to flow off the aggregate. This may result in some areas of the mat having excessive asphalt, others not enough. A range of 2 to 5 percent minus 200 material in the mix will help achieve a thick asphalt cement film. A number of State and local agencies have successfully used latex modified asphalt and other additives to improve OGFC performance.

(3) The air voids analysis is not necessarily required for each project. However, it should be conducted when developing master gradation bands for open graded mixes or when considering new aggregate sources.

(4) An OGFC should be tested for moisture susceptibility because its high air voids content increases the potential for stripping. The mix should be tested for retained coating (AASHTO T 182) and retained strength (modified AASHTO T 165 and T 167). If stripping is observed, the mix design must be revised. The aggregates may be changed or an asphalt cement additive selected. Additional tests should be performed using the revised mix design.

e. One ounce of silicone should be added to every 5000 gallons of asphalt cement. This additive will improve mix workability and reduce the potential of tearing the mat under the paver screed. It also improves mix discharge from the truck beds.

f. An OGFC is placed as a thin lift and loses heat quickly. An OGFC should only be placed when the underlying pavement surface and ambient temperature have reached 60° F, otherwise raveling may result. Late season placement of an OGFC may prevent adequate curing of the asphalt cement and should be discouraged.

g. An OGFC should be placed full width, from outside edge to outside edge of the shoulders, to provide a cross-section with uniform frictional properties. As a minimum, it should extend 3 feet onto the shoulder. Do not place dense graded mix or curb and gutter adjacent to an OGFC. This will obstruct the lateral flow of water.

h. Handwork during placement should be minimized to avoid roughening of the surface. Rolling of an OGFC should be limited to one or two passes of an 8 to 10 ton static steel wheel roller to seat the mix. Longitudinal and transverse joints should be kept to a minimum. Joints should be butted rather than lapped.

i. Maintenance on roadways surfaced with an OGFC should avoid any activities which may obstruct the lateral flow of water through the OGFC.

(1) Traffic striping may inhibit lateral water flow if the stripe material is applied at a heavy rate or an excessive amount of reflective beads are used.

(2) Snow and ice control should be limited to plowing and chemical deicers. The use of sand or other abrasive to improve traction must be avoided.

(3) All crack and joint sealing should be performed prior to placing OGFC. When sealing is required on reflective cracks through an OGFC, only transverse joints should be sealed.

(4) Only small dense graded patches which allow for lateral flow of water through the OGFC should be considered. When larger areas of patching are involved, OGFC should be replaced with OGFC.

(5) A fog coat can be applied to an OGFC to extend the life of the asphalt binder. The fog coat is a 50 percent dilution of asphalt emulsion applied in two passes at a rate of 0.05 gallons per square yard for each pass. The use of rejuvenating agents should be avoided.

(6) When any additional overlay is required on the pavement, the existing OGFC surface must be removed.

Anthony R. Kane
Associate Administrator for Program Development

Attachment

OPEN GRADED FRICTION COURSE (OGFC) FHWA MIX DESIGN PROCEDURE

This document combines and updates the design procedure found in Federal Highway Administration Report No. FHWA-RD-74-2, Appendix A and B and Supplements 1 & 2 to the report which were distributed by FHWA Bulletin, dated July 11, 1975. The procedure has been expanded to consider alternative equipment. A suggested laboratory report form is included at the end of the design procedure.

1.0 Material Requirements

Definitions. The grading terminology used in this design procedure is defined as follows:
Coarse Aggregate Fraction - the aggregate from each source or combined job mix formula (JMF), which ever is specified, that is retained on a No.8 sieve.
Fine Aggregate Fraction - the aggregate from each source or combined JMF, which ever is specified, that passes a No.8 sieve.
Predominant Aggregate Fraction - the aggregate from the combined JMF that passes a 3/8" sieve and is retained on a No. 4 sieve.

1.1 Aggregate. Use high quality, polish resistant aggregate with a capacity to provide and maintain good frictional characteristics. It is recommended that relatively pure carbonate aggregates or any aggregates known to polish be excluded from the coarse aggregate fraction. The coarse aggregate fraction should have at least 75 percent by weight of particles with at least two fractured faces and 90 percent with one or more fractured faces. The abrasion loss (AASHTO T 96) should not exceed 40 percent.

1.2 Mineral Filler. Mineral filler as specified in AASHTO M 17 or as specified in the State's Standard Material Specifications is suitable for OGFC design.

1.3 Gradation. The recommended gradation for OGFC is as follows:

U.S. Sieve Size	Percent Passing (by weight)
1/2"	100
3/8"	95-100
#4	30-50
#8	5-15
#200	2-5

1.4 Asphalt Cement. The recommended grade of asphalt cement is AC-20, AASHTO M 226 Table 2. Other grades of asphalt should be considered when local conditions indicate a necessity or when an improved performance can be achieved.

1.5 Asphalt Additives. Additives may be required to improve the properties of the asphalt binder to resist stripping, retard oxidation (aging) or improve temperature susceptibility. Additives routinely used by the highway agency should be acceptable for OGFC mixes. Additives which have not been previously used should be considered experimental features and examined accordingly. In either situation, all additives required for the mix must be incorporated in the mix design.

2.0 Preliminary Data

2.1 Gradation. Test the aggregate from each source, as received for the project, for gradation. If mineral filler is submitted as a separate item, it should also be tested for specification compliance. Analyze the gradation results to determine the JMF that will meet the specification limits of Section 1.3.

2.2 Specific Gravity. Separate the coarse and fine aggregate for each aggregate source and determine the bulk and apparent specific gravity of the coarse and fine aggregate fractions for each source of material submitted. Utilizing the information verified in Section 2.1, mathematically compute the bulk specific gravity (SG_b) for the coarse and fine aggregate fractions for the proposed JMF gradation. If the bulk specific gravities of the aggregate sources are significantly different, a gradation analysis based on aggregate weight will not reflect the actual particle size distribution. Re-examine the gradation of the aggregate blend on a volume basis for compliance with Section 1.3.

Compute the apparent specific gravity (SG_a) of the predominant aggregate fraction based on the proportion of predominant aggregate from each source and utilizing the specific gravity information obtained above.

2.3 Viscosity. Test the asphalt cement to be used for specification compliance with AASHTO M 226. The asphalt cement binder used for the temperature-viscosity data should include all additives proposed for the mix.

3.0 Asphalt Content

3.1 Surface Capacity. Determine the surface capacity of the predominant aggregate fraction in accordance with the following procedure (AASHTO T 270):

3.1.1 Quarter out a 105 gram sample of the predominant aggregate. Dry the sample on a hot plate or in an oven (230 ± 9°F) to a constant weight and allow the sample to cool to room temperature.

3.1.2 Reduce the sample to approximately 100.0 grams (measured to 0.1 gram) and place the sample in a metal funnel with a piece of screen (No. 10 sieve) fastened above the orifice. The suggested funnel size is top diameter 3-1/2 inches, height 4-1/2 inches, orifice 1/2 inch.

3.1.3 Completely immerse the specimen in S. A. E. No. 10 lubricating oil for 5 minutes at room temperature. [IF HIGHLY ABSORPTIVE AGGREGATE IS BEING USED, IMMERSE THE SPECIMEN FOR 30 MINUTES.]

3.1.4 Drain the sample in the funnel for 2 minutes. Place the funnel containing the sample in an oven (140 ± 5°F) for 15 minutes of additional drainage.

3.1.5 Pour the sample from the funnel into a tared pan, cool to room temperature, and reweigh the sample to the nearest 0.1 gram.

3.1.6 Compute the percent oil retained (POR) using the following equation:

$$POR = \frac{SG_a}{2.65} \times \frac{(B-A)}{A} \times 100$$

where SG_a = apparent specific gravity of the predominant aggregate
A = oven dry weight of the sample (Step 3.1.2)
B = coated weight of the sample (Step 3.1.5)

3.1.7 WHEN USING THE PROCEDURE FOR HIGHLY ABSORPTIVE AGGREGATE, AFTER DETERMINING THE POR, POUR THE SAMPLE ONTO A CLEAN DRY ABSORPTIVE CLOTH AND OBTAIN A SATURATED SURFACE DRY CONDITION.

3.1.8 POUR THE SAMPLE FROM THE CLOTH INTO A TARED PAN AND REWEIGH THE SAMPLE TO THE NEAREST 0.1 GRAM.

3.1.9 COMPUTE THE PERCENT OIL ABSORBED (POA) USING THE FOLLOWING EQUATION:

$$POA = \frac{(C-A)}{A} \times 100$$

WHERE A = DRY WEIGHT OF THE SAMPLE (STEP 3.1.2)
C = SATURATED SURFACE DRY WEIGHT OF THE SAMPLE (STEP 3.1.8)

DETERMINE THE PERCENT (FREE) OIL RETAINED (POR_A) USING THE FOLLOWING EQUATION:

$$POR_A = POR - POA$$

3.1.10 Compute the surface constant value (K_c) for the predominant aggregate using the following equation or use Figure 1 below:

$$K_c = 0.1 + 0.4(POR)$$

WHEN USING THE PROCEDURE FOR HIGHLY ABSORPTIVE AGGREGATE, THE EQUATION FOR THE K_{cA} VALUE IS:

$$K_{cA} = 0.1 + 0.4(POR_A)$$

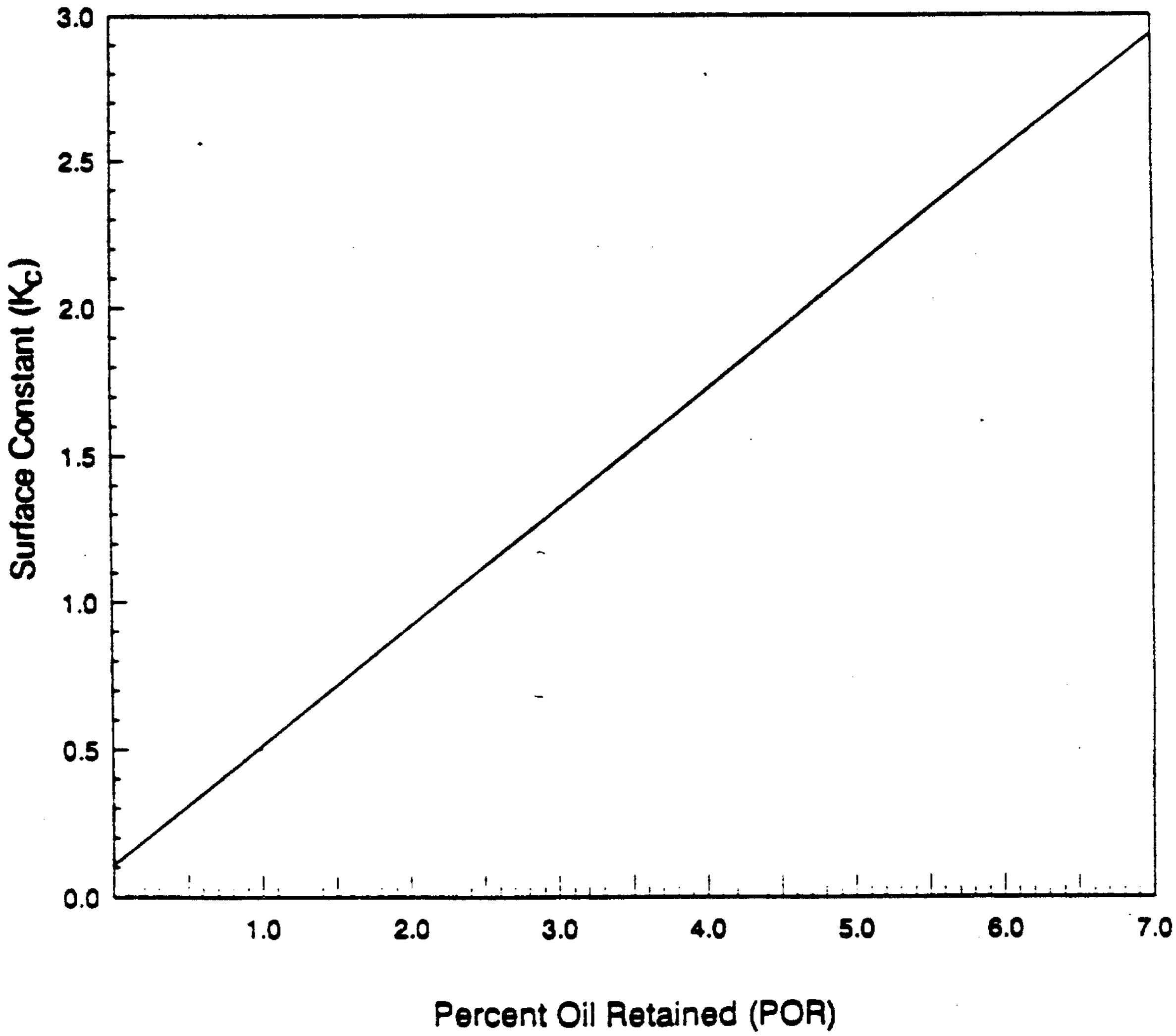

Figure 1 **SURFACE CAPACITY (K_c) GRAPH**

3.2 Asphalt Content. Compute the required JMF asphalt content (AC_{JMF}) which is based on the weight of aggregate from the following equation. The asphalt content computed from this formula is the same regardless of the asphalt grade or viscosity.

$$AC_{JMF} = (2(K_c) + 4.0) \times \frac{2.65}{SG_a}$$

WHEN USING THE PROCEDURE FOR HIGHLY ABSORPTIVE AGGREGATE, DETERMINE THE REQUIRED ASPHALT CONTENT (AC_{JMF}) AS FOLLOWS:

COMPUTE THE EFFECTIVE ASPHALT CONTENT (AC_{EFF}) FROM THE FOLLOWING EQUATION:

$$AC_{EFF} = (2(K_{ca}) + 4.0) \times \frac{2.65}{SG_a}$$

COMPLETE SECTION 4.0 AND 5.0, THEN CONTINUE WITH THE DETERMINATION OF THE ASPHALT CONTENT AS FOLLOWS:

PREPARE A TRIAL MIXTURE USING AN ASPHALT CONTENT EQUAL OR SOMEWHAT GREATER (ESTIMATE AMOUNT THAT WILL BE ABSORBED) THAN THE EFFECTIVE ASPHALT CONTENT (AC_{EFF}) DETERMINED ABOVE AND USING THE AGGREGATE GRADATION AS DETERMINED IN SECTION 5.2.

USING A SUITABLE TECHNIQUE, SUCH AS THE TEST FOR MAXIMUM SPECIFIC GRAVITY OF ASPHALT MIXTURES (AASHTO T 209), DETERMINE THE ACTUAL QUANTITY OF ASPHALT ABSORBED (IN PERCENT, BASED ON TOTAL WEIGHT OF AGGREGATE).

DETERMINE THE JMF ASPHALT CONTENT (AC_{JMF}) OF THE ABSORPTIVE MIXTURE USING THE FOLLOWING EQUATION:

$$AC_{JMF} = AC_{EFF} + \textit{actual asphalt absorbed}$$

4.0 Void Capacity of Coarse Aggregate

4.1 Unit Weight. Determine the unit weight of the coarse aggregate fraction of the proposed JMF by either of the following procedures (FHWA-RD-72-43 or ASTM D 4253 modified).

4.1.1 Apparatus

Compaction Mold. - A 6 inch nominal diameter solid-wall metal cylinder with a detachable metal base plate. A detachable metal guide-reference bar as shown in Figure 2 is required for Method 1.

Vibratory Compactor

Method 1 Rammer. - A portable electromagnetic vibrating rammer as shown in Figure 3, having a frequency of 3,600 cycles a minute, suitable for use with 115-volt alternating current. The rammer shall have a tamper foot and extension as shown in Figure 4.

Wooden Base. - A plywood disc 15 inches in diameter, 2 inches thick, with a cushion (rubber hose) attached to the bottom. The disc shall be constructed so it can be firmly attached to the base plate of the compaction mold.

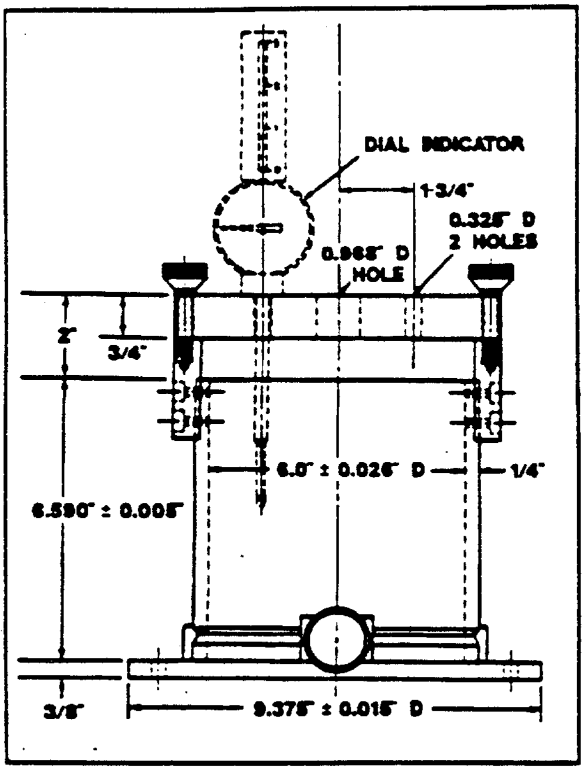

Figure 2 **COMPACTION MOLD**

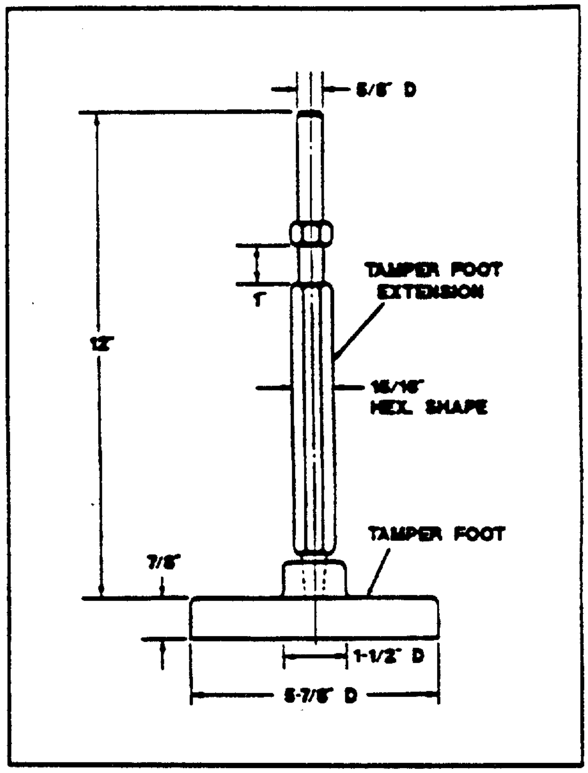

Figure 4 **TAMPER FOOT**

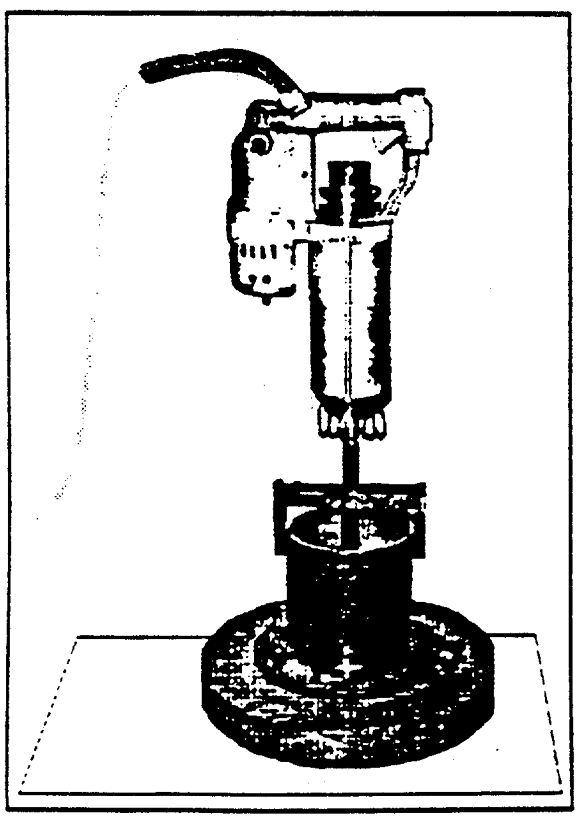

Figure 3 **VIBRATORY COMPACTION ASSEMBLY**

Method 2 (experimental) Vibrating Table. - A vibrating table capable of inducing a vibratory force to the sample at 3,600 cycles a minute and at an amplitude of (0.013 ±0.002 inch). (Soiltest CN-166 or equivalent)

Confining Load. - A circular steel disc weighing 60 pounds with a diameter of 5 7/8 inches. (Soiltest CN-167 or equivalent)

Timer. - A stopwatch or other timing device graduated in divisions of 1.0-second and accurate to 1.0-second, and capable of timing the unit for up to 2 minutes. An electric timing device or electrical circuits to start and stop the vibratory compactor may be used.

Dial Indicator - A dial indicator graduated in 0.001-inch with a travel range of 3.0 inches.

4.1.2 Sample. Select a sample of the coarse aggregate fraction (approx. 5 lb.) from the proposed JMF as verified in Section 2.1. If the bulk specific gravity of the coarse aggregate is less than 2.0, reduce the size of the sample to approximately 3.5-lb. Weigh the sample to the nearest 0.1 pound.

4.1.3 Procedure

Method 1. Pour the selected sample into the compaction mold and place the tamper foot on the sample. Place the guide-reference bar over the shaft of the tamper foot and secure the bar to the mold with the thumb screws.

Place the vibratory rammer on the shaft of the tamper foot and vibrate for 15 seconds. During the vibration period, the operator must exert just enough pressure on the hammer to maintain contact between the sample and the tamper foot.

Remove the vibratory rammer from the shaft of the tamper foot and brush any fines from the top of the tamper foot. Measure the thickness (t) of the compacted material to the nearest 0.01 inch.

Method 2. (experimental) Pour the selected sample into the compaction mold and place the surcharge base plate on the sample.

Lower the surcharge weight onto the surcharge base plate and vibrate the assembly for 2 minutes.

Remove the surcharge weight and brush any fines from the top of the surcharge base plate. Measure the thickness (t) of the compacted material to the nearest 0.01 inch.

4.1.4 Calculation. Calculate the vibrated unit weight (X) (in pounds per cubic feet) as follows:

$$X = \frac{6912\,w}{\pi\,d^2\,t}$$

Where w = weight of coarse aggregate fraction (pounds)
d = diameter of compaction mold (inches)
t = thickness of compacted specimen (inches)

4.2 Void Capacity. Determine the void capacity of the coarse aggregate (VCA) as percent of total volume using the following equation:

$$VCA = \left(1 - \frac{X}{U_c}\right) \times 100$$

Where X = vibrated unit weight from step 4.1.4
U_c = bulk dry solid unit weight of the coarse aggregate fraction (pcf).

5.0 <u>Optimum Content of Fine Aggregate</u>

5.1 Compute the optimum fine aggregate content with the following relationship:

$$Y = \frac{VCA - V - \dfrac{(AC_{JMF})\,(X)}{U_a}}{\dfrac{(VCA - V)}{100} + \dfrac{X}{U_f}}$$

Where Y = percent of fine aggregate by weight of total aggregate
VCA = voids in the coarse aggregate (percent)
V = design percent air voids = 15.0 percent
AC_{JMF} = asphalt content for the JMF (percent by weight of aggregate) [WHEN USING THE PROCEDURE FOR HIGHLY ABSORPTIVE AGGREGATE, USE AC_{crv} FROM SECTION 3.2, NOT AC_{JMF}]
X = vibrated unit weight of coarse aggregate (pcf)
U_a = unit weight of asphalt cement (pcf)
U_f = bulk dry solid unit weight of fine aggregate (pcf)

5.2 Compare the optimum fine aggregate content (Y) determined in Section 5.1 to the amount passing the No. 8 sieve of the proposed JMF. If these values differ by more than 1 percentage point, revise the JMF using the value determined for optimum fine aggregate content. Recompute the proportions of coarse and fine aggregates (as received) to meet the revised JMF. If the proposed and revised JMF gradations are significantly different, it may be necessary to rerun portions of this procedure.

7.0 Resistance to Effects of Water

Conduct the Immersion-Compression Test (AASHTO T 165 and T 167) on the designed mixture. Prepare samples at the optimum mixing temperature determined in Section 6.0. Use a molding pressure of 2000 psi rather than the specified value of 3000 psi. Determination of the Bulk Specific Gravity is not required for this design procedure.

After 4-day immersion at 120°F, the Index of Retained Strength shall not be less than 50 percent unless otherwise permitted. Additives to promote adhesion that will provide adequate retained strength may be used when necessary.

6.0 Optimum Mixing Temperature

Prepare a sample of aggregate (approximately 1000 grams) in the proportions determined under Section 5. Mix this sample with the proposed asphalt cement at the asphalt content (AC_{JMF}) determined under Section 3.2 at a mix temperature corresponding to an asphalt viscosity of 800 centistokes determined under Section 2.3. When the aggregate is completely coated, transfer the mixture to a pyrex glass plate (8-9 in. diameter) and spread the mixture with a minimum of manipulation. Place the plate with the sample in the oven at the mixing temperature. Observe the bottom of the plate after 60 minutes. A slight puddle of asphalt cement at the points of contact between the aggregate and the glass plate, as shown in Figure 5, is suitable and desirable after the 60 minute period. Otherwise, repeat the test at a higher or lower mixing temperature to achieve the desired contact area. If asphalt drainage occurs at a mixing temperature which is too low to provide for adequate drying of the aggregate (typically not lower than 225°F), an asphalt of a higher viscosity should be used.

An intermediate observation of the plate can be made at 15 minutes. If there is excessive drain down at the contact points, the sample can be discarded and the test repeated at a lower temperature.

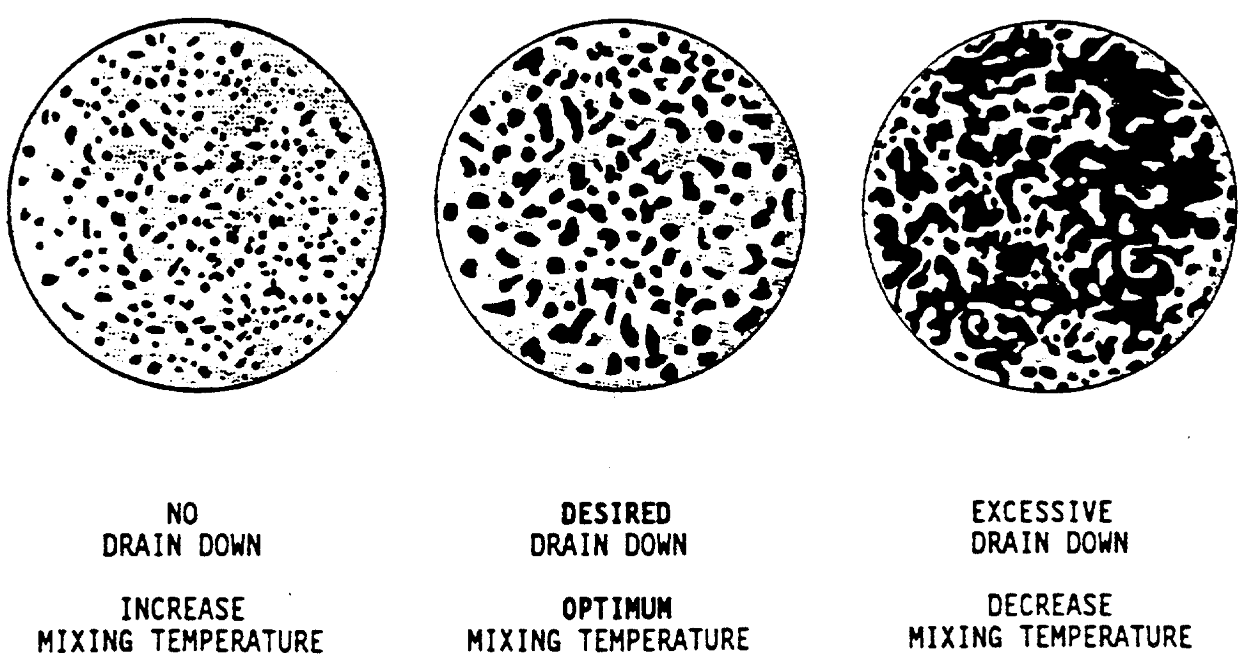

Figure 5 **DRAIN DOWN CHARACTERISTICS**

OGFC MIX DESIGN REPORT

1.0 MATERIAL PROPERTIES

A. Proposed Aggregate Proportions (by weight)

B. Proposed Job-Mix Gradation (percent passing)

Sieve Size	Specification Limit	Aggregate Sources			Job-Mix Formula
1/2"	100	____	____	____	____
3/8"	95 - 100	____	____	____	____
#4	30 - 50	____	____	____	____
#8	5 - 15	____	____	____	____
#16		____	____	____	____
#200	2 - 5	____	____	____	____

C. Specific Gravity - Unit Weight

	Aggregate Sources			JMF
COARSE AGGREGATE (Retained No. 8 Sieve) Bulk Sp. Gr. (SG_b)	____	____	____	____
Bulk Solid Unit Weight (U_c) where U_c=62.4(SG_b)				____ pcf
FINE AGGREGATE (Passing No. 8 Sieve) Bulk Sp. Gr. (SG_b)	____	____	____	____
Bulk Solid Unit Weight (U_f) where U_f=62.4(SG_b)				____ pcf
PREDOMINANT AGGREGATE (Passing 3/8" - Retained No.4) Apparent Sp. Gr. (SG_a)				____
ASPHALT BINDER Specific Gravity @ 77.0° F.				____
Unit Weight (U_a)				____

2.0 ASPHALT CONTENT

Percent Oil Retained POR = ____

Surface Capacity K_c = ____

Asphalt Content AC_{agg} = ____ % wt aggr

3.0 VOID CAPACITY

A. Void Capacity of Coarse Aggregate

Vibrated Unit Weight X = ____ pcf

Voids Coarse Aggregate VCA = ____ %

B. Optimum Fine Aggregate Content

Where: X = ____ pcf VCA = ____ %

U_f = ____ pcf V = 15 %

U_a = ____ pcf AC_{agg} = ____ %

Specs. Limit 5 < Y < 15 Y = ____ %

4.0 OPTIMUM MIXING TEMPERATURE

Asphalt Grade ____

Viscosity (cSt)	Temperature (°F)	Observed Drainage
600	____	____
700	____	____
800	____	____
900	____	____
1000	____	____

Target Mixing Temperature ____ °F

5.0 RESISTANCE TO EFFECTS OF WATER
(AASHTO T 165 & T 167, 2000 psi)

Air Dry Strength = ____ psi

Wet Strength = ____ psi (4 Days @ 120°F)

Retained Strength = ____ % (50% Minimum)

6.0 DESIGN SUMMARY

Aggregate Proportions (by Weight)

JMF Gradation (percent passing)

Sieve Size	JMF
1/2"	____
3/8"	____
No. 4	____
No. 8	____
No. 16	____
No. 200	____

Asphalt Grade ____

Asphalt Additives ____

Asphalt Content = ____ % wt aggr

= ____ % wt mix

Mixing Temperature Range ____ to ____ °F

REMARKS:

Mix Design Recommendation Accepted ___ Rejected ___

U.S. Department of Transportation
Federal Highway Administration

Memorandum

Subject: Automatic Profile Index Computation

Date: FEB 21 1991

From: Chief, Pavement Division
Washington, D.C. 20590-0001

Reply to Attn of: HNG-42

To: Regional Federal Highway Administrators
Federal Lands Highway Program Administrator

Recently, it has been brought to our attention that a significant difference exist between the results of an automated profilograph and a manual profilograph.

The attached South Dakota Department of Transportation report explains and analyzes the problems with automated profilographs. Some of the filtering algorithms used in the computerized profile index computations result in significantly underestimating of the profile heights for wavelengths shorter than 10 feet. Therefore, the overall profile index is also underestimated.

In view of the increased use of automated profilographs by the contractors and the States, it is important that the findings and recommendations of the attached report be shared with the divisions and States.

At the present time, we are developing a short training module to explain some of the problems in evaluating pavement profile. If you have any questions concerning this issue, please contact Mr. Aramis Lopez, Jr., at FTS 366-2084.

Louis M. Papet

Attachment

Analysis and Recommendations
Concerning
Profilograph Measurements
on
F0081(50)107
Kingsbury County

David L. Huft
South Dakota Department of Transportation
Office of Research
Pierre, South Dakota 57501-2586
(605)773-3292

November 26, 1990

I.Background

During the summer of 1990, Portland Cement Concrete pavement was placed on an 8.9 mile segment of US81 south of Arlington, South Dakota by Castle Rock Construction Company (Figure 1). The project number was F0081(50)107.

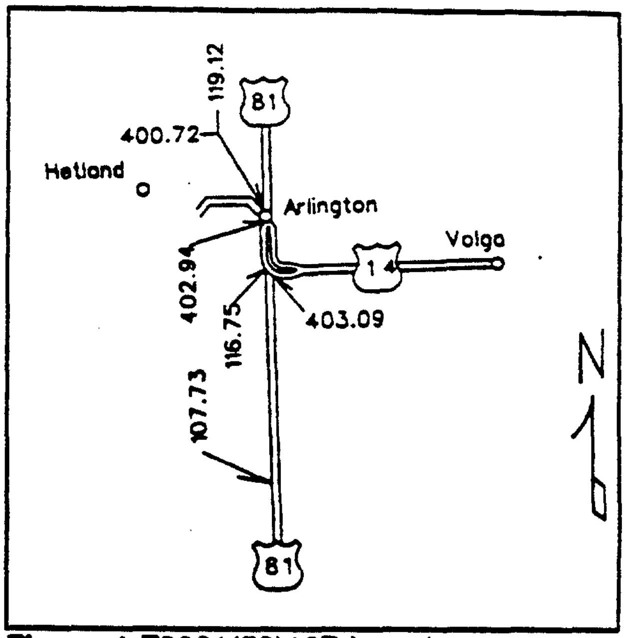

Figure 1 F0081(50)107 Location

In accordance with contract provisions, the contractor conducted profilograph tests to determine the ride quality of the finished pavement. The contractor's measurements indicated that generally high ride quality had been achieved, and that the contractor was entitled to an incentive bonus of nearly $89,000.

Profilograph tests performed by the South Dakota Department of Transportation's Office of Materials and Surfacing also showed good ride quality, but not as good as the contractor's tests had indicated. Profile indexes measured during SDDOT's quality control tests were typically one to two inches per mile higher than those measured by the contractor. Traces generated by the SDDOT unit consistentently showed higher profile amplitude than did traces from the contractor's (Figure 2).

SDDOT attempted a simple check to determine whether the contractor's profilograph measured the pavement profile accurately. When the profilograph was run over a short piece of one-by-two lumber, the unit indicated approximately one-half inch rather than three-quarters of an inch, the nominal wood thickness. This unexpected result seemed to suggest a problem in the contractor's profilograph.

The contractor and SDDOT also tested sections of pavement simultaneously to determine whether their profilographs would produce the same profile indexes on the same day. On August 28, both northbound and southbound lanes were tested at stations 21+71 to 48+11 and 438+83 to 470+51. Again, SDDOT's profile indexes and trace amplitudes were higher than Castle Rock's.

The contractor attempted to verify the operation of his profilograph by comparing its performance with another unit owned by the Iowa Department of Transportation. Castle Rock's profilograph measured profile indexes which agreed closely with Iowa's. When asked why the profilograph underestimated the thickness of the one-by-two, Iowa personnel speculated that the unit's filtering algorithm might be responsible. They advised that SDDOT might evaluate traces more carefully, to avoid misinterpreting spikes as roughness.

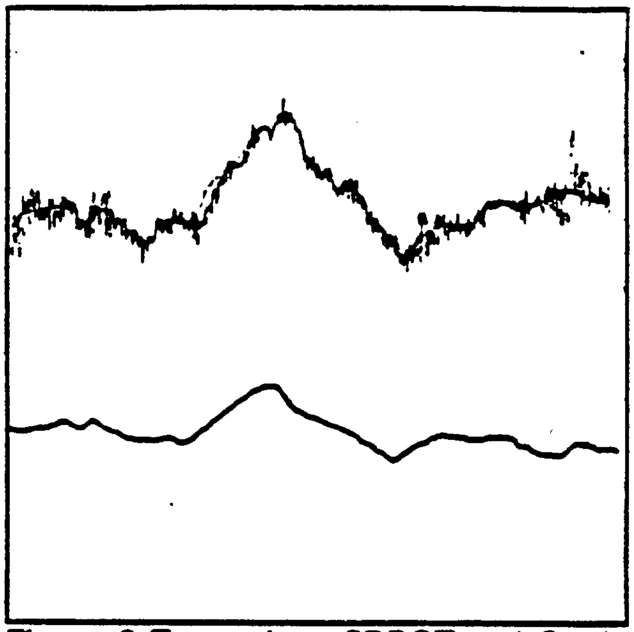

Figure 2 Traces from SDDOT and Castle Rock Profilographs

After the discrepancies were discovered, SDDOT retested the entire project. Profile indexes were consistently higher than those originally measured by the contractor. On the basis of SDDOT's measurements, the contractor was entitled to a bonus of less than $48,000. It seemed clear that the contractor's profilograph performed differently from SDDOT's profilograph, perhaps because of a filtering process performed by its on-board computer. But because SDDOT's tests were performed weeks after the contractor's, direct comparisons were not possible.

Because of the unresolved questions surrounding the profilograph measurements, the Aberdeen Region requested that the Office of Research provide technical assistance. Specifically, the objectives of this study were to:

1. Determine whether profilograph measurements obtained by the contractor's automated profilograph differ significantly from those obtained by SDDOT's manual profilograph;

2. If differences exist, determine their cause(s);

3. If differences are attributable to the filtering employed by the contractor's profilograph, develop a method to determine a fair ride quality bonus.

II. Significance of Profilograph Measurement Differences

It is essential to any analysis to first establish that the profile indexes measured by the contractor's and SDDOT's profilographs are statistically different. If observed differences only represent random variations, it would be pointless to conclude that either instrument was in error. If systematic differences exist, however, their causes might be determined. Two statistical tests were performed.

First, the project-wide profile indexes obtained by SDDOT on September 5-6 and 10-11, 1990 were compared to the profile indexes measured by the contractor within forty-eight hours of construction (Figure 3 and Figure 4). The hypothesis "Project-wide profile indexes measured by SDDOT are higher than the contractor's" was tested using the one-sided t-statistic with unknown standard deviations. That test demonstrated the hypothesis to be true with greater than 99% confidence.

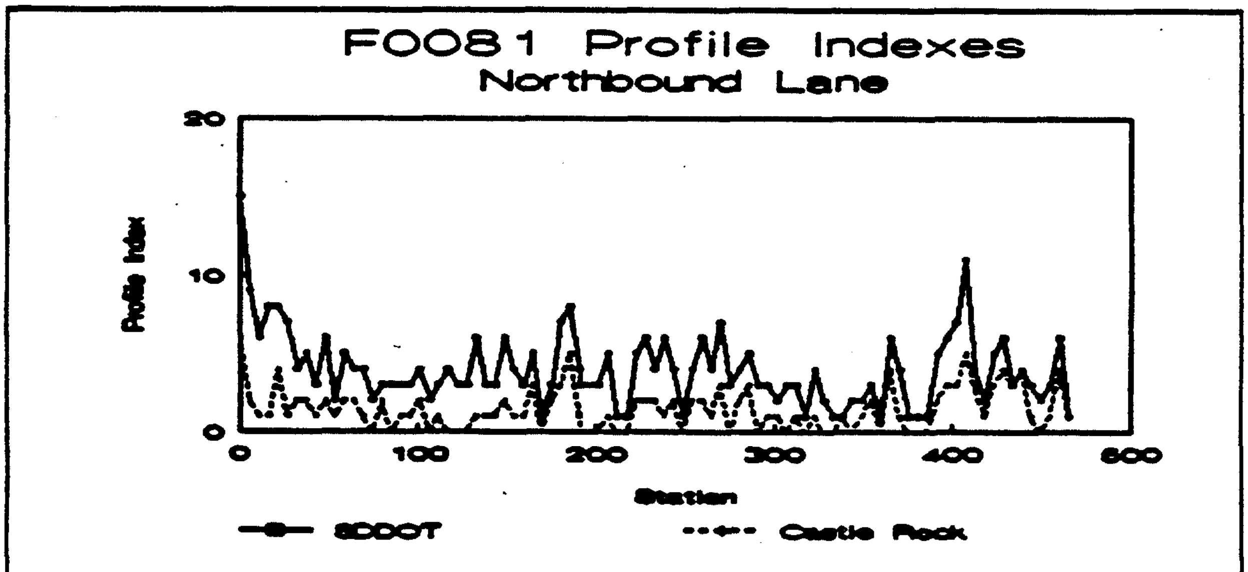

Figure 3 Project-Wide Profile Indexes Measured by SDDOT and Castle Rock Construction (Northbound Lane)

Second, the profile indexes obtained during head-to-head tests on August 28, 1990 were compared (Figure 5 and Figure 6). The hypothesis "SDDOT's profilograph generates higher profile indexes than does the contractor's" was tested, using the same statistical test. With more than 99% confidence, the hypothesis was determined to be true.

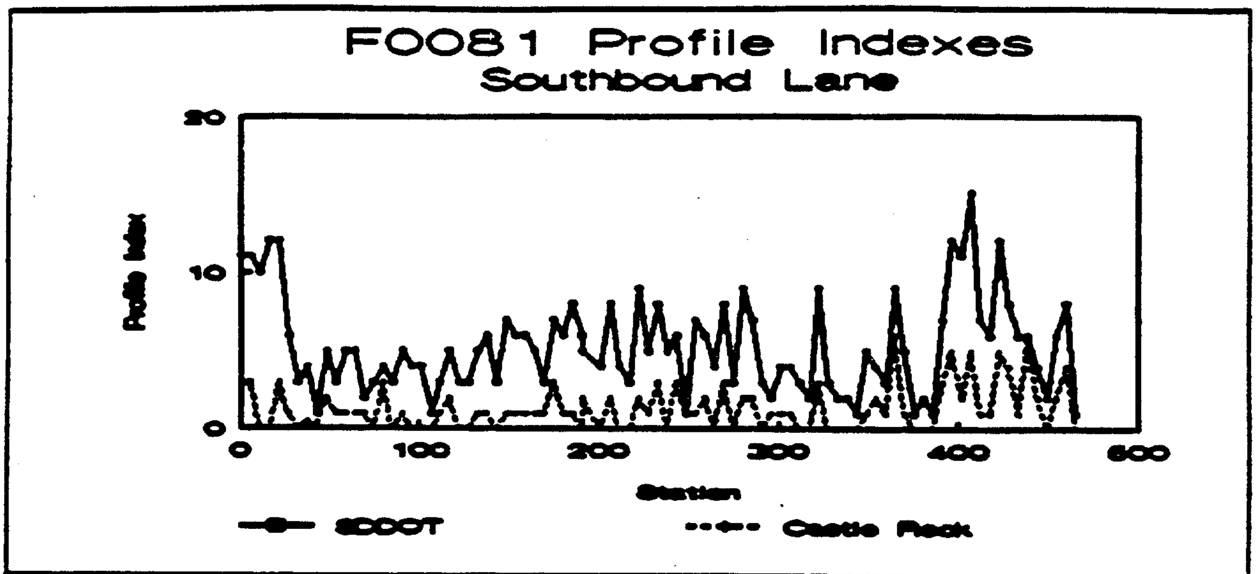

Figure 4 Project-Wide Profile Indexes Measured by SDDOT and Castle Rock Construction (Southbound Lane)

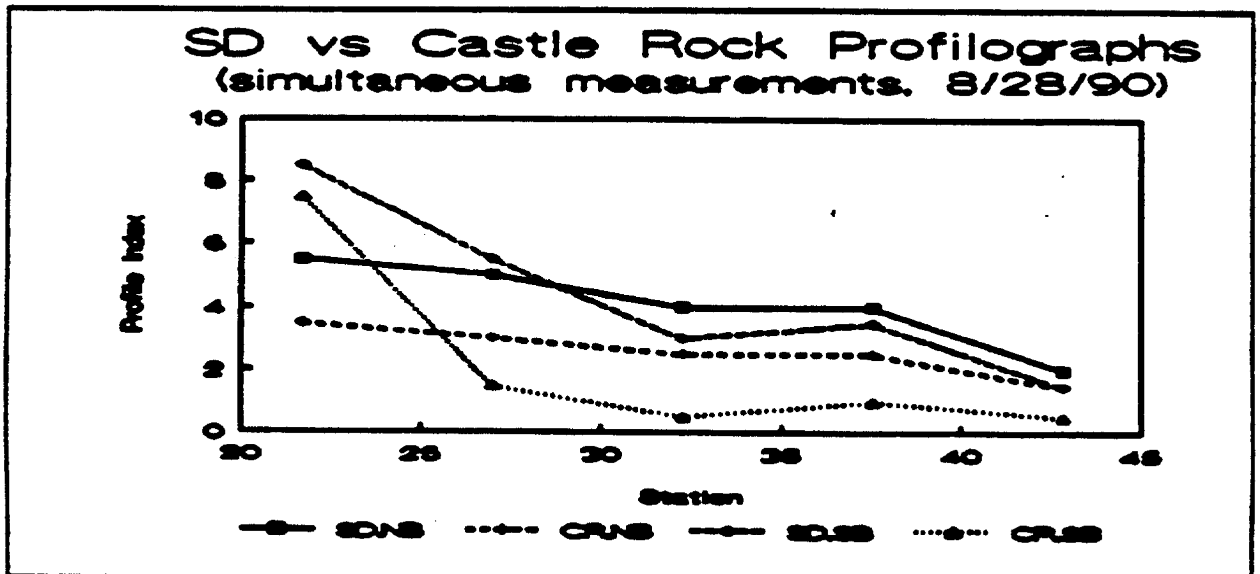

Figure 5 Profile Indexes Measured Simultaneously by SDDOT and Castle Rock Construction on August 28, 1990

On the basis of these statistical tests, it can be confidently stated that SDDOT's project wide measurements are higher than the contractor's and SDDOT's profilograph generates higher profile indexes than does the contractor's.

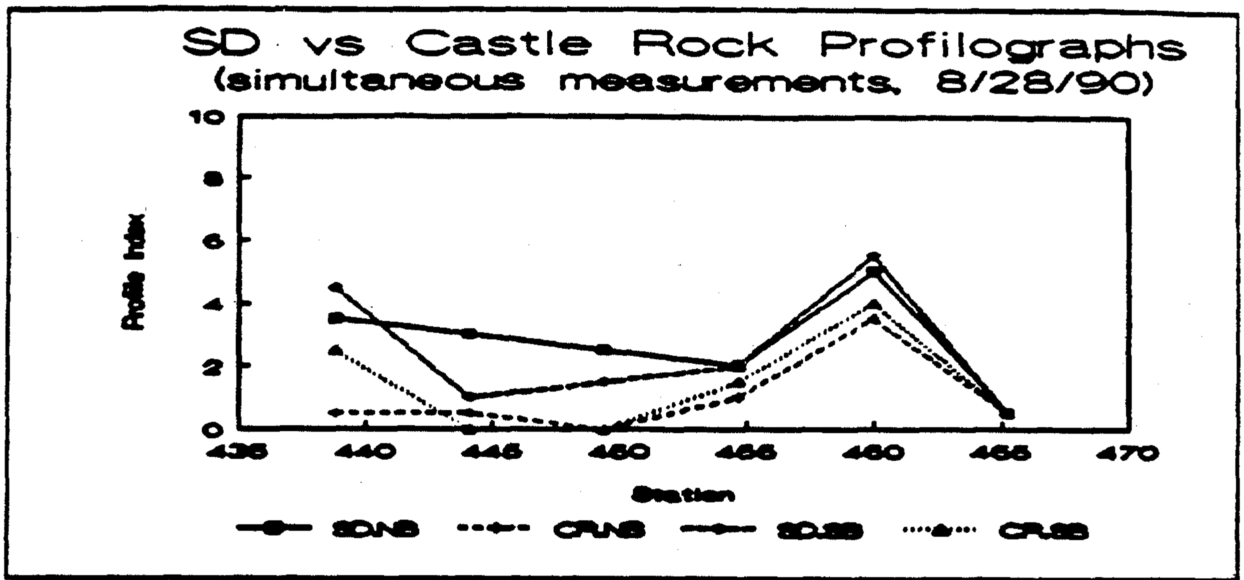

Figure 6 Profile Indexes Simultaneously Measured by SDDOT and Castle Rock Construction on August 28, 1990

IV. Profile Index Difference Causes

Two causes were considered likely to explain the differences between the profile indexes measured by SDDOT and those measured by the contractor. They were time between measurements (in the case of project-wide indexes) and differences between manual and automated profile interpretation.

A. Time Between Measurements

Although it seemed probable that differences between the profilographs were responsible for much of the discrepancy between SDDOT's and the contractor's profile index measurements, it also appeared that pavement roughness had changed since the time of paving. Indexes obtained by the contractor's profilograph on August 28 were higher than those taken immediately after construction, suggesting that the pavements had become slightly rougher following construction. This is reasonable, because the curing process and temperature changes can easily effect slab shape.

This observed change is important, because it means SDDOT measurements taken long after construction should not be used to determine ride quality bonuses. There can be no assurance that SDDOT's measured profile indexes accurately represent the ride quality immediately following construction.

B. Profilograph Differences

From the comparison of profile index measurements taken on the same pavement sections on August 28, 1990, it was clear that the contractor's profilograph measured lower profile indexes than did SDDOT's. Because the two machines are geometrically identical, the profile filtering process incorporated in the contractor's unit was considered the most likely cause of the difference.

(It should be noted that SDDOT's manual procedures were also evaluated, primarily because of Iowa's concern that spikes may have been incorrectly interpreted. However, no incorrect procedures were discovered. SDDOT's engineer had correctly smoothed the profile so spikes were ignored, just as Iowa had advised.)

Castle Rock's unit is a James Cox and Sons, Inc. Model CS8200 profilograph. It includes an on-board computer which digitizes the profile signal at 1.3" intervals and computes profile index automatically. To make profile interpretation less difficult, the computer uses a simple recursive digital filter to remove spikes (caused by extraneous mechanical vibrations) from the profile signal.

Mathematically, the filter is of the form

$$Y_n = AY_{n-1} + BX_n$$

where X_n is the raw (unfiltered) digitized elevation at point n, Y_n is the filtered elevation at point n, and Y_{n-1} is the filtered elevation at point n-1. A and B are constants which determine the filter's effect, and are defined:

$$B = N/65536$$

$$A = 1 - B$$

Cox recommends using a filter factor of N=8000 for most purposes.

The performance of this filter can be analysed with standard signal processing techniques. One useful analysis determines the response of the filter as a function of profile wavelength. Specifically, the analysis defines the filter's amplitude response $H(\lambda)$, which is the ratio of the filter's output to its input. It can be shown analytically that the amplitude response of this filter is given by the formula

$$H(\lambda)=\frac{B}{\sqrt{C^2+D^2}}$$

where

$$C=1-A\cos(\frac{2\pi\lambda_s}{\lambda})$$

and

$$D=A\sin(\frac{2\pi\lambda_s}{\lambda})$$

λ_s is the sampling interval of 1.3" used by the Cox profilograph.

As shown in Figure 7, the filter attenuates shortest wavelengths most. Wavelengths shorter than one foot are attenuated by more than 80%, so the effect of spikes will be reduced greatly. However, the filter also attenuates significantly longer wavelengths as well. Wavelengths of two feet are attenuated by over 60%; five foot wavelengths are attenuated by 30%. Even ten foot wavelengths are attenuated by 10%.

The filter can also be described in terms of its response to a step function. As Figure 8 shows, the filter is quite slow to recognize a step in the pavement profile. After the profilograph travels one foot past the step, it measures only 70% of the step's height. It is not until the profilograph has traveled three feet past the step that it measures 95% of the true height. This explains why the profilograph failed to measure the correct thickness of the one-by-two, which was less than one foot long.

The significance of Figure 7 and Figure 8 is that while the filter successfully removes spikes from the raw profile, it also removes longer features which are known to affect the pavement's ride quality. In other words, the filtering underestimates the amplitude of the pavement profile, causing estimates of profile index to be low.

Cox acknowledges that the profile index is affected by the filter. Their manual states, "It is important to understand that the test results are heavily affected by the selected filter factor." However, the filter's performance is fundamentally a consequence of its simple, first order formulation. Regardless of the filter factor used in the computation, the filter's selectivity will not be good. Invariably, long wavelengths will be removed along with the short. The performance of the filter could be improved by using a higher order filtering

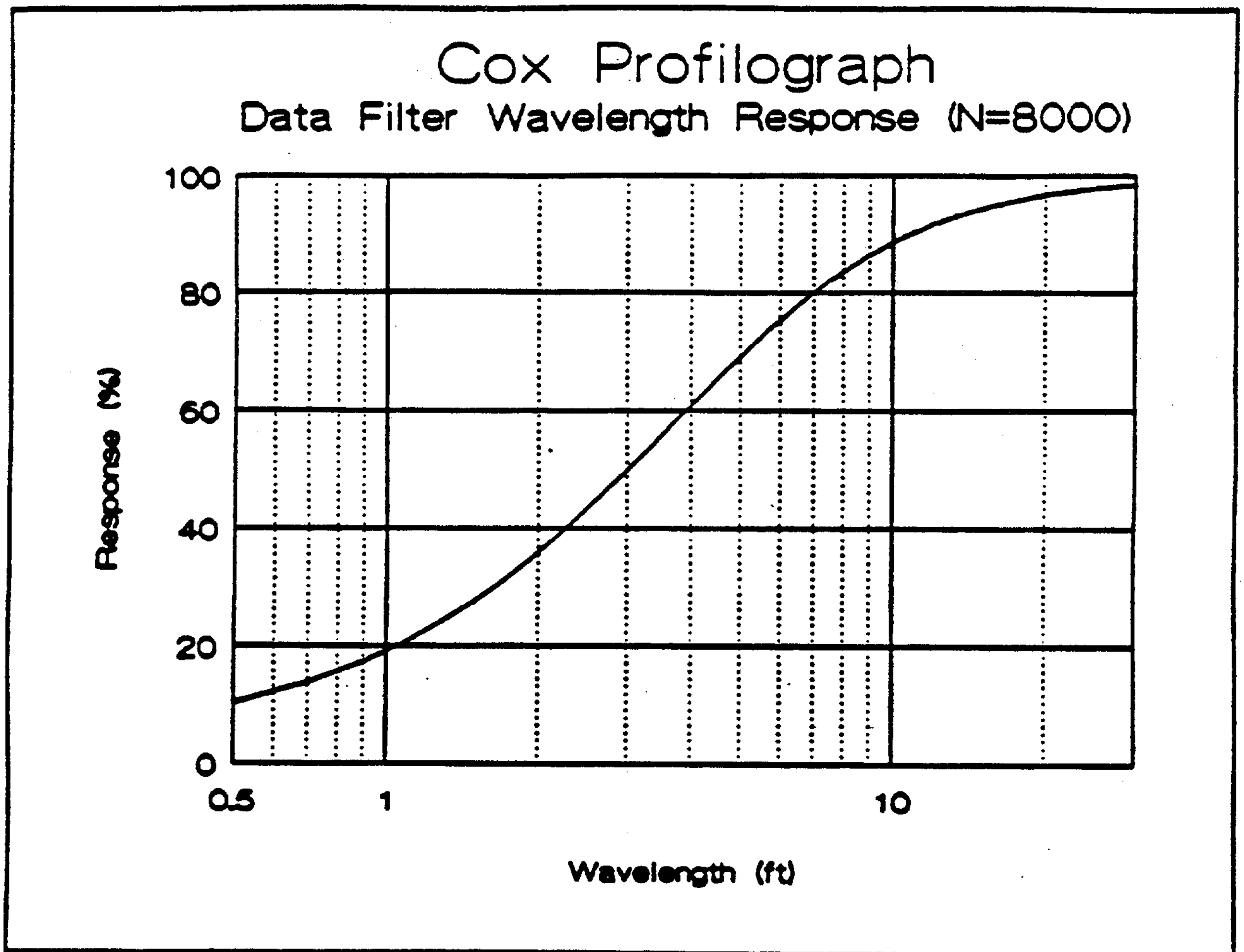

Figure 7 Response of Cox Data Filter as a Function of Wavelength

algorithm, if the profilograph's computer had sufficient power.

The filter's ultimate effect on computed profile index cannot be simply determined. Because the amount of attenuation depends upon the wavelengths present in the pavement profile, the effect on profile index will also vary. If a pavement contains predominantly short wavelengths, the profile index will be reduced greatly. If only longer wavelengths are present, the reduction will be slight.

This behavior explains why Castle Rock's automated profilograph correlated well with Iowa's manual profilograph. Iowa's test section consists of large amplitude bumps at predominantly long wavelengths (longer than twenty feet). At these wavelengths, the effect of filtering is too slight to be detected by visual inspection of profilograph traces.

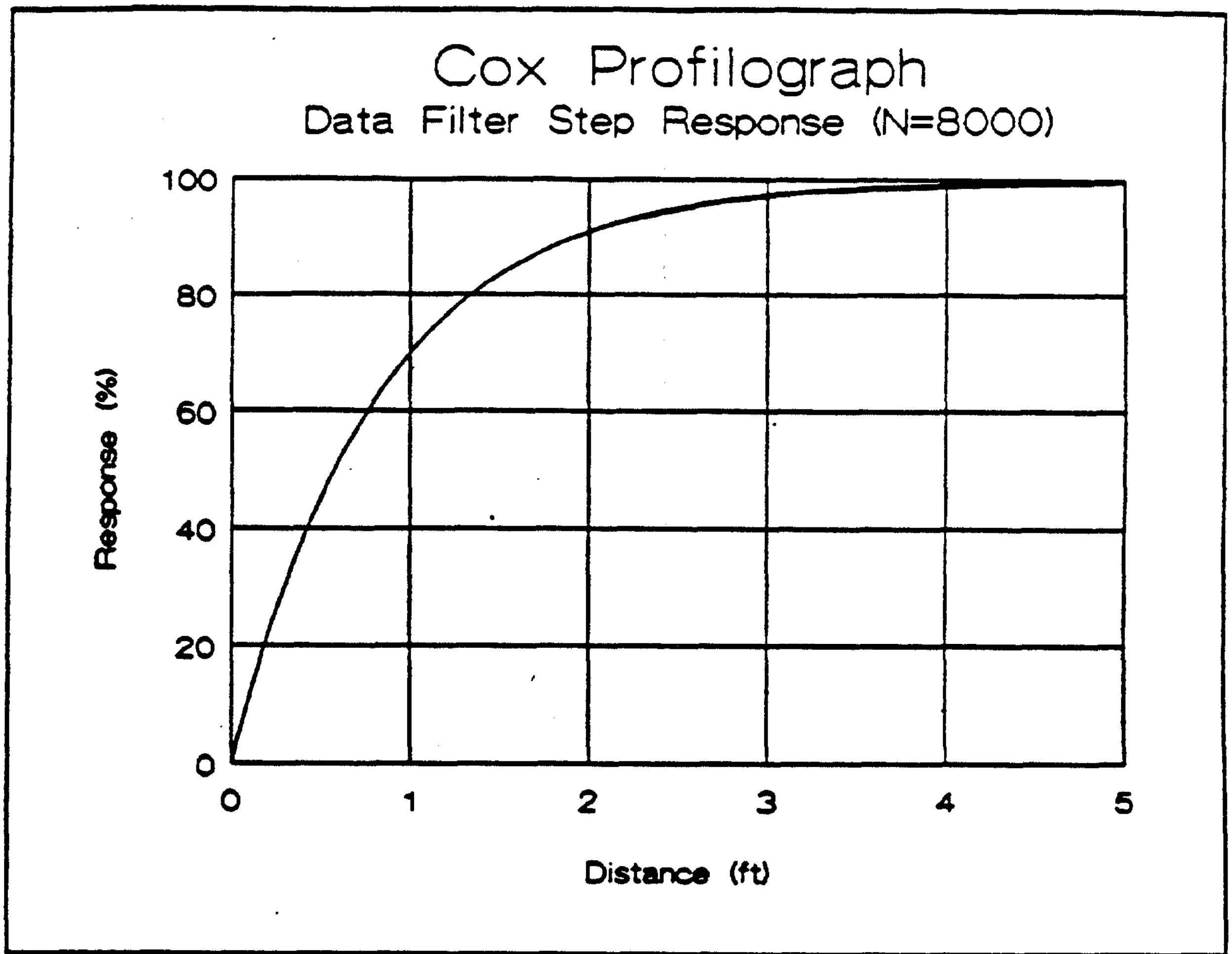

Figure 8 Cox Filter Step Response

III. Bonus Computation

In view of the discrepancies between profilograph measurements taken by SDDOT and those taken by Castle Rock, the question of fair bonus payments arises. Because the contractor's profilograph underestimated the height of profile features on the pavement, profile indexes were artificially low, and inconsistent with the measurement method assumed in the specifications. Consequently, the bonuses computed from the profile indexes were excessive. SDDOT's profilograph did not underestimate the profile, but because measurements were not taken within forty-eight hours of paving, they cannot be used directly as a basis for bonus payment.

In the interest of fairness to the contractor and the state, it would be best to correct the contractor's measurements somehow to remove the adverse effects of filtering. This could be done directly, but would require complete redigitizing of all profile traces taken from the contractor's profilograph. The procedure would use complex mathematics (Fourier transforms and inverse Fourier transforms) to reconstruct a profile of proper amplitude.

An alternative to this difficult and lengthy process is to correlate the two profilographs, using the measurements which were taken by both instruments on August 28. The resulting regression equation can then be used to adjust the contractor's profile indexes so they better represent unfiltered values.

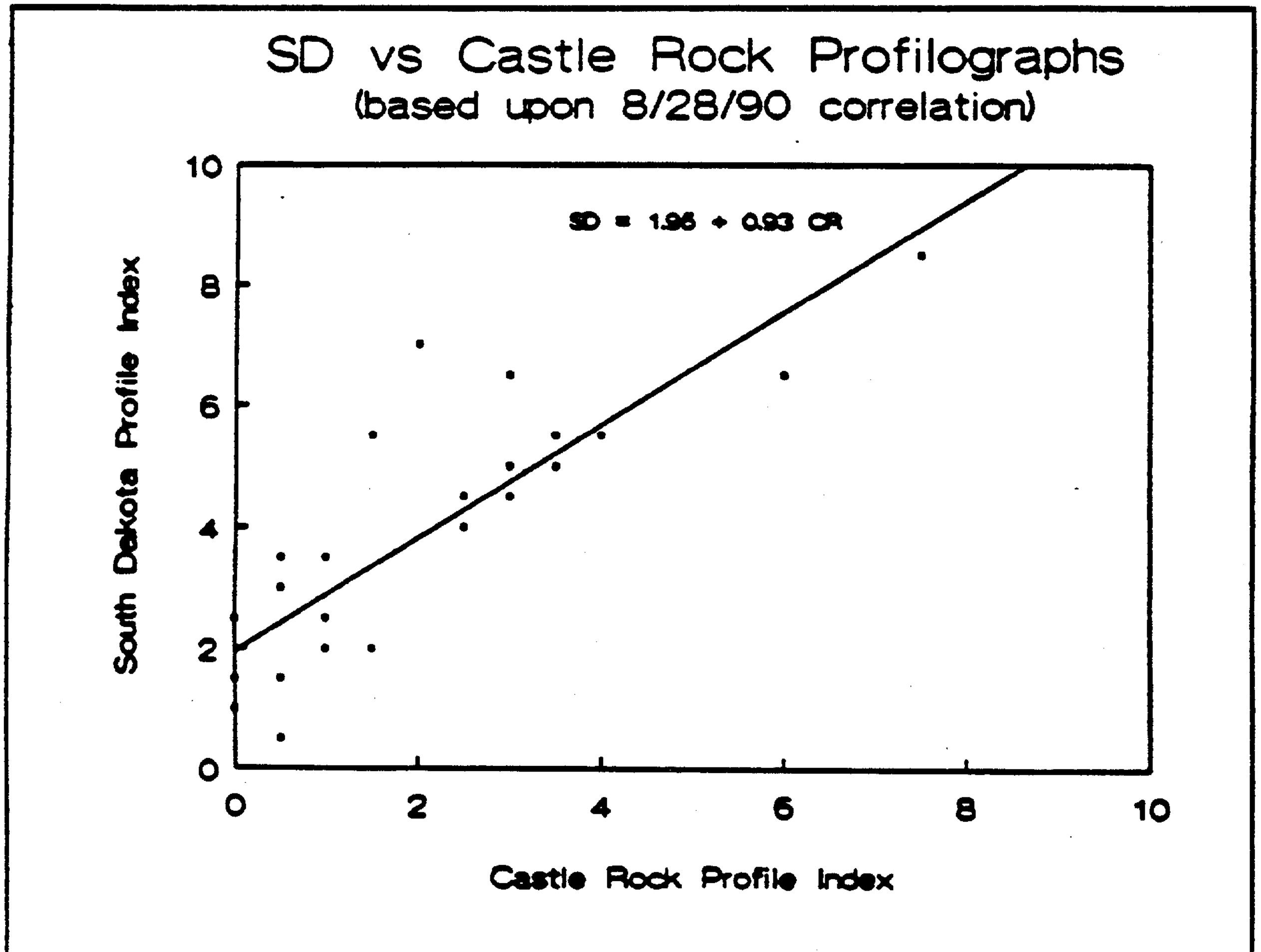

Figure 9 Correlation of SDDOT Profile Index vs Castle Rock Profile Index

Figure 9 shows the profile indexes measured by the two profilographs. From the data obtained in the simultaneous testing, the best equation relating filtered (Castle Rock) and unfiltered (SDDOT) profile indexes is:

$$PI_{unfiltered}=1.95+0.93PI_{filtered}$$

Application of this equation to the contractor's profile indexes produces adjusted values which more realistically represent the ride quality of the pavements. The bonus computed from these adjusted values total $68,975.36, which coincidentally falls approximately halfway between the contractor's original estimate and SDDOT's estimate derived from late measurements. A complete listing of SDDOT, contractor, and adjusted profile indexes, along with bonuses computed from them, is presented in Appendix A.

IV.Conclusions and Recommendations

Comparisons between profile indexes measured by the South Dakota Department of Transportation and Castle Rock Construction Company show that the profilographs used for the measurements are inconsistent with each other. Analysis of the filtering algrorithm used on the contractor's profilograph demonstrates that the Cox unit significantly underestimates profile heights at wavelengths shorter than ten feet. Therefore, profile indexes are also underestimated.

It was possible to derive a regression equation from profile indexes measured by the SDDOT unit and the contractor's unit on the same pavement sections on the same day. The contractor's profile indexes were adjusted using this equation, yielding new indexes which more realistically describe the ride quality achieved in the paving operation. The bonus computed from the adjusted profile indexes is approximately midway between the bonus computed from the contractors unadjusted indexes and the bonus computed by SDDOT from its late measurements.

It is clear that the filtering algorithm incorporated in the Cox unit should not be used in conjunction with SDDOT's special provision for paving incentives. Because the filtering falsely indicates a smoother pavement, its use (with the present special provision) is unfair to the state and to other paving contractors. To be fair, one or more of the following is recommended:

> Prohibit filtering. This might make automatic profile index computation impossible, if the computer requires smoothed profiles. The benefits of objectivity and quick test results would be lost.
>
> Lower the acceptable inches per mile if filtering is used. Additional correlation between filtered and unfiltered profile indexes, especially at higher roughness levels, may be desirable if this method is adopted.

Improve the filtering algorithm by using a higher order filter. This option would place additional computational demands upon the profilograph's on-board computer, but would provide much better selectivity between short and intermediate wavelengths.

A final recommendation concerns the criterion by which SDDOT and other transportation agencies determine whether a contractor's profilograph operates acceptably. It is common practice to accept a contractor's profile indexes if they fall within two inches per mile of the state's indexes. This criterion fails if a profilograph generates indexes which are within two inches per mile, but consistently low (or high). A statistical test to determine whether mean profile indexes are different would provide a much better indication of measurement validity, and would ensure fairness to both the contractor and the state.

In view of the increased use of automated profilographs, and ongoing development of new units by other vendors, it is important that the issues raised in this report be addressed prior to the 1991 construction season.

NORTHBOUND						
Station	State		Contractor		Adjusted	
	PI	Bonus	PI	Bonus	PI	Bonus
0+59 - 5+87	15	($533.87)	6	$106.77	7	$0.00
5+87 - 11+15	9	$0.00	2	$533.87	4	$320.32
11+15 - 16+43	6	$106.77	1	$533.87	3	$427.09
16+43 - 21+71	8	$0.00	1	$533.87	3	$427.09
21+71 - 26+99	8	$0.00	4	$320.32	6	$106.77
26+99 - 32+27	7	$0.00	1	$533.87	3	$427.09
32+27 - 37+55	4	$320.32	2	$533.87	4	$320.32
37+55 - 42+83	5	$213.55	2	$533.87	4	$320.32
42+83 - 48+11	3	$427.09	1	$533.87	3	$427.09
48+11 - 53+39	6	$106.77	2	$533.87	4	$320.32
53+39 - 58+67	2	$533.87	1	$533.87	3	$427.09
58+67 - 63+95	5	$213.55	2	$533.87	4	$320.32
63+95 - 69+23	4	$320.32	2	$533.87	4	$320.32
69+23 - 74+51	4	$320.32	1	$533.87	3	$427.09
74+51 - 79+79	2	$533.87	0	$533.87	2	$533.87
79+79 - 85+07	3	$427.09	2	$533.87	4	$320.32
85+07 - 90+35	3	$427.09	0	$533.87	2	$533.87
90+35 - 95+63	3	$427.09	1	$533.87	3	$427.09
95+63 - 100+91	3	$427.09	1	$533.87	3	$427.09
100+91 - 106+19	4	$320.32	2	$533.87	4	$320.32
106+91 - 111+47	2	$533.87	0	$533.87	2	$533.87
111+47 - 116+75	3	$427.09	1	$533.87	3	$427.09
116+75 - 122+03	4	$320.32	0	$533.87	2	$533.87
122+03 - 127+31	3	$427.09	0	$533.87	2	$533.87
127+31 - 132+59	3	$427.09	0	$533.87	2	$533.87
132+59 - 137+87	6	$106.77	1	$533.87	3	$427.09
137+87 - 143+15	3	$427.09	1	$533.87	3	$427.09
143+15 - 148+43	3	$427.09	1	$533.87	3	$427.09
148+43 - 153+71	6	$106.77	2	$533.87	4	$320.32
153+71 - 158+99	4	$320.32	1	$533.87	3	$427.09
158+99 - 164+27	3	$427.09	1	$533.87	3	$427.09
164+27 - 169+55	5	$213.55	3	$427.09	5	$213.55
169+55 - 174+83	1	$533.87	0	$533.87	2	$533.87
174+83 - 180+11	3	$427.09	2	$533.87	4	$320.32
180+11 - 185+39	7	$0.00	3	$427.09	5	$213.55
185+39 - 190+67	8	$0.00	5	$213.55	7	$0.00

190+67 - 195+95	4	$320.32	1	$533.87	3	$427.09
195+95 - 201+23	3	$427.09	0	$533.87	2	$533.87
201+23 - 206+51	3	$427.09	0	$533.87	2	$533.87
206+51 - 211+79	5	$213.55	1	$533.87	3	$427.09
211+79 - 217+07	1	$533.87	0	$533.87	2	$533.87
217+07 - 222+35	1	$533.87	0	$533.87	2	$533.87
222+35 - 227+63	5	$213.55	2	$533.87	4	$320.32
227+63 - 232+91	6	$106.77	2	$533.87	4	$320.32
232+91 - 238+19	4	$320.32	2	$533.87	4	$320.32
238+19 - 243+47	6	$106.77	1	$533.87	3	$427.09
243+47 - 248+75	4	$320.32	2	$533.87	4	$320.32
248+75 - 254+03	1	$533.87	0	$533.87	2	$533.87
254+03 - 259+31	4	$320.32	2	$533.87	4	$320.32
259+31 - 264+59	6	$106.77	2	$533.87	4	$320.32
264+59 - 269+87	4	$320.32	1	$533.87	3	$427.09
269+87 - 275+15	7	$0.00	3	$427.09	5	$213.55
275+15 - 280+43	3	$427.09	0	$533.87	2	$533.87
280+43 - 285+71	4	$320.32	2	$533.87	4	$320.32
285+71 - 290+99	5	$213.55	3	$427.09	5	$213.55
290+99 - 296+27	3	$427.09	0	$533.87	2	$533.87
296+27 - 301+55	3	$427.09	1	$533.87	3	$427.09
301+55 - 306+83	2	$533.87	1	$533.87	3	$427.09
306+83 - 312+11	3	$427.09	0	$533.87	2	$533.87
312+11 - 317+39	3	$427.09	1	$533.87	3	$427.09
317+39 - 322+67	1	$533.87	0	$533.87	2	$533.87
322+67 - 327+95	4	$320.32	1	$533.87	3	$427.09
327+95 - 333+23	2	$533.87	0	$533.87	2	$533.87
333+23 - 338+51	1	$533.87	0	$533.87	2	$533.87
338+51 - 343+79	1	$533.87	1	$533.87	3	$427.09
343+79 - 349+07	2	$533.87	0	$533.87	2	$533.87
349+07 - 354+35	2	$533.87	1	$533.87	3	$427.09
354+35 - 359+63	3	$427.09	2	$533.87	4	$320.32
359+63 - 364+91	1	$533.87	0	$533.87	2	$533.87
364+91 - 370+19	6	$106.77	4	$320.32	6	$106.77
370+19 - 375+47	4	$320.32	1	$533.87	3	$427.09
375+47 - 380+75	1	$533.87	0	$533.87	2	$533.87
380+75 - 386+03	1	$533.87	0	$533.87	2	$533.87
386+03 - 391+31	1	$533.87	0	$533.87	2	$533.87
391+31 - 396+59	5	$213.55	2	$533.87	4	$320.32
396+59 - 401+87	6	$106.77	3	$427.09	5	$213.55

401+87 - 407+15	7	$0.00	3	$427.09	5	$213.55
407+15 - 412+43	11	($106.77)	5	$213.55	7	$0.00
412+43 - 417+71	4	$320.32	3	$427.09	5	$213.55
417+71 - 422+99	2	$533.87	1	$533.87	3	$427.09
422+99 - 428+27	5	$213.55	3	$427.09	5	$213.55
428+27 - 433+55	6	$106.77	4	$320.32	6	$106.77
433+55 - 438+83	3	$427.09	3	$427.09	5	$213.55
438+83 - 444+11	4	$320.32	4	$320.32	6	$106.77
444+11 - 449+39	3	$427.09	1	$533.87	3	$427.09
449+39 - 454+67	2	$533.87	0	$533.87	2	$533.87
454+67 - 459+95	3	$427.09	1	$533.87	3	$427.09
459+95 - 465+23	6	$106.77	4	$320.32	6	$106.77
465+23 - 470+51	1	$533.87	1	$533.87	3	$427.09
NB Subtotal		$28,401.62		$44,417.57		$33,740.27

SOUTHBOUND						
Station	State		Contractor		Adjusted	
	PI	Bonus	PI	Bonus	PI	Bonus
0+59 - 5+87	11	($106.77)	3	$427.09	5	$213.55
5+87 - 11+15	11	($106.77)	3	$427.09	5	$213.55
11+15 - 16+43	10	$0.00	0	$533.87	2	$533.87
16+43 - 21+71	12	($213.55)	0	$533.87	2	$533.87
21+71 - 26+99	12	($213.55)	3	$427.09	5	$213.55
26+99 - 32+27	6	$106.77	1	$533.87	3	$427.09
32+27 - 37+55	3	$427.09	0	$533.87	2	$533.87
37+55 - 42+83	4	$320.32	1	$533.87	2	$533.87
42+83 - 48+11	1	$533.87	0	$533.87	2	$533.87
48+11 - 53+39	5	$213.55	2	$533.87	4	$320.32
53+39 - 58+67	3	$427.09	1	$533.87	3	$427.09
58+67 - 63+95	5	$213.55	1	$533.87	3	$427.09
63+95 - 69+23	5	$213.55	1	$533.87	3	$427.09
69+23 - 74+51	2	$533.87	1	$533.87	3	$427.09
74+51 - 79+79	3	$427.09	0	$533.87	2	$533.87
79+79 - 85+07	4	$320.32	3	$427.09	5	$213.55
85+07 - 90+35	3	$427.09	0	$533.87	2	$533.87
90+35 - 95+63	5	$213.55	1	$533.87	3	$427.09
95+63 - 100+91	4	$320.32	0	$533.87	2	$533.87
100+91 - 106+19	4	$320.32	0	$533.87	2	$533.87
106+91 - 111+47	1	$533.87	0	$533.87	2	$533.87
111+47 - 116+75	3	$427.09	1	$533.87	3	$427.09

116+75 - 122+03	5	$213.55	2	$533.87	4	$320.32
122+03 - 127+31	3	$427.09	0	$533.87	2	$533.87
127+31 - 132+59	3	$427.09	0	$533.87	2	$533.87
132+59 - 137+87	5	$213.55	1	$533.87	3	$427.09
137+87 - 143+15	6	$106.77	1	$533.87	3	$427.09
143+15 - 148+43	3	$427.09	0	$533.87	2	$533.87
148+43 - 153+71	7	$0.00	1	$533.87	3	$427.09
153+71 - 158+99	6	$106.77	1	$533.87	3	$427.09
158+99 - 164+27	6	$106.77	1	$533.87	3	$427.09
164+27 - 169+55	5	$213.55	1	$533.87	3	$427.09
169+55 - 174+83	3	$427.09	1	$533.87	3	$427.09
174+83 - 180+11	7	$0.00	3	$427.09	5	$213.55
180+11 - 185+39	6	$106.77	1	$533.87	3	$427.09
185+39 - 190+67	8	$0.00	1	$533.87	3	$427.09
190+67 - 195+95	6	$106.77	0	$533.87	2	$533.87
195+95 - 201+23	5	$213.55	2	$533.87	4	$320.32
201+23 - 206+51	4	$320.32	0	$533.87	2	$533.87
206+51 - 211+79	8	$0.00	2	$533.87	4	$320.32
211+79 - 217+07	4	$320.32	0	$533.87	2	$533.87
217+07 - 222+35	3	$427.09	0	$533.87	2	$533.87
222+35 - 227+63	9	$0.00	2	$533.87	4	$320.32
227+63 - 232+91	5	$213.55	1	$533.87	3	$427.09
232+91 - 238+19	8	$0.00	3	$427.09	5	$213.55
238+19 - 243+47	5	$213.55	0	$533.87	2	$533.87
243+47 - 248+75	6	$106.77	3	$427.09	5	$213.55
248+75 - 254+03	1	$533.87	1	$533.87	3	$427.09
254+03 - 259+31	7	$0.00	1	$533.87	3	$427.09
259+31 - 264+59	6	$106.77	2	$533.87	4	$320.32
264+59 - 269+87	4	$320.32	0	$533.87	2	$533.87
269+87 - 275+15	8	$0.00	3	$427.09	5	$213.55
275+15 - 280+43	3	$427.09	0	$533.87	2	$533.87
280+43 - 285+71	9	$0.00	2	$533.87	4	$320.32
285+71 - 290+99	7	$0.00	2	$533.87	4	$320.32
290+99 - 296+27	3	$427.09	0	$533.87	2	$533.87
296+27 - 301+55	2	$533.87	1	$533.87	3	$427.09
301+55 - 306+83	4	$320.32	1	$533.87	3	$427.09
306+83 - 312+11	4	$320.32	1	$533.87	3	$427.09
312+11 - 317+39	3	$427.09	0	$533.87	2	$533.87
317+39 - 322+67	2	$533.87	0	$533.87	2	$533.87
322+67 - 327+95	9	$0.00	3	$427.09	5	$213.55

327+95 - 333+23	3	$427.09	0	$533.87	2	$533.87
333+23 - 338+51	2	$533.87	0	$533.87	2	$533.87
338+51 - 343+79	2	$533.87	0	$533.87	2	$533.87
343+79 - 349+07	1	$533.87	0	$533.87	2	$533.87
349+07 - 354+35	5	$213.55	1	$533.87	3	$427.09
354+35 - 359+63	4	$320.32	2	$533.87	4	$320.32
359+63 - 364+91	3	$427.09	1	$533.87	3	$427.09
364+91 - 370+19	9	$0.00	6	$106.77	7	$0.00
370+19 - 375+47	5	$213.55	1	$533.87	3	$427.09
375+47 - 380+75	1	$533.87	0	$533.87	2	$533.87
380+75 - 386+03	2	$533.87	0	$533.87	2	$533.87
386+03 - 391+31	1	$533.87	0	$533.87	2	$533.87
391+31 - 396+59	7	$0.00	3	$427.09	5	$213.55
396+59 - 401+87	12	($213.55)	5	$213.55	7	$0.00
401+87 - 407+15	11	($106.77)	2	$533.87	4	$320.32
407+15 - 412+43	15	($533.87)	5	$213.55	7	$0.00
412+43 - 417+71	7	$0.00	1	$533.87	3	$427.09
417+71 - 422+99	6	$106.77	1	$533.87	3	$427.09
422+99 - 428+27	12	($213.55)	5	($213.55)	7	$0.00
428+27 - 433+55	8	$0.00	4	$320.32	6	$106.77
433+55 - 438+83	6	$106.77	1	$533.87	3	$427.09
438+83 - 444+11	6	$106.77	6	$106.77	7	$0.00
444+11 - 449+39	4	$320.32	2	$533.87	4	$320.32
449+39 - 454+67	2	$533.87	0	$533.87	2	$533.87
454+67 - 459+95	6	$106.77	2	$533.87	4	$320.32
459+95 - 465+23	8	$0.00	4	$320.32	6	$106.77
465+23 - 470+51	1	$533.87	0	$533.87	2	$533.87
SB Subtotal		$19,539.46		$44,204.02		$35,236.09
TOTAL		$47,941.08		$88,621.59		$88,975.38

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM
SYNTHESIS OF HIGHWAY PRACTICE **167**

MEASUREMENTS, SPECIFICATIONS, AND ACHIEVEMENT OF SMOOTHNESS FOR PAVEMENT CONSTRUCTION

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TRANSPORTATION RESEARCH BOARD
NATIONAL RESEARCH COUNCIL
WASHINGTON, D.C. NOVEMBER 1990

8.7.01

MEASUREMENTS, SPECIFICATIONS, AND ACHIEVEMENT OF SMOOTHNESS FOR PAVEMENT CONSTRUCTION

SUMMARY

The concern about the smoothness of highway surfaces precedes the development of motorized vehicles. In the early days, the simple straightedge was used as the sole indicator of smoothness. But even before the turn of the century, efforts were directed at developing improved devices for smoothness evaluation. From 1900 to near midcentury, numerous devices of increasing complexity were invented. These were primarily mechanical devices with elaborate multi-wheeled support systems. Advances in several technological fields have now been applied to smoothness-measuring equipment, resulting in the incorporation of electrical circuitry, electronics, ultrasonics, lasers, and computerization.

Although the early devices were primarily of concern to the practicing engineer, the advent of test road construction brought the research engineer onto the scene. Many devices were developed in connection with specific research efforts. The automotive industry became interested because of the effect that certain types of pavement had on motor vehicles. In recent years highway managers have recognized that the public rates a highway primarily on its riding characteristic. Thus it is necessary to program an increasing amount of highway funds to address the issue of pavement smoothness on a system-wide basis.

As a consequence, several smoothness-measuring devices have been developed and are in current use. The fundamentals of operation, cost, and appropriateness to address a specific need vary considerably. Certain devices are far better suited than others to the purpose of controlling the smoothness of newly constructed pavements. Therefore, it is important for those concerned with obtaining smoothness in construction to be aware of the equipment best suited for that purpose and the relation of that equipment to the entire spectrum of smoothness-measuring devices.

Smoothness-measuring equipment currently used in new pavement construction includes straightedges (static and rolling), profilographs, response-type road-roughness-measuring systems, and inertial profilometers. All agencies use a straightedge—a few as the sole approach to smoothness control, but most as an adjunct to other equipment. The type of instrument receiving increased application is the profilograph, either the California or Rainhart type. These devices are similar in that they portray graphically certain characteristics of pavement smoothness, are relatively simple mechanical devices, can be used on new concrete pavement surfaces soon after construction, are low-cost/low-maintenance devices, and provide information that is readily acceptable by specifying agencies and the construction industry. Profilographs provide an analog trace to which specification tolerances are applied. The traces can be used to locate specific pavement features in the field. The primary disadvantages with this type of instrument are the slow speed of operation (3 mph) and the time required for evaluating the profiles, although the latter item has been addressed by computerized models that are now available. Other disadvantages include the exaggeration and suppression of parts of the surface wavelength spectrum, the occasional

exclusion by the blanking band of surface irregularities that may be of importance, and a mediocre correlation to other reference roughness standards.

Other devices being used in evaluating smoothness of new construction, including response-type road-roughness-measuring systems and inertial profilometers, are used considerably less often than profilographs for a variety of reasons. They are not able to be used on concrete pavements for a considerable time after paving (i.e., until the concrete gains sufficient strength), they don't allow ready identification or location of pavement surface aberrations, and, in some cases, they are very costly items. However, they can operate at high speeds; thus a considerable amount of data can be obtained at a lower cost. Also, the smoothness statistic is achieved with little or no manual processing. High-speed equipment has its greatest application in entire highway system assessment, research applications, and for calibration purposes.

Numerous research efforts as well as symposia and workshops have been directed toward providing information on the use of smoothness-measuring equipment. Although there are vast differences in equipment types and their ultimate application, the relationships of several smoothness indexes have been compared and are reasonably well defined.

A survey of practices in use in the United States and Canada revealed that there is a wide diversity in the use of smoothness specifications and equipment. However, emphasis on smoothness by specifying agencies, together with strong support from the construction industry, has led to the attainment of increasingly smoother pavements.

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7. Author(s) Larry A. Scofield, Sylvester Kalevela, Mary Anderson, Asm Hossain	8. Performing Organization Report No.
9. Performing Organization Name and Address Arizona Transportation Research Center College of Engineering, ERC, Rm 405 Arizona State University	10. Work Unit No.
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12. Sponsoring Agency Name and Address ARIZONA DEPARTMENT OF TRANSPORTATION 206 S. 17TH AVENUE PHOENIX, ARIZONA 85007	13. Type of Report & Period Covered Final-Sept. 1990 - June 1991
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15. Supplementary Notes

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16. Abstract

This study was performed to establish equipment and operator variability for mechanical and computerized California profilographs. Future work, based on testing conducted during this study, should develop precision and bias statements for profilographs.

The research consists of two phases. Phase I, reported herein, provided a literature review, performed the field testing and conducted the statistical anlysis. The historical development of the profilograph and California test procedures and specifications were evaluated in relationship to todays incentive/disincentive specifications. Additionally, equipment parameters which influence test variability were reviewed.

Two field experiments were conducted. The first experiment, designed to evaluate variability, consisted of a 4x4x2 randomized block design with replication. Two levels of pavement roughness, four operators, and four profilogrphs were utilized. The second experiment, designed to evaluate the effects of data filter settings on profile index obtained with computerized profilographs, consisted of a 3x2x2x2 randomized block design with replication. Two levels of pavement roughness, two computerized profilographs, two operataors, and three data filter settings were used.

The results of the study indicated that the average repeatability was 0.75 inches/mile and 0.56 inches/mile for the rough and smooth track conditions, respectfully.

The average repeatability for an operator performing trace reduction was 0.94 inches/mile for one device and 1.72 inches/mile for a second device.

The data filter setting used on computerized profilographs has a significant affect on the resulting profile index. For each 1000 unit change in the data filter setting, a 7% reduction in the profile index was obtained when compared to the manufacturers recommended value of 8000.

23. Registrant's Seal

17. Key Words	18. Distribution Statement
Profilograph, Pavement Roughness, Pavement Smoothness, California Profilograph, Profile, Specifications, Incentive, Disincentive	Document is available to the U.S. public through the National Technical Information Service, Springfield, Virginia 22161

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