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FRICTIONAL PROPERTIES OF HIGHWAY SURFACES

Final Report

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16. Abstract Purpose of study was to evaluate effects of differential friction; compare experimental & standard texturing techniques of PCCP; determine if Mu-Meter is capable of measuring frictional effects of grooving & develop wet pavement accident analysis system for use in Arizona. All phases of project were successfully completed.

Results of program have shown that differential friction can cause an extremely hazardous condition for a braking vehicle. As this problem can occur at high or low friction levels, & present maintenance & construction techniques can cause differential friction, it should be given major consideration in any pavement friction evaluation. Equations have been derived which will allow engineer to determine magnitude of problem.

There is a strong indication that differential friction may be as important in causation of wet pavement accidents as low friction levels. If so, a major reevaluation of present friction evaluation & corrective techniques may be necessary.

All burlap drag textures tested produced poor, non-uniform finishes that were significantly less durable than other textures tested. For this reason, it is suggested that burlap drag texture not be used in the future. Nylon bristle broom texture produced best overall results, while magnesium fluted float produced unusual texture that definitely has merit.

The Mu-Meter is capable of measuring side-force effects of grooving PCCP. Benefits obtained from longitudinally grooving, however, are dependent on original friction level.

Through study, a viable system has been developed for predicting wet exposure accident rates & for realistically comparing wet-dry accident results.

17. Key Words Differential Friction, Skidding, Vehicle Control, Concrete Texturing, Durability, Friction, Noise, Grooving, Mu-Meter, Accident, Wet Pavement, Analysis, Rainfall, Remedial Actions, Pavement Friction, Side Force	18. Distribution Statement No Restrictions This document is available to the public thru NTIS, Springfield, Virginia, 22161.

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ABSTRACT

On May 1, 1973, the Arizona Department of Transportation initiated a research program entitled "Frictional Properties of Highway Surfaces" sponsored by the Federal Highway Administration. The specific aim of this program was to determine the effects of particular pavement friction properties on vehicular traffic. The program evaluated the following:

1. The effects of differential wheelpath friction.
2. Comparison of standard and experimental texturing of portland cement concrete pavement.
3. Frictional effects of grooving as measured by the Mu-Meter.
4. Development of a wet accident analysis system for the State of Arizona and other areas with arid climates.

This research has yielded results which are of significant importance and which indicate a major re-evaluation of present pavement friction evaluation and corrective techniques may be necessary.

Some of the major findings of this research are as follows:

1. Differential friction can cause an extremely hazardous condition for a braking vehicle. As this problem can occur at high as well as low friction levels, it should be given major consideration in any pavement friction evaluation.
2. Present construction and maintenance techniques can acutally create this problem.

3. There are numerous causes of differential friction and there is a strong indication that differential friction may be as important in the causation of wet pavement accidents as low friction level. If this is the case, a major re-evaluation of pavement friction evaluation, design, and corrective techniques may be necessary.
4. The Mu-Meter, unlike the ASTM locked wheel skid trailer, is capable of detecting the frictional effects of grooving.
5. Thru this study a viable system for predicting wet exposure accident rates has been developed for Arizona and other areas with arid climates.

As this study investigated several separate subjects, the final report has been broken into four separate parts. Each part will describe the work performed and the results for that phase of the research. At the end of the paper, the conclusions, recommendations and implementation for all four parts are discussed.

Parts 1, 3 and 4 of this study were accomplished with FHWA, HPR Funds, while Part 2 was performed under the FHWA Experimental Projects Program. Evaluation of pavement textures as described in Part 2 will be continued.

FRICTIONAL PROPERTIES OF HIGHWAY SURFACES

PART ONE

DIFFERENTIAL FRICTION RELATED TO SKIDDING

INTRODUCTION

As part of the Federally sponsored research project HPR 1-12 (146), Frictional Properties of Highway Surfaces, a thorough evaluation of the phenomena of differential friction was made. This portion of the paper describes the work that was performed which evaluated the effects differential friction has on a skidding car. It also describes the magnitude of the problem, as well as some of the causes and possible solutions.

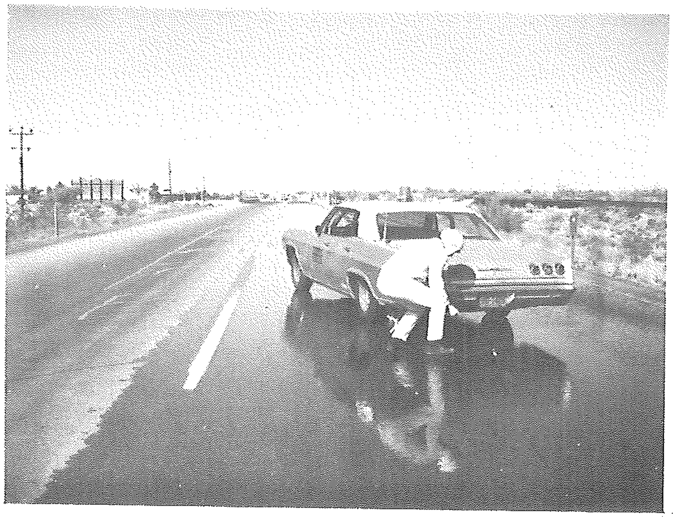
DIFFERENTIAL FRICTION

Differential wheelpath friction is a term the author has derived to describe the condition that occurs when the individual wheelpaths on which a vehicle rides have different or unequal coefficients of friction. Although this phenomena is usually not considered in a pavement friction evaluation, it can have a significant effect on a braking vehicle.

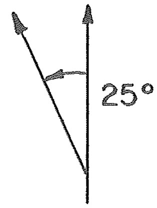
Several years ago it was noted that, during stopping distance tests with a skidding car, some pavements caused a car to spin uncontrollably[1]. Under normal conditions, the car was designed to stop in a straight line and not rotate. It was found that this rotation was produced by different friction levels in each of the wheelpaths. As the tires on the left side of the car were exposed to a different friction level than those on the right side, a turning movement was caused, with the vehicle rotating towards the higher friction side. As the car spun, the front wheels moved onto a higher friction level and the rear wheels onto the lower. Thus the car was still unstable and the spin continued until the vehicle stopped. An example of this is shown in Figure 1, where the right wheeltrack was beginning to flush (excessive asphalt on surface) and the left (on a different ribbon) was not. The wet stopping distance number (SDN) at 40 miles per hour (MPH), (SDN_{40}), was 50 for the right wheelpath and 60 for the left wheelpath. This is only a ten SDN, or 17% difference. In Picture A, the car skidded at 30 MPH and rotated 25° counterclockwise. In Picture B, the car skidded at 40 MPH and rotated 40° counterclockwise. In Picture C, the car skidded at 50 MPH and rotated 95° counterclockwise. When the direction of skidding was reversed, the similar values were recorded with the car turning clockwise. A more severe case is shown in Figure 2, in which the left wheeltrack was bleeding and the right wheeltrack had been chip sealed. The right wheelpath had a wet SDN_{40} of 67 and the left wheelpath had a wet SDN_{40} of 41, which is a 26 SDN, or 39 percent difference. In Picture A, the car skidded at 40 MPH and rotated 90° clockwise. In Picture B, the car skidded at 50 MPH and rotated 270° clockwise. Again the direction of skidding was reversed and the same values were recorded with the car rotating counterclockwise.

Figure 2 is an extreme case used to portray what might happen if one wheelpath was flushing and the other was not. The case shown in Figure 1 is common and although both wheelpaths have a satisfactory level of friction,

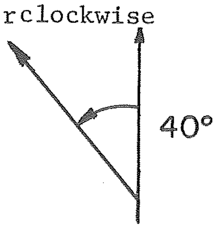
RIGHT WHEEL PATH = SDN_{40} 50
LEFT WHEEL PATH = SDN_{40} 60
17% Difference

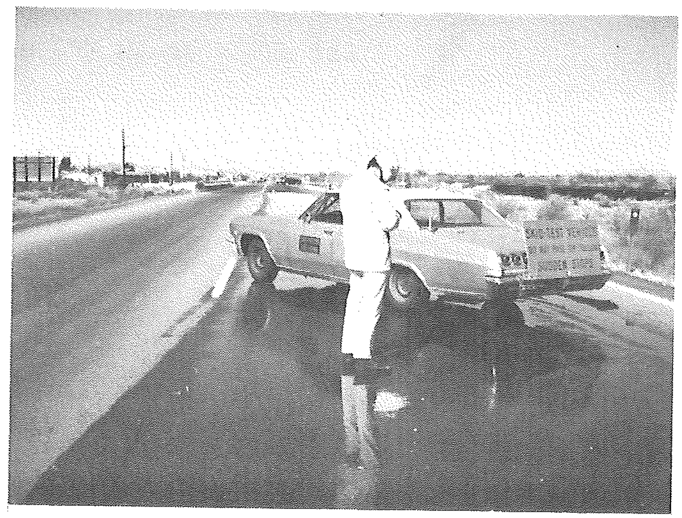


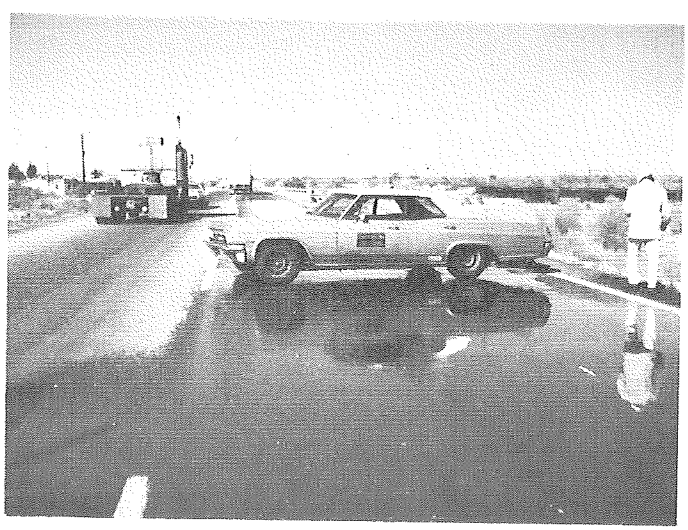
(A) Car skids at 30 MPH and rotates 25° counterclockwise



(B) Car skids at 40 MPH and rotates 40° counterclockwise







(C) Car skids at 50 MPH and rotates 95° counterclockwise

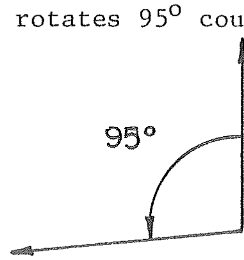
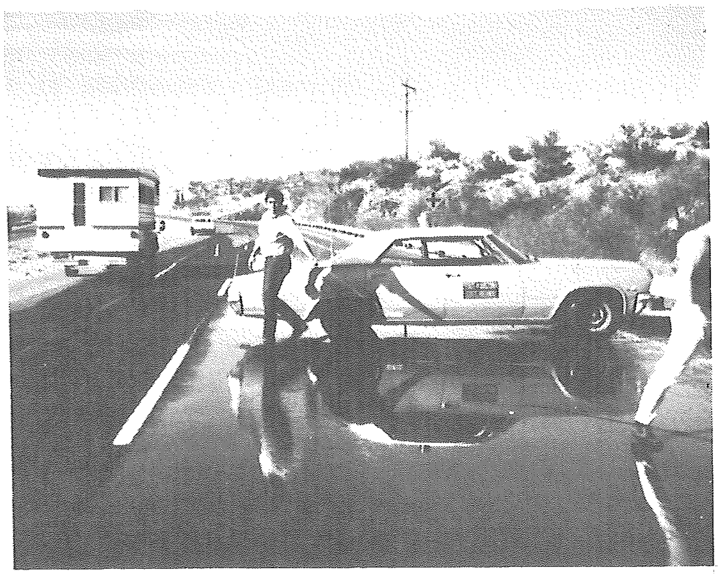


Figure 1

RIGHT WHEEL PATH = SDN_{40} 67

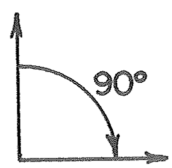
39% Difference

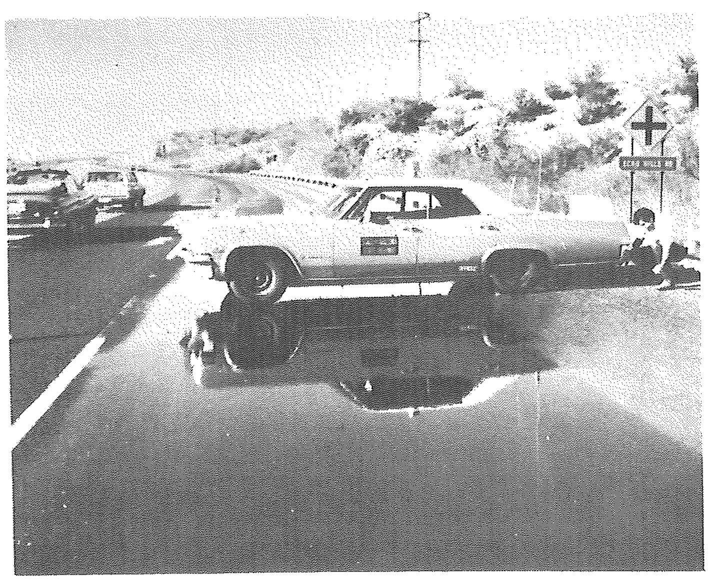
LEFT WHEEL PATH = SDN_{40} 41



(A)

Car skids at 40 MPH and rotates 90° clockwise





(B)

Car skids at 50 MPH and rotates at 270° clockwise

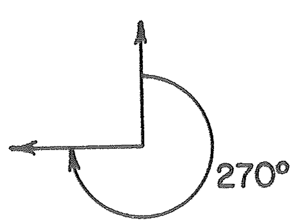


Figure 2

a hazardous condition occurs due to their difference. As the speed increases, the effects of differential friction increase dramatically. Under such conditions, the average driver tends to remove his foot from the brake as he begins to rotate. When he does, his car is propelled in the direction the vehicle is facing. This could be off the road or into oncoming traffic.

It was because of the potential hazard that this condition might cause, that a special research project was initiated to study this and other frictional problems. The project was HPR 1-12 (146) Frictional Properties of Highway Surfaces, which was begun in May 1973. The remainder of Part I will discuss the findings of this phase of the research program.

LOCKED WHEEL SKID TESTS

In order to fully evaluate the effects of differential friction, it was necessary to evaluate numerous locations to obtain test sites that would have a wide range of differential friction numbers and surface types. From thirty potential sites, sixteen were selected for detailed testing. The tests were performed using a 1969 Plymouth Fury and a 1973 AMC Matador.

Detailed testing included the following:

1. Measurement of the individual wet Stopping Distance Number[2] (SDN) for each side at 20 and 40 MPH with the locked wheel skid car. The value recorded for each side was used to represent an individual wheelpath when the surfaces were tested simultaneously.

2. Measurement of each individual side with the Mu-Meter at 20 and 40 MPH, after which the Mu Number (MuN) was recorded.
3. Measurement of the wet SDN on both surfaces simultaneously at 20, 30, 40 and 50 MPH. Besides measuring the stopping distance, the degree of rotation was also measured at each speed.
4. Skidding in the reverse direction to insure that the rotation was due to the differential friction and not due to the mechanics of the vehicle or any possible driver bias.

When the differential stopping distance tests were performed as described, special care was taken to keep the tests uniform. This was done in the following manner. When the desired test speed was reached, the driver would lock the brakes and hold the wheel from rotating, thus keeping the wheels pointed straight ahead with reference to the car. It was decided to use the straight wheel method because it was found that when all the brakes were locked, rotating the wheel had no effect (as expected) on reducing the spin of the car. The only effect it did have, was to possibly cause the driver to lose his concentration and not keep the brakes fully locked. If this had happened when the driver was in a spin, a major accident could have occurred.

When the car stopped, the rotation in degrees was measured. This was done in reference to a large protractor fastened to the trunk of the vehicle and a fifteen-foot (4.57m) length of string, which was extended from the center of the protractor and aligned parallel to the centerline of the highway. This method proved very accurate and reliable.

LOCKED WHEEL SKID TEST RESULTS

The actual skid test results are shown in Appendix A. From these diagrams it can be seen that as the differential friction level increases, the amount of rotation in degrees per foot of skid also increases. This indicates that although a pavement has a good coefficient of friction in both its wheelpaths, if they are unequal it may cause a more hazardous condition than a low friction pavement with uniform friction. Similar findings were reported by Zuk at the University of Virginia[3]. In the bottom-right-hand corner of these figures is the skid number (actual stopping distance number) recorded for the individual wheelpaths at 20 and 40 MPH. The differential friction at 40 MPH is obtained by calculating the absolute difference between the SDN for each wheelpath recorded at 40 MPH. To calculate the rotation in degrees per foot, the actual rotation at a particular speed (shown in the center of the figure), is divided by the stopping distance (bottom left) recorded at that speed. The rotation in degrees per foot (3.28 deg. per meter), shown in the right corner, is that recorded for 40 MPH.

A complete listing of the test results recorded for all sixteen sites is shown in Figure 3. This data was used to derive equations relating the rotation of a car to SDN, Differential Friction Number and vehicle speed. The Differential Friction Number (DFN) is defined by the author as the absolute difference between the wet SDN's of each wheelpath recorded at the same speed and subscripted by that speed if the test speed was other than 40 MPH.

Figures 4, 5 and 6 show the derived equations for 30, 40 and 50 MPH relating the degrees per foot rotation to the DFN recorded at 40 MPH. The correlation coefficients ranged between 0.93 and 0.94 and, as can be seen from the graphs, reliable equations were derived. Figure 7 shows all three equations plotted on the same graph. Site Number six was not used in the derivation of these equations, as it contained a portland cement concrete pavement (PCCP) and asphaltic concrete (AC) surface differential. The degree of rotation was high, but lower than expected. This may have been due to the longitudinal burlap drag texture on the PCCP pavement which would inpart a higher side force friction and thus inhibit rotation. If this is the case, grooving or grinding might be used to correct a differential friction problem.

From these three equations, a new equation was derived for use with any speed. The equation is as follows:

$$\text{Deg/Ft.} = 0.148 - .0049(V) + [.00263 + .009(V)]\text{DFN}$$

where:

Deg/Ft. = Degrees per foot rotation for given velocity (V)

FIGURE 3

DIFFERENTIAL WHEELPATH FRICTION TEST RESULTS

Separate Wheelpath Test Data					Differential Test Data					
Site Number	Speed MPH	Left Wheel Track Average SDN	Right Wheel Track Average SDN	Differential Friction Number	Test Speed MPH	Stopping Distance Feet	Stopping Distance Meters	Degrees* Rotation	Degrees Per Foot Rotation	Degrees Per Meter Rotation
1	20	64	70	6	22	25.5	7.8	---	---	
	40	57	62	5	30	48.5	14.8	---	---	
					40	96.5	29.4	16 cw	.166	.545
					50	149.5	45.6	12 cw	.080	.262
2	20	44	64	20	21	23.5	7.2	15 cw	.638	2.093
	40	36	57	21	31	55.5	16.9	27	.487	1.598
					42	122.0	37.2	95	.779	2.556
					48	190.5	58.1	142	.745	2.444
3	20	41	56	15	20	27.5	8.4	5 cw	.182	.597
	40	31	45	14	32	73.5	22.4	30	.408	1.338
					41	145.0	44.2	82	.566	1.857
					48	208.0	63.4	137	.659	2.162
4	20	56	62	6	20	27.5	8.4	10 cw	.364	1.194
	40	40	49	9	30	67.0	20.4	17	.254	.833
					40	120.0	36.6	57	.475	1.558
					49	205.0	62.5	129	.629	2.064
5	20	61	63	2	20	20.5	6.2	6 cw	.293	.961
	40	52	61	9	31	56.0	17.0	14	.250	.820
					40	97.5	29.7	25	.256	.840
					50	163.0	49.7	37	.227	.745
6	20	51	76	25	20	21.0	6.4	14 cw	.667	2.188
	40	47	73	26	31	51.5	15.7	34	.654	2.146
					40	92.5	28.2	57	.612	2.008
					51	157.0	47.9	119	.758	2.487
7	20	42	44	2	20	29.3	8.9	6 cw	.205	.672
	40	42	48	6	30	67.1	20.4	16	.238	.781
					40	117.5	35.8	27	.230	.754
					49	174.1	53.1	42	.241	.791
8	20	49	61	12	32	75.5	23.0	22 cw	.291	.955
	40	36	44	8	40	143.2	43.6	24	.168	.551
					45	157.2	47.9	28	.178	.584

FIGURE 3
Continued

DIFFERENTIAL WHEELPATH FRICTION TEST RESULTS

Separate Wheelpath Test Data					Differential Test Data					
Site Number	Speed MPH	Left Wheel Track Average SDN	Right Wheel Track Average SDN	Differential Friction Number	Test Speed MPH	Stopping Distance Feet	Stopping Distance Meters	Degrees* Rotation	Degrees Per Foot Rotation	Degrees Per Meter Rotation
9	20	60	38	22	21	24.5	7.5	6 ccw	.245	.804
	40	47	25	22	30	65.6	20.0	40	.611	2.004
					41	165.0	50.3	149	.903	2.963
					50	245.0	74.7	275	1.122	3.681
10	20	58	40	18	21	29.8	9.1	9 ccw	.302	.991
	40	49	32	17	32	82.2	25.1	50	.608	1.995
					41	150.4	45.8	139	.924	3.031
					50	244.6	74.6	209	.854	2.802
						152.1	46.4			
11	40	53	57	4	30	51.0	15.5	5 cw	.098	.322
					40	98.0	29.9	12	.122	.400
					50	185.0	56.4	35	.189	.620
12	20	58	56	2	21	31.4	9.6	1 cw	.032	.105
	40	45	49	4	30	66.3	20.2	4	.060	.197
					40	126.3	38.5	17	.135	.443
13	40	60	49	11	40	100.4	30.6	23 ccw	.229	.751
14	40	58	46	12	30	56.0	17.1	26 ccw	.464	1.522
					40	95.0	29.0	37	.389	1.276
					50	167.0	50.9	81	.485	1.591
15	40	57	38	19	31	60.0	18.3	30 ccw	.500	1.640
					41	120.0	36.6	98	.816	2.677
					51	206.7	63.0	173	.837	2.746
16	40	51	55	4	30	51.0	15.5	0 cw	.000	0.0
					40	95.0	29.0	6	.063	.207
					50	157.0	47.9	12	.076	.249

*cw = Clockwise
*ccw= Counter clockwise

V = Velocity of vehicle in MPH when the brakes are locked.

DFN = Differential Friction Number (Difference between wet Stopping Distance Numbers measured at 40 MPH in each wheelpath).

Deg/meter = 3.28 x Deg/Ft.

From this equation, it can be seen that the rotation in degrees per foot can be determined easily for any desired speed by simply knowing the SDN for each wheelpath.

When this value is multiplied by the stopping distance for the given surface, the total number of degrees that the vehicle will spin can be calculated. Our experiments have shown that the actual stopping distance on the differential surface is approximately equal to the average of the two stopping distances on the individual surfaces at the same speed. For example, if at 40 MPH a car stopped in 100 feet (30.48m) (SDN = 53) in the left wheelpath, and 200 feet (60.96m) (SDN = 27) in the right wheelpath, then when braking on both surfaces simultaneously, the vehicle would stop in 150 feet (45.72m) (SDN = 36, DFN = 26). If the degrees of rotation are desired, the 150 feet (45.72m) would be multiplied by the ^{o}/ft. rotation value, which at 40 MPH would be 1.1 (from the equation) for an estimated total of 165.5 degrees total rotation.

ROTATION (DEGREES/FOOT AT 30 MPH) VS SDN_{40} DIFFERENTIAL

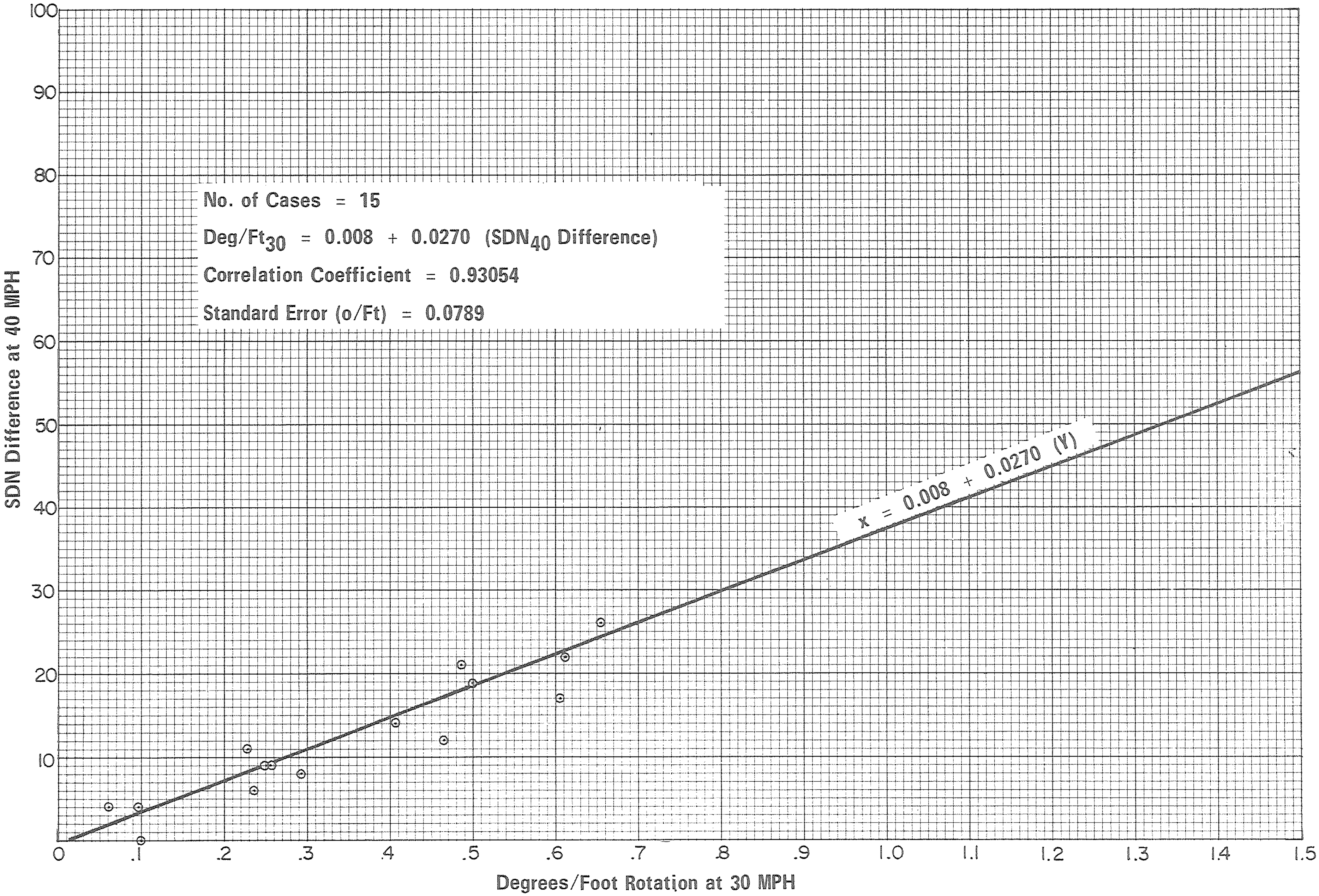


Figure 4

ROTATION (DEGREES/FOOT AT 40 MPH) VS SDN_{40} DIFFERENTIAL

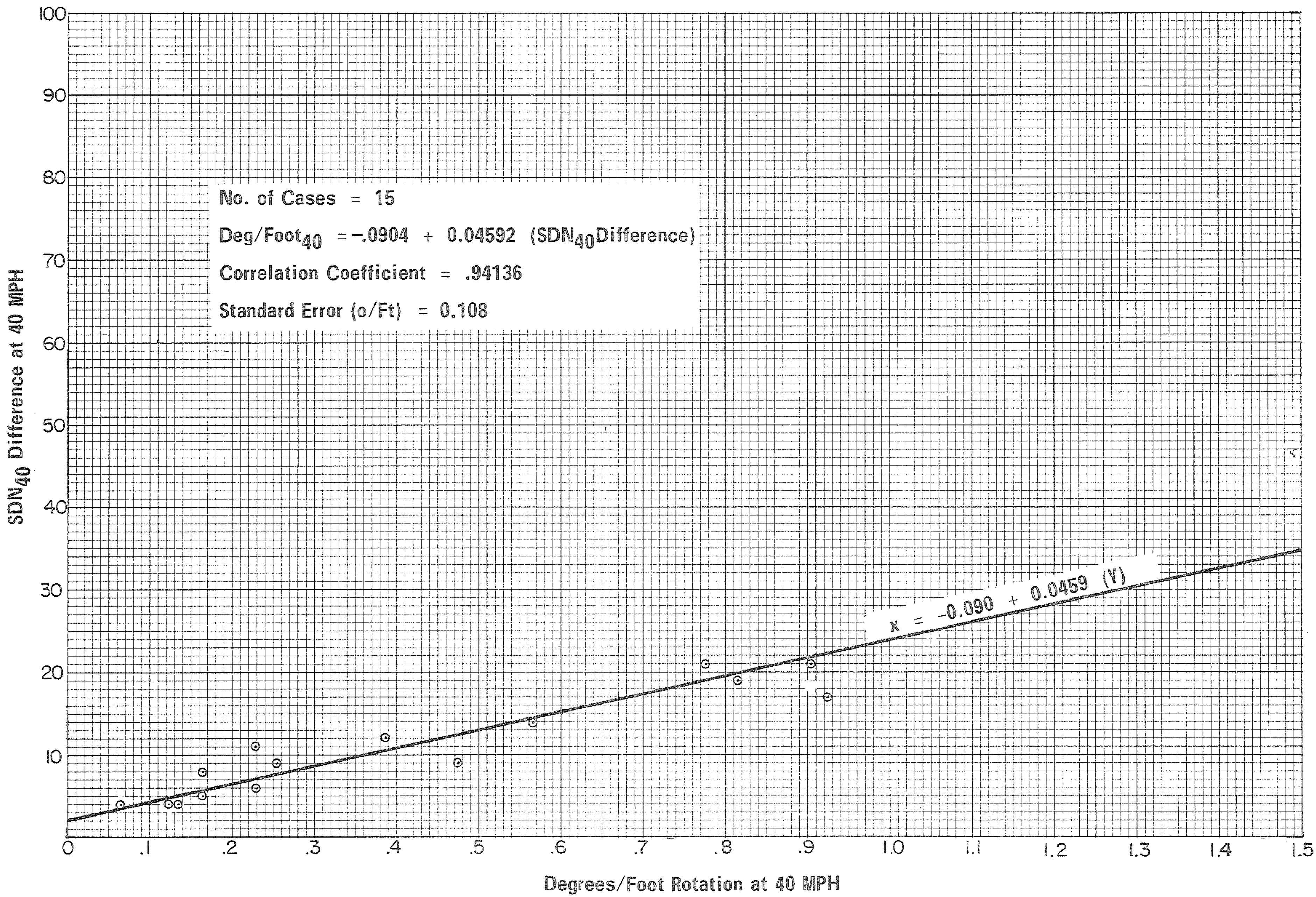
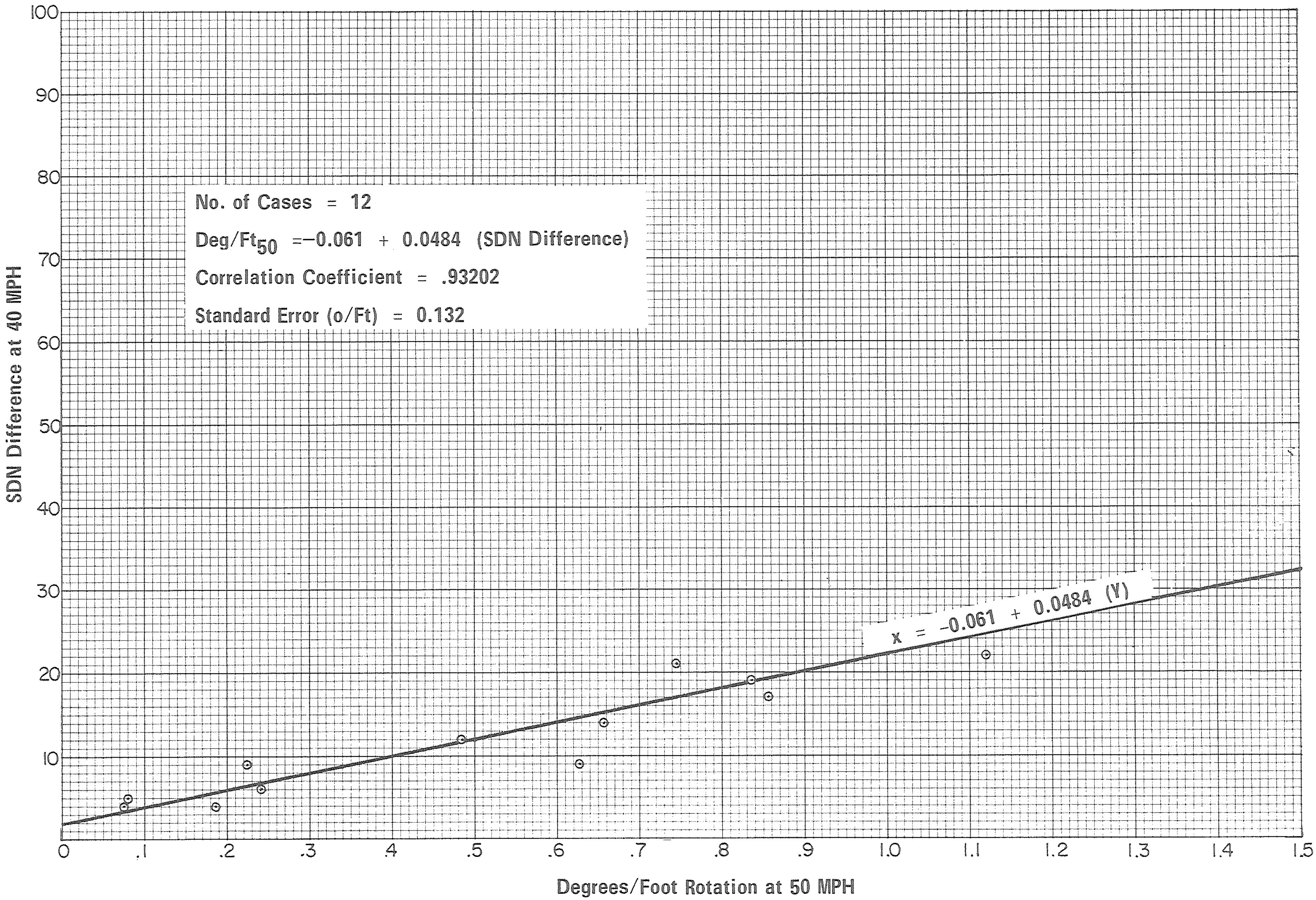


Figure 5

ROTATION (DEGREES/FOOT AT 50 MPH) VS SDN_{40} DIFFERENTIAL



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ROTATION (DEGREES/FOOT) VS SDN_{40} DIFFERENTIAL

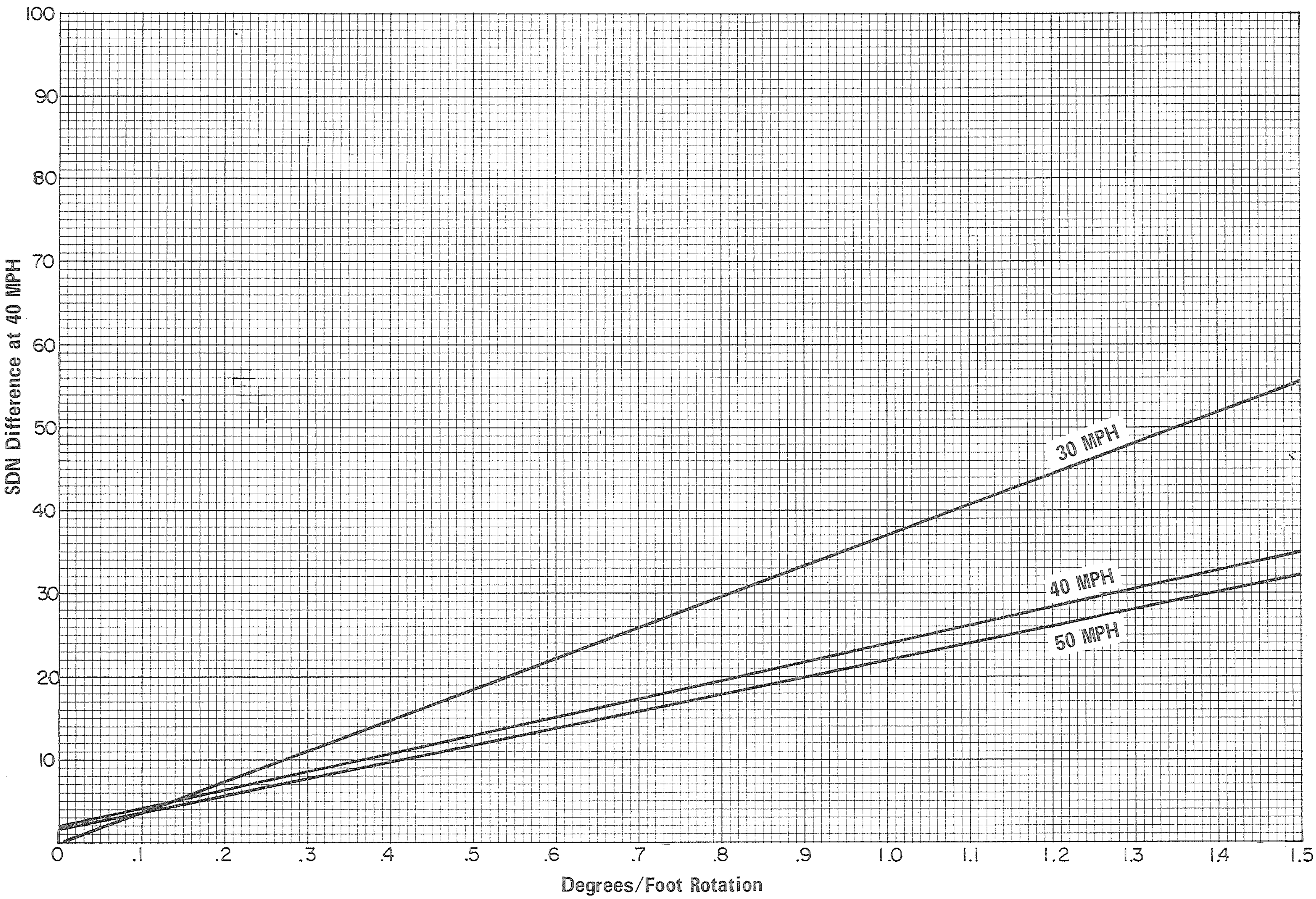


Figure 7

VEHICLE CONTROL TESTS

The second part of the evaluation was concerned with the maneuvering problems associated with the differential friction surface. The normal tendency for a driver is to release the brake after he begins to spin and to try simultaneously to regain control of his vehicle. For this reason we decided to reconstruct a similar condition. This was done in two phases. The first phase was an experiment to determine how many degrees a car could rotate before it could not be safely corrected by the driver. Figure 8 shows the schematic of the system that was used in the test. The premise of the test was that a vehicle would have a twelve-foot (3.66m) wide lane in which it could safely maneuver. If the vehicle moved out of its lane for a distance of up to six feet (1.83m) on either side, it was considered to be in another vehicle's lane and thus in a potential accident zone for oncoming or parallel traffic. The test vehicle was given additional maneuvering space as it was assumed that other traffic could move six feet (1.83m) over and avoid the spinning car. If the test car moved more than six feet (1.83m) into another lane, it was assumed that a theoretical collision with other traffic would occur, as an oncoming vehicle would not be able to avoid the spinning car. Traffic cones were placed along the imaginary collision zone, which allowed our vehicle a width of twenty-four feet (7.32m) in which to maneuver. During each test, the driver was instructed to avoid these cones, as they represented oncoming cars.

VEHICLE CONTROL TESTS

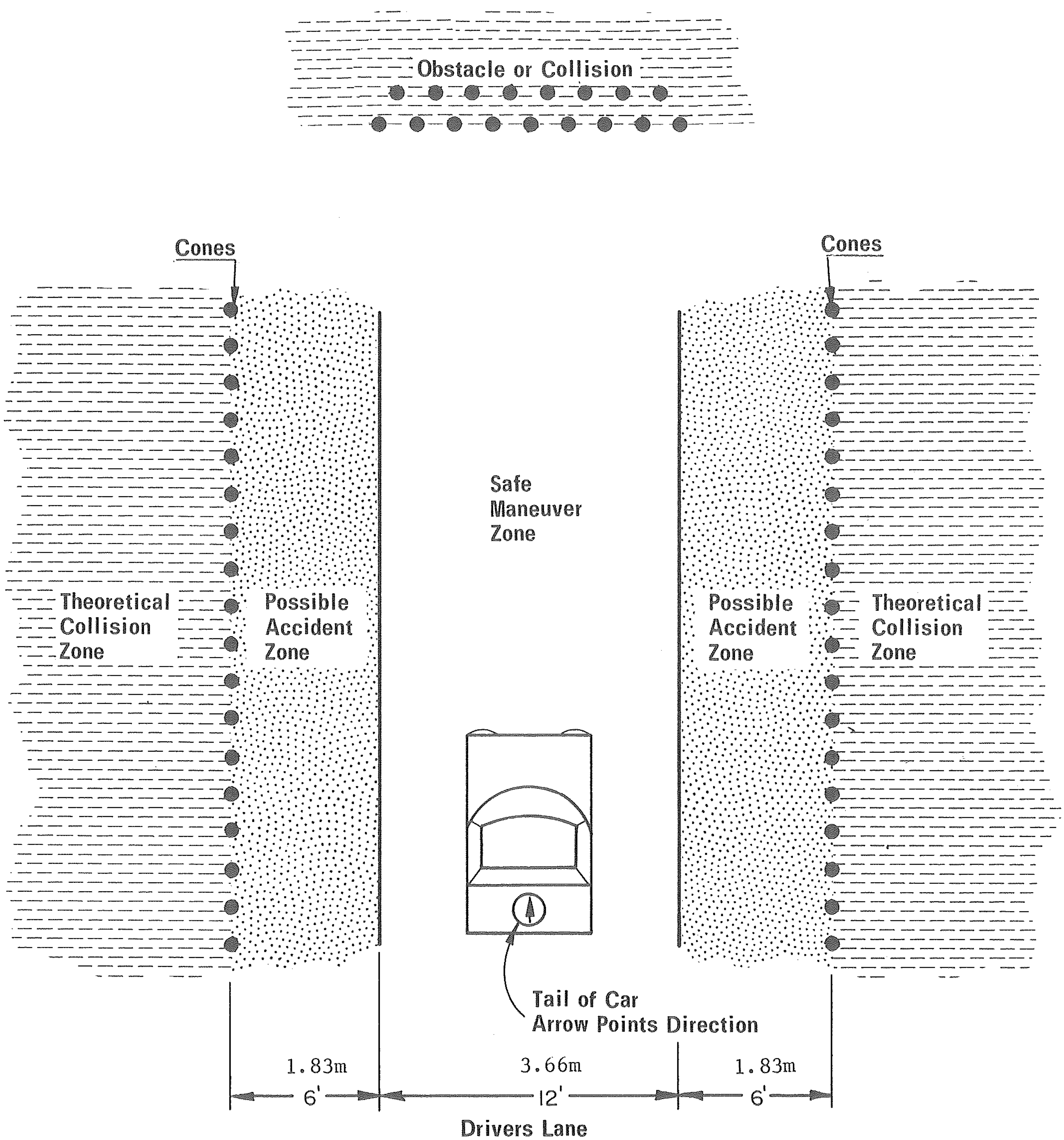


Figure 8

The second phase of the experiment evaluated the control problems associated with stopping when the brakes were not locked and the driver was allowed to freely maneuver his vehicle. In this case, cones were also set at various distances from a beginning braking point. This distance was ten feet (3.05m) further than the maximum braking distance recorded at any particular speed. The driver was then directed to stop in the given distance without hitting the side cones (theoretical collision) or hitting the cones at the end of the distance, which represented an obstacle.

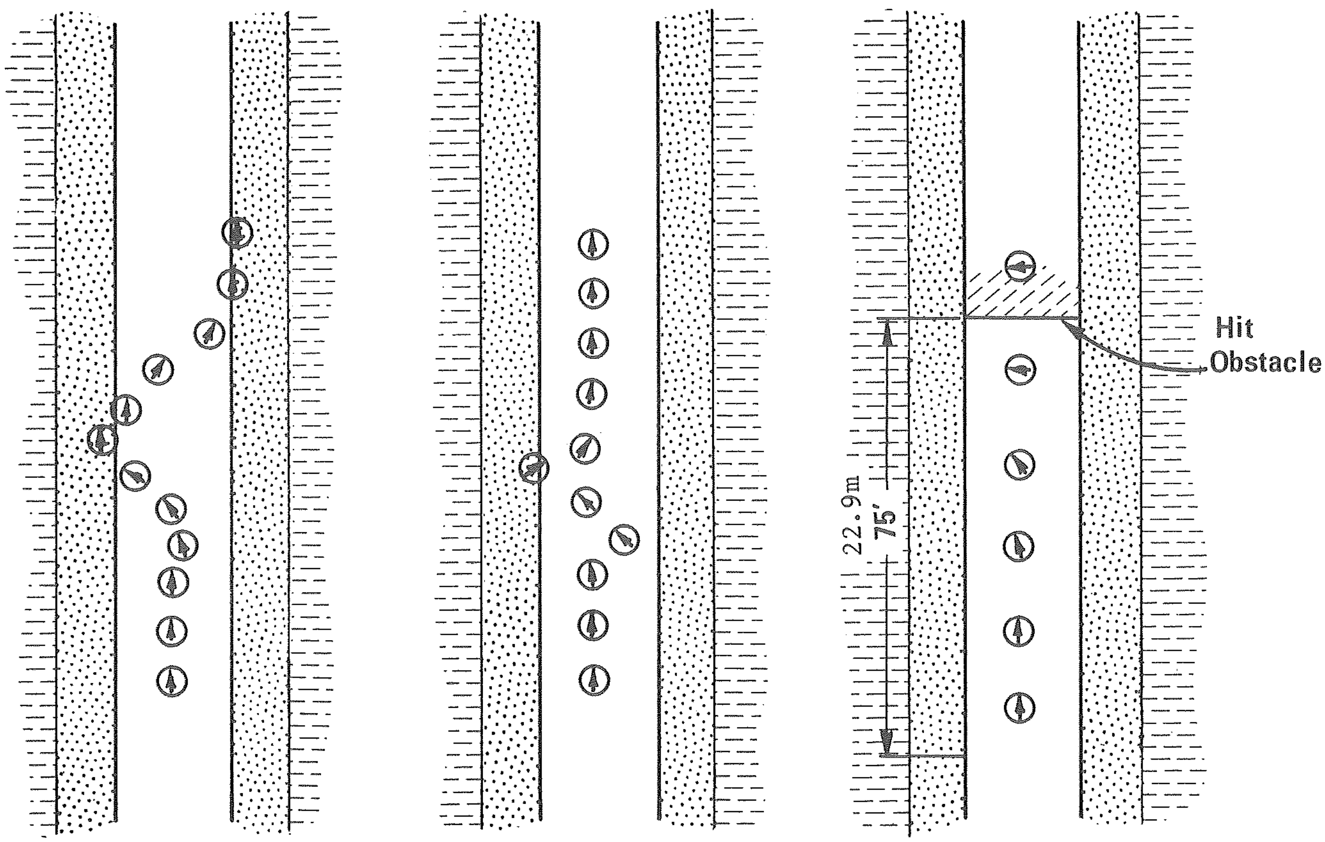
Motion pictures were made of all tests. Figures 9 thru 11 show the trajectory of the vehicle recorded on film for various test speeds. In these figures, a circle was used to represent the position of the rear end of the vehicle. The arrow inside the circle indicates the direction the front of the car was facing at that moment in time.

VEHICLE CONTROL TEST RESULTS

Figure 9 shows the trajectory of the vehicle during tests at 30 MPH. It can be seen that when the vehicle was allowed to rotate twenty degrees before releasing the brake, it moved into the possible accident zone, but did not enter the collision zone. In these tests, the vehicle had almost come to a complete halt before the brakes were released. When an obstacle was placed in the highway, the driver was able to stay in his lane, but unable to stop before reaching the object.

Figure 10 shows the results of the 40 MPH tests. The figure shows

VEHICLE CONTROL TESTS ON A DIFFERENTIAL FRICTION SURFACE (DF_{40}=17)



20° Rotation Prior To Releasing Brake

30° Rotation Prior To Releasing Brake

Unlocked Brake Tests 91 Feet to Stop

30 MPH TESTS

Figure 9

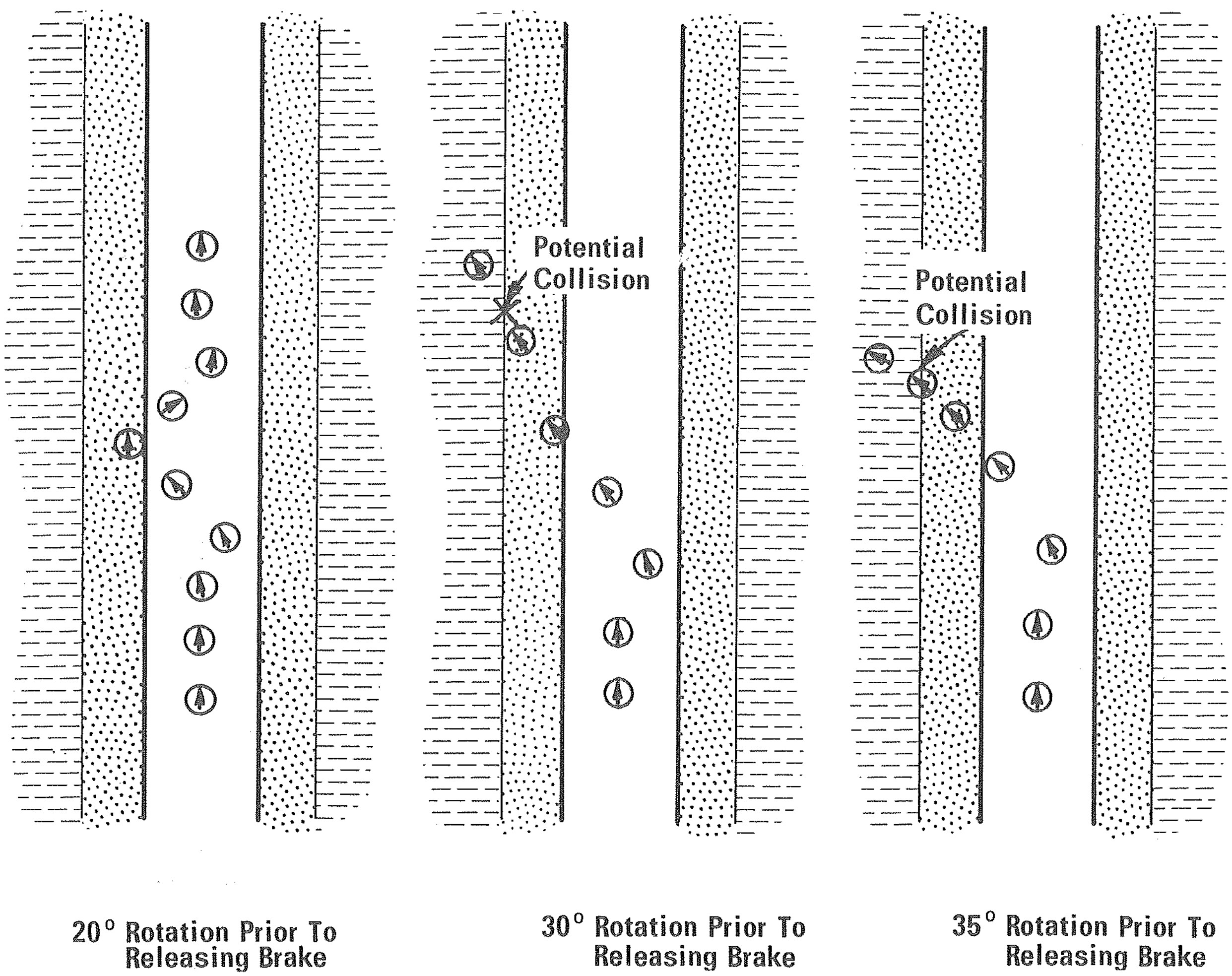
that the only case in which the vehicle did not move into the collision zone was for the 20^{o} rotation test. In this case it moved only into the possible accident zone. The car was basically uncontrollable after rotating thirty degrees or more. When an obstacle was placed in the road, the driver did avoid it at 40 MPH; however, to do this, his car moved into the collision zone and spun out of control. When the obstacle was removed and the driver given unlimited stopping distance, he was still unable to prevent the car from spinning out of control and entering the collision zone.

Figure 11 shows the results of the 50 MPH tests. In this case the driver was unable to control the vehicle after ten degrees rotation and moved into the collision zone. When an obstacle was placed in the road, the vehicle not only moved into the collision zone, but also spun into the obstacle.

From the results of these speed tests, it is obvious that differential friction can significantly effect the control of a braking vehicle and produce a potentially hazardous condition which the driver may not be able to correct.

The greatest problem arises when the driver releases the brake after the car has begun to spin. When this is done, the vehicle is propelled in the direction it is facing, whether it be off the road or into oncoming traffic. The greater the rotation the more uncontrollable the vehicle. After passing ninety degrees, the car will actually be propelled rearward if the brake is released.

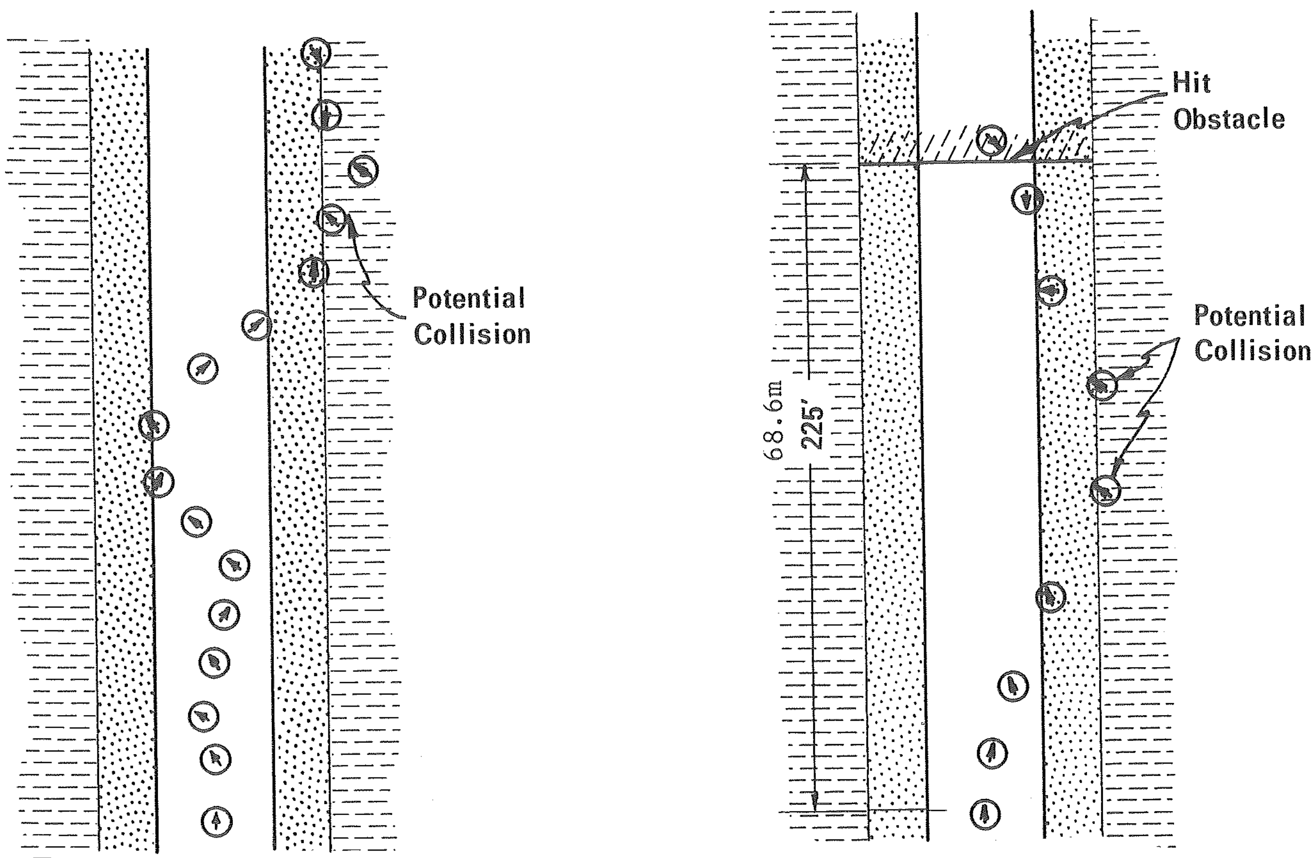
VEHICLE CONTROL TESTS ON A DIFFERENTIAL FRICTION SURFACE (DF_{40}=17



40 MPH TESTS

Figure 10

VEHICLE CONTROL TESTS ON A DIFFERENTIAL FRICTION SURFACE (DF_{40}=17)



10° Rotation Prior To Releasing Brake

Unlocked Brake Test With Obstacle At End
268 Feet To Stop

50 MPH TESTS

Figure 11

If the driver keeps his brakes locked, the car will slide straight ahead and spin about its center of gravity or front wheels. Unfortunately, the average driver is not conditioned to do this. It is the tendency of the average driver to be confused by the rotation and possibly release his brakes after spinning approximately 30 degrees. If the vehicle is still moving, this is an extremely dangerous thing to do.

From our tests, we have generally concluded that if the differential friction causes a vehicle to rotate more than approximately twenty-five degrees while it is still sliding at a speed greater than fifteen miles per hour, the driver may not be able to prevent his vehicle from entering the collision zone if his brakes are released. As there are numerous combinations of speeds and differential friction levels that will produce this condition, it is evident that differential friction can indeed be a potential hazard to the driving public. It is estimated that when braking, a major loss of control may occur if the differential friction surface produces total rotations greater than those listed in the following table.

Speed at which wheels are locked	Total rotation after car has stopped
30 MPH	30^{o}
40 MPH	50^{o}
50 MPH	70^{o}

As the total rotation increases above these values, the potential loss of control is drastically increased.

These tests were performed under theoretically controlled conditions in which the driver was familar with the surface. Even under these conditions, the tests were hazardous at speeds in excess of 30 MPH, and thus only one site was thoroughly investigated. Spot testing at other locations confirmed that this site was representative of the results that could be expected at other sites. It is unfortunate that at some locations the driving public may be faced with such conditions at speeds of 55 MPH or more.

There is a strong indication that differential friction may be as important in the causation of wet pavement accidents as low friction level. If this is the case, a major re-evaluation of present pavement friction evaluation, design and corrective techniques may be necessary.

Dry skid tests were also performed at two locations to determine the magnitude of this problem under such conditions. The results indicated that under normal conditions on dry surfaces, with no loose material and a non-bleeding surface, that the rotation was not significant. This should be expected, as most surfaces produce similar dry coefficients of friction and thus there is no real differential friction created.

In the case of a partially bleeding surface, or a loose aggregate chip seal, the results can be quite different. Under such conditions, the bleeding surface may actually melt, or the loose chips rotate and act as ball bearings, both of which will produce a lower dry surface friction. Under these conditions, differential friction is indeed a problem and could possibly cause a vehicle to spin violently out of control.

CAUSES OF DIFFERENTIAL FRICTION

There are numerous causes of differential friction. Some are created or induced by construction practices while, others are caused by maintenance techniques. Most are initiated or compounded by exposure to traffic.

It should be remembered that friction is a force generated at the tire-pavement interface. For this reason the pavement, the tires, and the vehicle dynamics greatly effect the coefficient of friction. Thus, the vehicle and tires may cause a differential friction, even though the pavement has a uniform coefficient of friction. For example, a vehicle that has bald tires or defective brakes on one side of the car, and has good tires or good brakes on the other, could experience differential friction when braking. This would be in no way related to the pavement surface. Thus, it is important to consider possible vehicle interaction when evaluating differential friction.

As this report is primarily interested in the effects of highway surfaces, only the differential friction caused by the pavement will be considered here.

The following is a list of the most commonly found differential wheel-path friction conditions.

1. Differential Flushing or Bleeding - (Figure 12)
   This condition is created if a portion of the lane is flushing while another is not. Such a condition can also occur if full lane repairs are not made or when

the contact of each ribbon falls inside the lane instead of at the edge. Maintenance operations can be one of the main contributors to such a problem. For example, if the chips do not adhere, or a slurry seal bleeds, in one ribbon and not the other, an extremely hazardous condition can exist.

2. Unequal Wear or Flushing - (Figure 13)

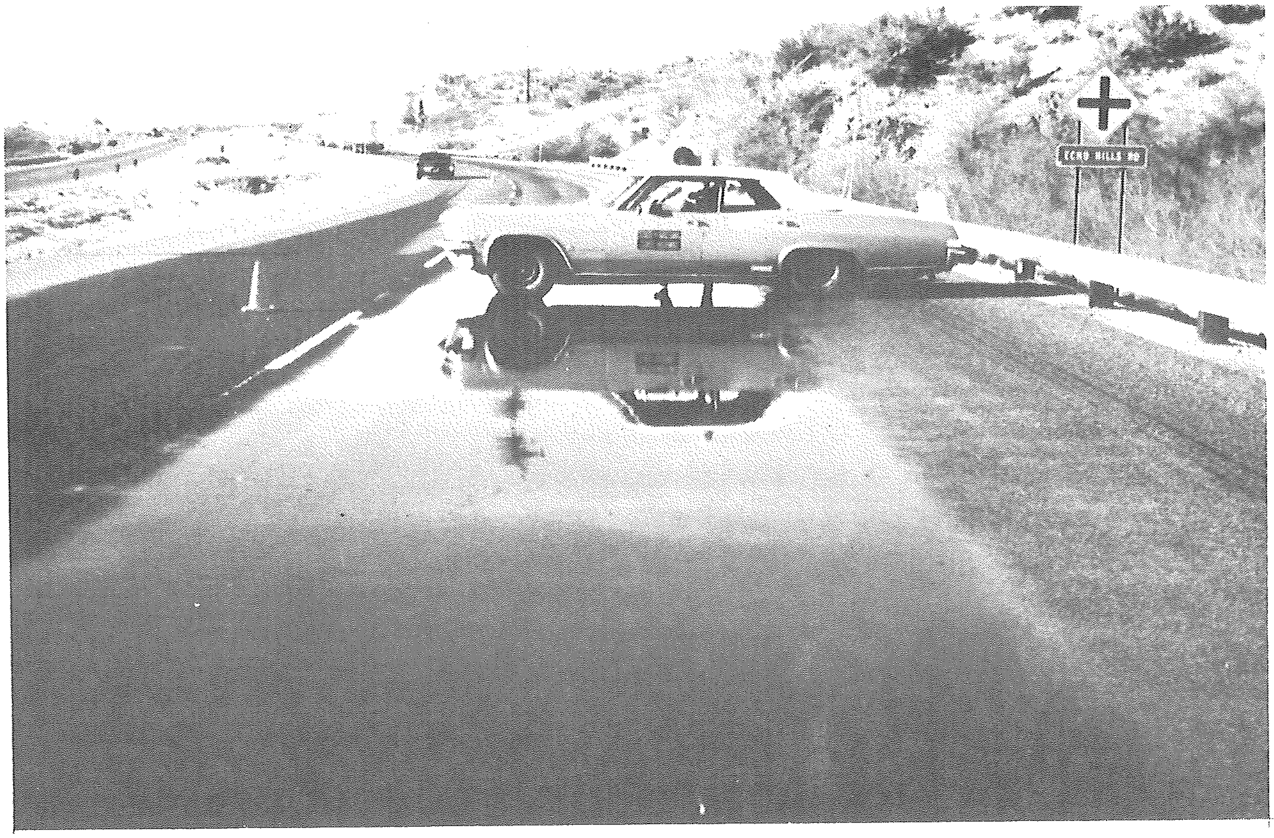
   This condition, as the previous one, may be caused when the contact of two asphalt ribbons fall inside one travel lane. Unlike the above, however, the main contributor is traffic wear. If the two ribbons are not alike, or if they polish at different rates, vehicles riding in the lane will experience differential friction. An unequal transverse distribution of traffic in very wide lanes can also cause this problem, as truck and passenger car traffic may ride in different portions of the lane causing differential wear.

3. Chip or Slurry Seals Occupying Only a Portion of the Lane

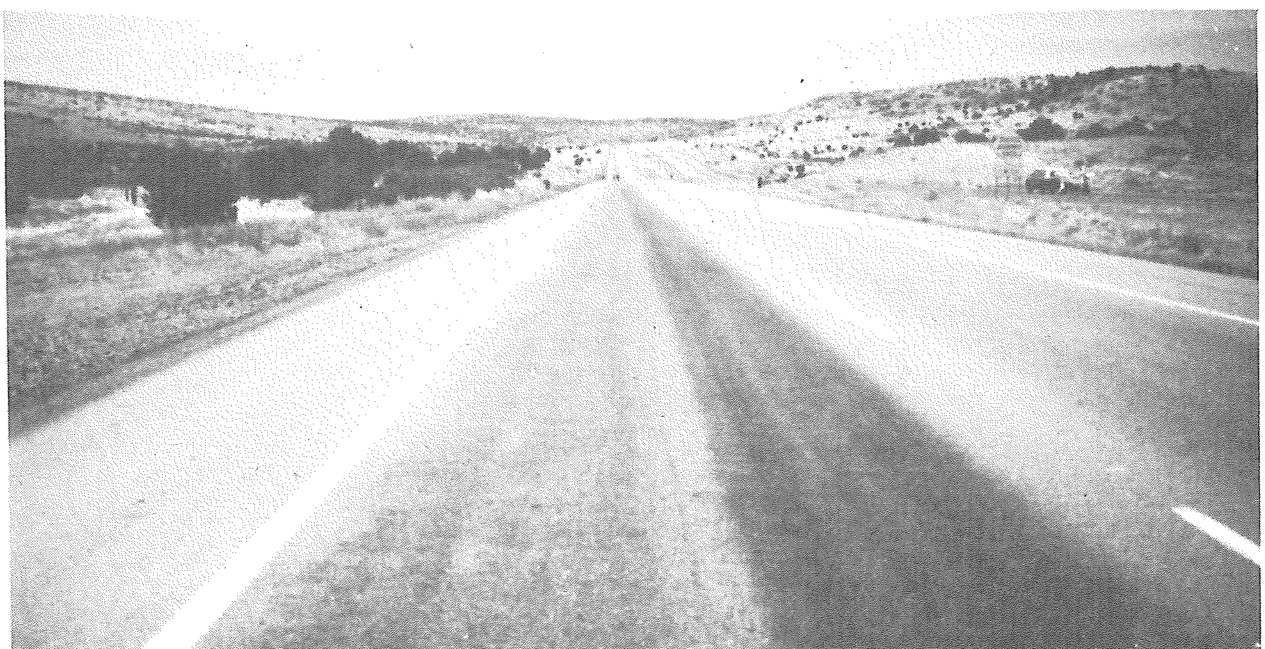
   When a chip or slurry seal is placed across only a portion of the lane width (Figure 14), a major differential friction condition may exist. This is done in some cases where only half the lane has a bleeding or cracking problem. If only a portion of the lane is bleeding, it may be selectively treated in an attempt to correct a differential friction problem. This does not correct the situation. On the contrary, it creates a differential condition by having a higher friction on the treated side and a lower friction on the untreated side. This is just the reverse of what may have existed before. If the treated side bleeds, it will again create a differential friction which may be of a greater magnitude than existed prior to the corrective action. Maintenance operations can contribute to this problem when they partially treat a lane attempting to conserve on available funds.

4. Dissimilar Surface Type for Shoulder or Distress Lane

When a chip seal is used as the riding surface for the shoulder or distress lane, while the travel lane has a dissimilar surface such as Asphaltic Concrete (Figure 15), a differential friction condition is created when a vehicle rides on both surfaces simulataneously and attempts to stop. This condition can also exist when a concrete pavement has an asphaltic concrete shoulder (Figure 16). A preliminary report[4] has shown that in a major study of lane placement of vehicles, that sixty five percent of truck traffic rides partially on the distress lane. It was also established that of those trucks encroaching

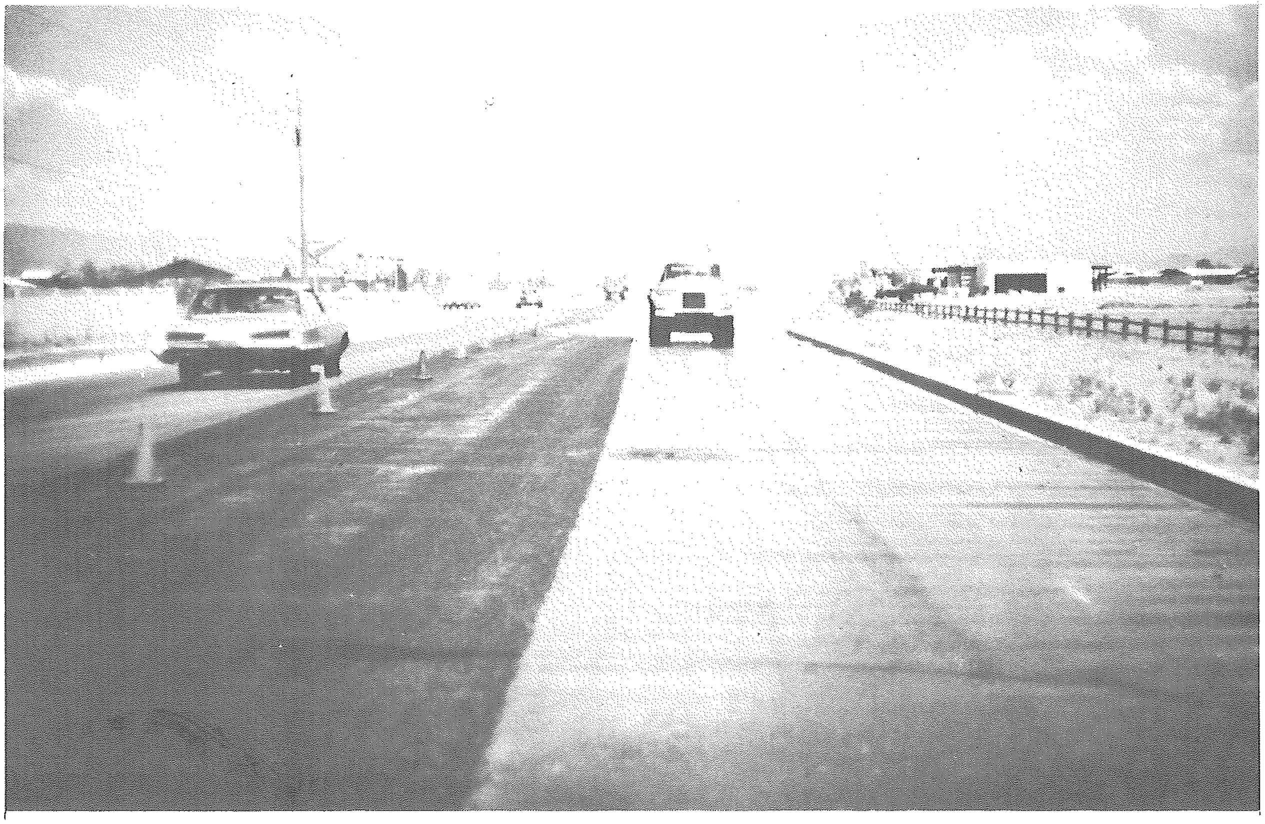


DIFFERENTIAL FLUSHING OR BLEEDING
Figure 12



UNEQUAL WEAR OR FLUSHING

Figure 13



CHIP OR SLURRY SEAL OCCUPYING ONLY A PORTION OF A LANE

Figure 14

DISSIMILAR SURFACE TYPE FOR SHOULDER OR DISTRESS LANE

Figure 15

upon the distress lane, that 53% of the encroachments occurred in straight sections with no other vehicles present.

In the past, the reason for using different shoulder material was a visual demarcation as well as a rumble strip. In the referenced study, it was shown that there was an eighty percent increase in encroachment when a bituminous surface treatment shoulder was used as compared to the AC shoulder. Obviously, this strengthens the conclusions that rough textured shoulders have not generally discouraged encroachment. It has also been noted that small, uniformly spaced grooves in the pavement will produce a much rougher ride with an audible high pitched sound. In the author's opinion, small grooves are much more successful and do not create a differential friction. Due to the amount of traffic riding on such a differential friction surface, major consideration should be given in future designs to using similar material for both the distress lanes and the travel lanes.

5. Maintenance Crack Patching

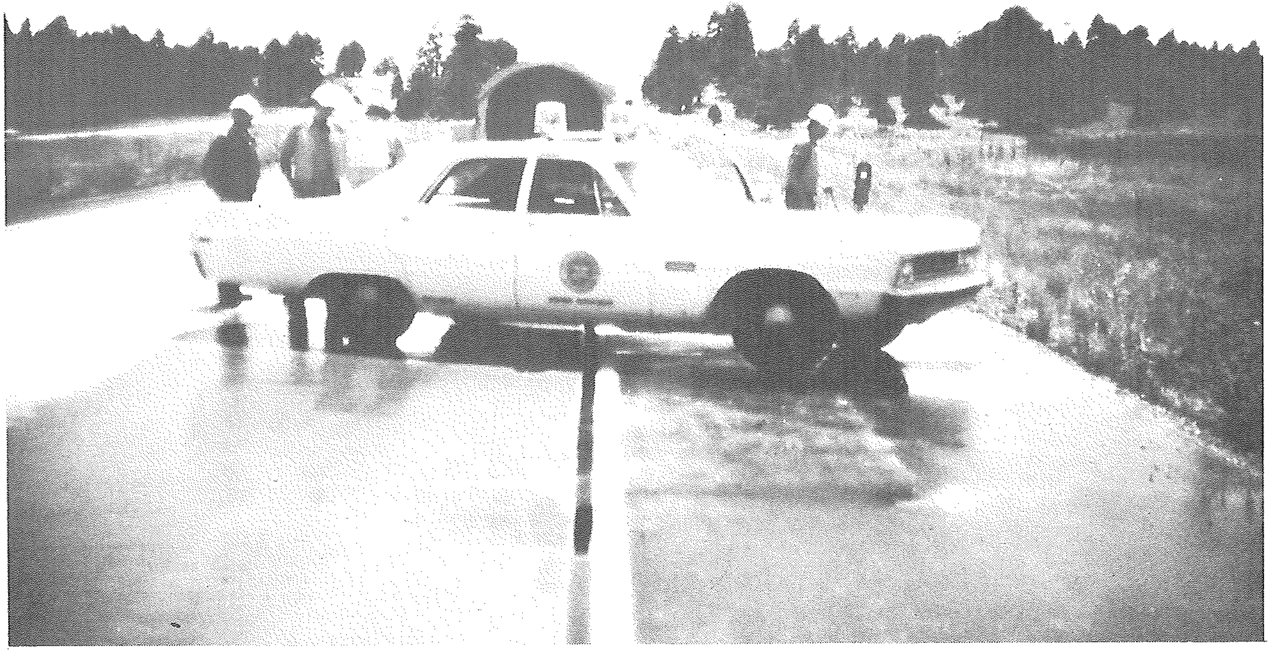
This can be a major problem when the rate of crack patching in one wheeltrack is much greater than the other wheeltrack (Figure 17). If excessive amounts of asphalt are used, the material may be tracked for many thousands of feet producing a very lengthy problem which will be difficult to correct. If sand is used in one wheeltrack to correct this situation it only reverses the problem. Once this problem exists, it is almost impossible to cure without a major corrective action.

6. Unequal Drainage Properties

This problem occurs when the pavement's surface drainage capabilities are different (Figure 18), as in the case of an open and dense graded AC, or a chip seal. This problem may occur when the designs are different, or when asphalt content is changed during construction. It may also occur when a lane is widened and a different mix design or aggregate is used.

7. Unequal Water Layer Thickness

This differential friction condition can cause a major problem to the driving public. It is usually created by low spots or geometric problems which cause puddling or variation in water layer thickness. As one side of a car may be hydroplaning, an extreme differential may be created even though there is uniformity in the surface type. This condition can also occur if only a portion of the pavement is wet, such as might be created by water blowing across the road from roadside sprinklers.



PCCP TRAVEL LANE WITH AC DISTRESS LANE

Figure 16

MAINTENANCE CRACK PATCHING

Figure 17

UNEQUAL DRAINAGE PROPERTIES

Figure 18

There are numerous other situations which can yield major differences in the wheelpath friction levels. Most of the causes of differential friction need not occur and can be avoided if proper consideration is given to the phenomena.

PREVENTION OF DIFFERENTIAL FRICTION

Differential friction problems can be avoided if the phenomena is considered during construction and maintenance operations. During construction, the ribbons should be placed so that all longitudinal joints fall on the outside of each lane at, or very near, the location of the lane stripes. Present construction methods allow this, and it should easily be achieved at no extra expense if it is stipulated in the specifications.

During construction, it would be desirable that the distress lane be constructed of the same surface type as the travel lanes. This is because there is a tendency for traffic at times to ride with one wheel in the distress lane. The use of a demarcation chip seal in combination with an ACFC travel lane may cause problems when the pavement is wet. The same problem may exist when using asphaltic concrete shoulders with concrete travel lanes.

When the same surface type is used for the travel and distress lanes, a well maintained stripe at the edge of the travel lane is needed. This is necessary to provide visual guidance for the driving public.

During maintenance operations, it is imperative that most operations be uniform across the full lane width. The placement of any corrective action for only a portion of the lane should be avoided when possible. Our studies have shown that maintenance operations can be the major contributor to a differential friction condition. It is ironic that in attempting to correct one problem, that another may be created. In some cases, the maintenance force may actually be attempting to correct a differential friction problem and cause a similar condition. For example, when one wheeltrack is bleeding and the other is not, the maintenance force may place a slurry or chip seal on the bleeding wheelpath. This does not correct the problem, as the treated wheelpath now has a higher friction level than the untreated one, thus causing the reverse problem.

The problem is only corrected when both wheelpaths have similar coefficients of friction. This is not difficult to achieve if the underlying variables are understood.

SUMMARY

The problems associated with differential friction can range from minor to extremely serious, depending on the differential friction number and average coefficient of friction.

Differential friction can cause an extremely hazardous condition for a braking vehicle. As the problem can occur at high, as well as low, friction levels, it should be given major consideration in any pavement friction evaluation.

There is a strong indication that differential friction may be as important in the causation of wet pavement accidents as low friction. If this is the case, a major re-evaluation of present pavement friction evaluation, design and corrective techniques may be necessary.

For any particular combination of differential friction number and speed, the number of degrees of rotation can be calculated by equations presented here. Thus, the magnitude of the problem at various locations can be predicted via normal skid testing techniques, without the hazards of actual vehicle rotation tests.

When riding on a differential friction surface, the greatest problem arises when the driver releases the brakes after the car has begun to spin. When this is done, the vehicle is propelled in the direction it is facing. This could be off the road or into oncoming traffic. The greater the degree of rotation, the more uncontrollable the vehicle.

Surface friction inventories made in only one wheeltrack may not detect this problem unless visual observations are recorded. Although one or both wheelpaths may have a high friction level, if they are unequal, it may cause a hazardous condition for a braking vehicle. Observations by trained technicians can usually detect this problem and indicate it's severity. Observations of large variations in the actual recorded friction trace may also indicate this condition.

There are numerous causes of differential friction, most of which can be avoided during construction, design, and maintenance operations. As maintenance operations may be the largest contributor to this problem, it is hoped that in the future the maintenance engineer, as well as other highway engineers, will give more consideration to this problem and its correction. This report outlines some of the major causes and corrective techniques that need to be considered with this condition.

PART TWO

TEXTURING OF PORTLAND CEMENT CONCRETE PAVEMENTS

INTRODUCTION

The second phase of this program was designed to evaluate the frictional effects of various texturing techniques on portland cement concrete pavements. The analysis was performed on seven basic types of texturing applied in either a longitudinal or transverse method. This was done to evaluate the frictional performance of both directions and also evaluate the problems encountered with either direction. This part of our research study was performed under the FHWA Experimental Projects Program and will be a continuing study.

PROJECT INVESTIGATED

Project I 17-2 (35), Cordes Junction - Flagstaff (Munds Park - Flagstaff Airport), was selected for the experiment. The location is approximately fourteen miles (22.5km) south of Flagstaff, Arizona at an elevation of approximately 6700 feet (2042.16m). The project had about 200,000 square yards (167,220 sq. m) of portland cement concrete pavement, 8" (20.32cm) thick, which was placed by a CMI slip form paver. The concrete was placed at a width of 24 feet (7.3m), and approximately 5,000 lineal feet (1524m) of pavement was placed in one day.

SPECIFICATIONS FOR PORTLAND CEMENT CONCRETE PAVEMENT

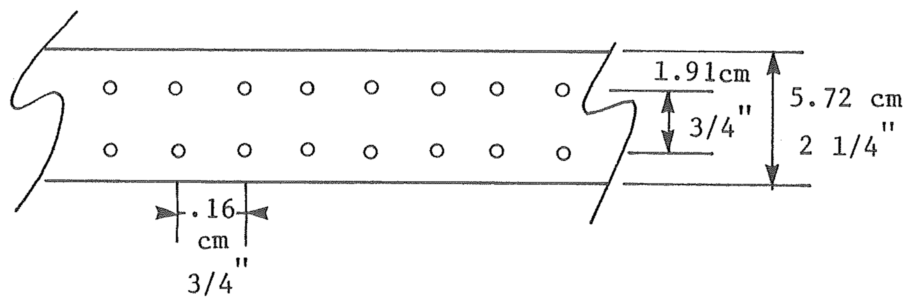
The specifications given in Appendix B are those used on this project. The concrete was designed with a cement factor of six sacks per cubic yard, with an air content requirement of 4 - 7% and a slump of 1 - 3 inches (2.54 - 7.62 cm). The temperature of concrete was not to exceed $90^{\circ}F$; ($32.2^{\circ}C$) and the air temperature was to be not less than $40^{\circ}F$ ($4.4^{\circ}C$). Concrete operations were not to be started unless the ascending air temperature was at least $35^{\circ}F$ ($1.7^{\circ}C$).

TEXTURE TYPES

Twelve texture sections were examined during this research project. They were as follows:

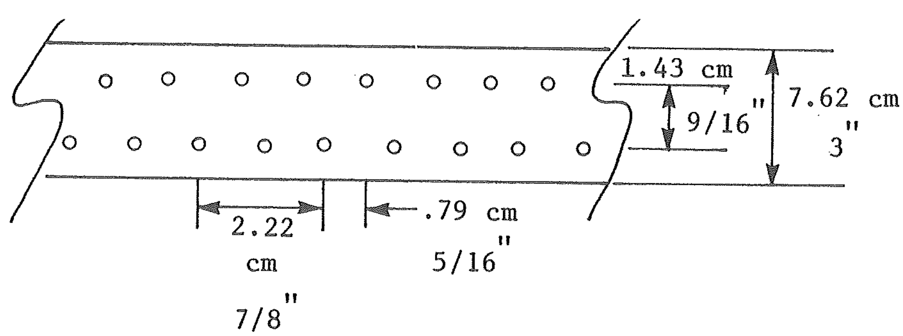
1. Burlap Dragging (with at least three feet in contact with surface).
   a. Normal burlap drag
   b. 4-Ply, 10 oz. (283.5g) burlap
   c. Burlap drag with 16 penny nails inserted in trailing edge - approximately four nails to the inch.
   d. 4-Ply, 10 oz. (283.5g) burlap with from six to twelve inches (30.5 cm) of the transverse threads from the trailing edge removed.
2. Nylon Bristle Broom

   The nylon broom consisted of a 48 inch (121.92 cm) long by 2-1/4 inch (5.72 cm) wide by 2 inch (5.08 cm) deep wooden block with six inch long, 25 - bristle tufts in two rows on 3.4 inch (1.91 cm) centers. Each bristle was 1/16 inch (0.16 cm) in diameter.



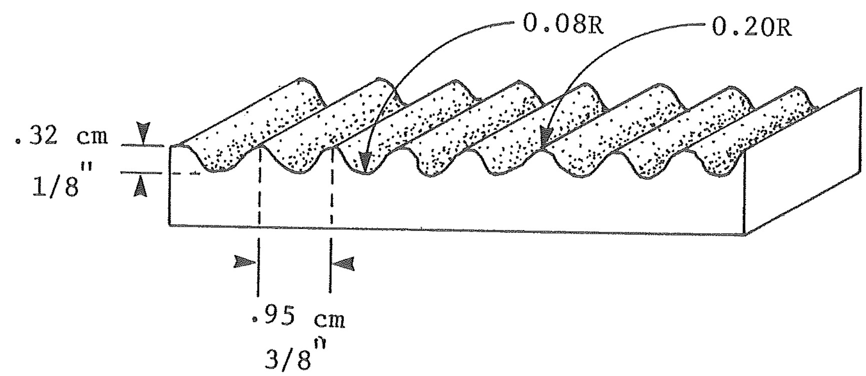
3. <u>Wire Broom</u>

A typical wire-bristle broom has a 4.5 lb (2.04 kg) broom head consisting of a 28 inch long by 3 inch (7.62 cm) wide by 2 inch (5.08 cm) deep wooden block with six inch (15.24 cm) long, 10-bristle tufts in two rows on 7/8 inch (2.22 cm) centers; individual bristles consist of 1/16 inch (.16 cm) by 1/100 inch (.03 cm) steel.



4. Fluted Float (Magnesium)

The fluted float has a 10.4 lb (4.72 kg) float head 40 inches (101.6 cm) long and 7 1/2 inches (19 cm) wide which is fluted to produce a normal pattern. This texture was previously described in a New York Department of Transportation report[5].



All texturing was done mechanically and is described in Figure 19. This figure notes both the texture type, direction, location and section number.

TEXTURE EVALUATION

The evaluation of the various texturing techniques was based on texture depth, coefficient of friction, ease and uniformity of application, vehicle-pavement noise levels and texture durability. The results of these studies are described in the following section.

FIGURE 19

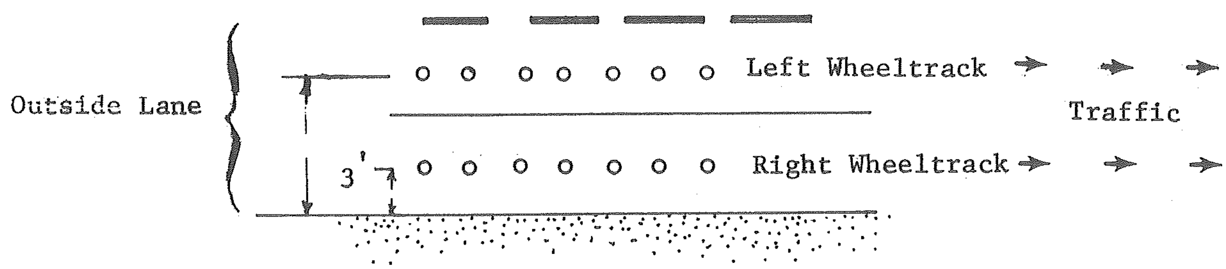
EXPERIMENTAL TEXTURE SECTIONS-MUNDS PARK

Section. Number	Texture Type	Texture Direction	Station Number	Section Length (Ft)
11	Nylon Bristle Broom	Longitudinal	7064+00 7069+00	500
12	Nylon Bristle Broom	Longitudinal	6745+00 6750+00	500
1	Normal Burlap Drag	Longitudinal	6739+00 6735+00	400
2	4-Ply,10 oz.,Burlap Drag	Longitudinal	6735+00 6730+00	500
3	Nylon Bristle Broom	Transverse	6683+40 6678+40	500
4	Wire Broom	Transverse	6675+25 6670+25	500
5	Burlap Drag With Nails	Longitudinal	6661+70 6656+70	500
6	Burlap Drag With Nails	Transverse	6643+52 6642+50	102
7	Fluted Float	Longitudinal	6604+00 6599+00	500
8	Fluted Float	Transverse	6594+00 6592+00	200
9	Wire Broom	Longitudinal	6552+75 6547+75	500
10	4-Ply,10 oz., Burlap Drag Threads Removed	Longitudinal	6541+50 6536+50	500

TEXTURE DEPTH

Texture depth was measured by sand patch test method[6]. Appendix "B" shows pictures of the finished textures as well as their typical cross sections and texture depths.

Sand patch tests were performed at six equally spaced locations in each of the two wheeltracks of the outside lane in all but two test sections. An example of this is shown below.



Five tests were performed at each location for a total of thirty tests per wheeltrack, or sixty tests per section. The results of these tests are summarized in Figure 20.

From these results, it can be seen that initially the nylon bristle broom yielded the deepest and most uniform texture for all types tested. This was followed by the wire broom which produced a harsh texture but which was very variable and produced latence which would wear quickly. Following these, the burlap drag and fluted float yielded approximately equal texture

FIGURE 20

TEXTURE INFORMATION

			Three (3) Feet From Right Edge				Nine (9) Freet From Right Edge			
Section Number	Section Length (ft.)	Texture Type	Number[1] of Tests	Average Texture Depth[2] (in)	Average Texture Depth (cm)	Standard Deviation (in)	Number[1] of Tests	Average Texture Depth[2] (in)	Average Texture Depth (cm)	Standard Deviation (in)
1	400	Normal Burlap Drag (Longitudinal)	6	.051	.130	.007	6	.037	.094	.002
2	500	Burlap Drag (4 ply, 10 oz)(Longitudinal)	6	.036	.091	.009	6	.028	.071	.006
3	500	Nylon Bristle Broom (Transverse)	6	.060	.152	.003	6	.063	.160	.011
4	500	Wire Broom (Transverse)	6	.075	.190	.022	6	.073	.185	.023
5	500	Burlap Drag with Nails (Longitudinal)	6	.036	.091	.008	6	.045	.114	.004
6	103	Burlap Drag with Nails (Transverse)	2	.056	.142	.008	2	.052	.132	.010
7	400	Fluted Float (Longitudinal)	6	.041	.104	.002	6	.039	.099	.002
8	200	Fluted Float (Transverse)	3	.039	.099	.009	3	.036	.091	.004
9	500	Wire Broom (Longitudinal)	6	.078	.198	.009	6	.069	.175	.024
10	500	Burlap Drag (4 ply, 10 oz. Longitudinal) Threads Removed	6	.041	.104	.003	6	.035	.089	.004
11	500	Nylon Bristle Broom (Longitudinal)	6	.096	.244	.010	6	.079	.200	.018
12	500	Nylon Bristle Broom (Longitudinal)	6	.085	.216	.019	6	.085	.216	.019

1. Each Test Represents the Average of Five Readings.

2. Average Texture Depth, as Measurbed by the Sand Patch Method.

depths which were signifcantly less than those achieved by the broom textures. Of these lesser textures, the longitudinal fluted float produced the most uniform surface, while the burlap drag produced irregular results ranging from fair to poor. The transverse burlap drag with nails produced a good uniform surface, however, this method was very sensitive to time of texturing. This can be seen by the results of the same system used longitudinally.

Texture tests will be made at one year intervals to monitor the durability of these textures. As this section of highway is exposed to snow plow blades and other snow removal equipment, the effects of wear will be accelerated. After only one winters exposure, visual observations have noted that several of the transverse textures have been almost completely worn away, while the similar longitudinal texture is only slightly effected.

Texture depths created in this study were generally greater than usually experienced in the field due to the plastic nature of the portland cement concrete.

EASE OF APPLICATION

As ease of application and uniformity of various texture applications are extremely important in the construction phase of any project, notes on these variables were made from visual observations during the texturing phase of this program. A brief summary of the results is as follows:

1. Burlap Drag

Although easy to apply, poor and non-uniform textures were derived. Time of texturing is critical. Of the various types of burlap drag textures used, the burlap drag with nails produced the best and most uniform texture.

2. <u>Wire Broom</u>

   Although good and uniform textures were easily obtained, this method tended to be destructive to the surface of the PCCP.

3. <u>Fluted Float</u>

   This texture definitely has merit. Success was achieved with simple homemade equipment. The longitudinal pattern is favored, and it is believed that if equipment were built for this use, excellent results could be obtained.

4. <u>Nylon Bristle Broom</u>

   Excellent textures were obtained for both the longitudinal and transverse drag.

   This finish is far superior to the other methods, as a uniform high friction surface can be obtained easily and quickly. The time of finishing is not as critical with this method. The longitudinal pattern is favored. A mechanical drag system should be used, as this system is not as successful if texturing is produced by hand.

FRICTION ANALYSIS

The wet pavement friction level was recorded by the Mu-Meter. Tests were made in the left wheeltrack of the outside lane at 40 MPH. The results of these tests are shown in Figure 21. From these tests, it can be seen that the surfaces with the highest friction level, after exposure to approximately 700,000 vehicles, are those with initially the greatest longitudinal texture.

All of the surfaces had an initially high friction level. After seven months of exposure to approximately 700,000 vehicles, there was a noticeable difference in the friction levels recorded. The latest tests show that the longitudinal nylon bristle broom has the highest Mu-Meter Number (MuN) which is 78. This is followed closely by the wire broom textures and the transverse nylon bristle broom which have a MuN of between 75 and 76. The longitudinal fluted float (MuN = 71), longitudinal burlap drag with nails (MuN = 70), and the transverse fluted float (MuN = 68), all had similar friction levels and where the next highest group. All the remaining burlap drag textures had Mu-Meter Numbers of between 59 and 63. These represented the lowest group of friction levels.

Figure 21 also shows that the nylon bristle broom textures had approximately a 16% reduction in their friction level between tests. In comparison, the wire broom showed a 20% reduction; the fluted float and burlap drag with nails a 24% reduction; and the other four burlap drag textures a 31% reduction. Thus, the surfaces with the highest friction level also had the lowest percent decrease in friction level.

I-17 CONCRETE TEXTURE SECTIONS

FIGURE 21

Mu-Meter$_{40}$ Readings - Right Lane, Left Wheeltrack

Texture Section Number	Type	Direction[1]	Date Tested 11-7-74	6-17-75[2]	Value Decrease	Percent Decrease
1	Normal Burlap Drag	L	88	61	27	31
2	4-Ply Burlap Drag	L	86	60	26	30
3	Nylon Bristle Broom	T	94	75	19	20
4	Wire Broom	T	93	76	17	18
5	Burlap Drag with Nails	L	91	70	21	23
6	Burlap Drag with Nails	T	87	59	28	32
7	Fluted Float	L	94	71	23	24
8	Fluted Float	T	90	68	22	24
9	Wire Broom	L	96	75	21	22
10	Burlap Drag Threads Removed	L	90	63	27	30
11	Nylon Bristle Broom	L	93	77	16	17
12	Nylon Bristle Broom	L	91	78	13	14

1. L = Longitudinal

   T = Transverse

2. Approximately 700,000 vehicles during 7 months of exposure.

FIGURE 22

VEHICLE NOISE VERSUS PAVEMENT TEXTURE

Test Section	1	2	3	4	5	6	7	8	9	10	12
Overall Average	65.5	65.4	69.2	70.2	66.6	67.7	68.8	67.3	67.5	65.4	65.7
Average for All Vehicles:											
At 70 MPH	72.0	71.7	75.7	76.3	73.3	74.4	76.0	73.7	73.9	71.3	72.3
At 50 MPH	66.2	66.3	69.6	70.9	67.0	68.0	69.2	67.8	68.3	65.6	66.2
At 30 MPH	58.4	58.2	62.4	63.5	59.6	60.8	61.2	60.3	60.2	59.2	58.6
At 50 & At 70 MPH	69.1	69.0	72.7	73.6	70.2	71.2	72.6	70.8	71.1	68.5	69.3
Average at All Speeds:											
For Auto	62.2	62.2	67.7	70.3	63.8	65.2	66.6	65.3	64.8	62.1	62.5
For Truck (w/o Load)	67.7	67.3	70.2	71.0	68.1	68.9	69.9	68.1	69.2	67.1	67.2
For Truck (with Load)	66.8	66.6	69.8	71.4	68.0	69.1	69.8	68.4	68.3	66.8	67.3

All values are in decibels (A-weighted) and are peak values. Original noise levels were read to nearest decibel only. All decimal values are therefore only the product of mathematical averaging and are not indicative of any capability to read individual peak noise levels with decimal precision. Refer to figure 20 for test section description. All measurements were made at a distance of 50 feet perpendicular to the highway.

When comparing the transverse and longitudinal textures for any particular surface type, it can be seen that in general the longitudinal textures had the higher friction levels and the lowest percentage decrease. It is expected that this trend will be magnified in the future. The reason for this is that the test area receives heavy snow plow and ice removal operations. Due to this, the transverse textures are being worn away at a rapid rate in comparison to the longitudinal textures which yield little resistance to the snow plow blade. From the rapid rate of wear on the transverse texture seen after only one winters wear, it would appear that transverse textures may be undesirable in areas where snow and ice removal operations are anticipated.

NOISE LEVEL

For automobiles and light duty-trucks, the primary sources of noise are the engine, the exhaust, tire noise, and wind resistance. For well-tuned and properly muffled vehicles, tire noise and wind resistance tend to be the dominant noise sources at highway speeds. During the noise-pavement texture tests conducted jointly by personnel of the Environmental Planning and Materials Division on the southbound lanes of Interstate Highway 17 south of Flagstaff on July 9-10, 1974, pavement texture was essentially the only variable affecting noise emission. The other factors were held constant through continued use of the same vehicles at specified operating speeds. The vehicles used were a 1970 one ton Chevrolet Pickup truck and a 1973 AMC Ambassador passenger car. Sound measurements were made at a distance of fifty feet, perpendicular to the roadway.

The results of the tests are shown in Figure 22. These results indicate that Sections #2 and #10 (longitudinally dragged with 4-ply, 10-ounce burlap), were the quietest. It is significant to note that Section #12 (longitudinal nylon bristle broom), which represented the typical texture on the overall project, was one of the next quietest sections and only 0.3 decibels greater than Sections Number 10 and 2.

Pavement Sections #4 and #3 (brushed transversely with wire broom and nylon bristle respectively), were the noisiest. Section #4 was somewhat noisier than Section #3 and approximately five decibels noisier than Section #10. A difference of five decibels is significant and distinctly audible. Transversely finished pavement is normally expected to be noisier than longitudinally finished pavement. The transverse ridges combine with the longitudinal grooves on most tire treads to effectively create airtight pockets. In these pockets air is compressed and then released with explosive force as the tire rolls on. In contrast, longitudinal pavement finishes combine with normal tire tread patterns to permit more gradual release of trapped air. This may be seen in comparing sections finished similarly but along different axes. Section #6 (transverse) was one decibel noiser than the similarly finished Section #5 (longitudinal). Also, Section #4 (transverse) was more than two decibels noisier than the similarly finished Section #9 (longitudinal). The greatest difference was seen between Section #3 (transverse) which was almost four decibels higher than Section #12 (longitudinal).

TEXTURE MOLDS

Silicon rubber molds were made on each texture type. These molds are flexible and produce a three dimensional durable, representation of the exact texture that was present when tested. Such molds are ideal for use on new construction, where a sample is needed to show the contractor and inspector the desired texture. Although specifications can be written concerning texture depth and friction values, an actual sample of the desired texture is extremely useful. With such molds, both contractors and inspectors will see that the pavement is properly textured. Without such samples, the Design Engineer can not be sure of the final results.

The system used in taking samples, and the one commonly used in Arizona, is as follows:

1. Select the texture to be duplicated.
2. Clean the surface and spray lightly with a plastic sealer.
3. Place a 5 inch, open, square mold frame on surface and spray inside of frame and pavement with silicon mold release.
4. Pour plaster of paris into the frame to a thickness of 1 1/2 inches and let dry.
5. When hardened, gently remove plaster of paris. This now represents a negative of the actual surface texture.
6. Return to Lab and frame plaster mold, then lightly spray with plastic sealer. When dry, spray surface and inside of frame with silicon mold release.

7. Produce "positive" molds by mixing General Electric RT V-41 Silicon Rubber and pouring it into mold. A silicon rubber mold 3/4" thick is very durable and can be kept in the vest pocket during inspections. Any number of "positive" silicon rubber molds can be made from the original plaster molds.

TEXTURE DURABILITY

In Appendix C, pictures of each of the pavement texture types taken on November 7, 1974 and June 17, 1975 are shown. The pavement was exposed to approximately 700,000 vehicles during this period. The pictures indicate that the wear in texture is comparable to that of the surface friction loss previously described.

Some test sections have been almost completely worn away by the effects of traffic and snow removal operations. In all cases, the longitudinal textures are much more durable than the transverse textures. Texture loss ranges from almost 100% for the burlap drag to very slight for the nylon bristle broom. Sand patch tests will be performed after one years exposure to traffic.

FUTURE EVALUATION

Similar tests to those previously mentioned will be made at one year intervals as a continuing study to evaluate the effects of traffic wear.

SUMMARY

The initial results of this texturing study have shown that for the textures evaluated, that the longitudinal nylon bristle broom produced the most skid resistant, most uniform, most easily achievable, and most durable texture.

All the burlap drag textures produced poor, non-uniform textures that were heavily dependent on time of texturing and operator techniques. Of all the burlap drag methods tested, the one with nails produced the best surface.

The wire broom, although producing a deep texture, was non-uniform and may adversely effect the surface.

The fluted float produced an unusual surface that is very uniform and definitely has merit. If equipment was produced to apply such a texture, it is believed that excellent results could be obtained.

The transverse textures in this experiment was less durable than the longitudinal textures. This is because the area receives heavy snow plow and ice removal operations. From this study, it appears that transverse textures may be undesirable in areas where snow and ice removal operations are anticipated.

PART THREE

FRICTIONAL EFFECTS OF GROOVING

INTRODUCTION

In Part Two we have seen the effects of texture and pavement wear on the friction level of portland cement concrete pavement (PCCP). When a PCCP pavement is not textured sufficiently or is polished by traffic, it may not have a sufficient coefficient of friction to meet the needs of the driving public under wet pavement conditions. When PCCP pavements reach this low friction level, they are frequently grooved by diamond impreganted saws. This method has become an accepted method. It has been well documented in significantly reducing wet pavement accidents in numerous states.

Although the reduction of wet pavement accidents by longitudinal grooving has been definitely proven[7], tests using the ASTM locked wheel skid trailer show no change in the friction level between the before and after grooving tests. In fact, they sometimes record a slight reduction in the friction level. The reason for this appears to be that longitudinal grooving does not increase the coefficient of friction in the longitudinal direction. It does, however, allow new drainage channels for water to escape and thus reduce the chance of hydroplaning. In addition to this, it significantly increases the cornering or side force friction available to traffic. This is why it reduces the number of wet pavement accidents occuring on curves and in areas of cornering maneuvers. These two locations account for the majority of wet pavement accidents on interstate highways.

As the ASTM locked wheel skid trailer appears insensitive to the effects of cornering force, it was decided to test with the Mu-Meter, which measures side-force friction, to determine if it is capable of measuring this phenomena.

TEST LOCATIONS

The tests were performed during November 1973 at numerous locations on the Interstate Highways in the Los Angeles area of California. The pavements tested had been thoroughly cleaned by numerous rainstorms the week before and also for several weeks before that. For this reason, the friction levels were higher than normally expected, which unfortunately reduced the friction range we had hoped to study.

TEST METHOD

Two different test methods were used in the evaluation. The first test method involved tests on particular projects where only a portion of the project has been grooved. In these tests, the Mu-Meter would make a continuous test for several thousand feet beginning on the un-grooved section and continuing thru the grooved section. In this way, a comparison could be made on sections that were identical in all respects except that one surface had been grooved. Tests were performed on both portland cement concrete pavement and asphaltic concrete pavements using this method.

In the second method, Mu-Meter tests were made on pavements that were scheduled to be grooved in the following two days. After grooving, the same locations were again tested to evaluate before and after results. It was expected that the results of these tests might differ with the first methods as traces of the slurry, caused by grooving, might still remain after such a short period and affect the readings. Only PCCP pavements were studied in this second evaluation.

TEST RESULTS

The data recorded for all tests are shown in Figure 23. These results were used to derive the equations shown in Figures 24 thru 26.

In Figure 24, the MuN of the grooved pavement was correlated to the MuN of the un-grooved PCCP pavement. In Figure 25, the same comparison was made using AC pavements. Figure 26 compares the MuN after grooving to the MuN before grooving, as described in the second method of this study.

From these results, it can be seen that equations were generated which yield reliable correlations with low error. Thus, it is possible to predict the expected MuN after grooving, and that the effects of grooving are detectable by use of the Mu-Meter.

It is of interest to note the equations generated for the PCCP surface.

A. $X = 30.2 + 0.57\ (Y)$ for Grooved MuN vs Un-grooved.

B. $X = 27.1 + 0.58\ (Y)$ for After Grooving vs Before Grooving.

These equations are essentially identical except that the tests made at the same location after grooving are approximately three MuN's lower. This is possible due to the presence of some retained slurry after grooving.

FIGURE 23

EFFECTS OF GROOVING

MuN Un-Grooved	MuN Grooved	Pavement Type
58	59	PCCP
64	62	PCCP
57	64	PCCP
60	62	PCCP
58	63	PCCP
62	66	PCCP
75	76	PCCP
59	67	PCCP
56	66	PCCP
60	72	AC
61	64	AC
64	68	AC
78	81	AC
60	61	AC
59	65	AC
66	75	AC
71	71	AC
62	66	PCCP
65	70	PCCP
68	67	PCCP
69	73	PCCP
69	68	PCCP
68	71	PCCP
62	69	PCCP
60	60	PCCP
51	63	PCCP
59	62	PCCP
61	64	PCCP
64	66	PCCP
50	57	PCCP
51	62	PCCP

FIGURE 23 Continued

EFFECTS OF GROOVING

MuN Before Grooving 11-27-73	MuN After Grooving 11-29-73	Pavement Type
62	62	PCCP
57	61	PCCP
55	60	PCCP
56	59	PCCP
66	64	PCCP
68	67	PCCP
63	63	PCCP
68	68	PCCP
69	68	PCCP
68	66	PCCP

COMPARISON OF GROOVED AND UN-GROOVED PORTLAND CEMENT CONCRETE

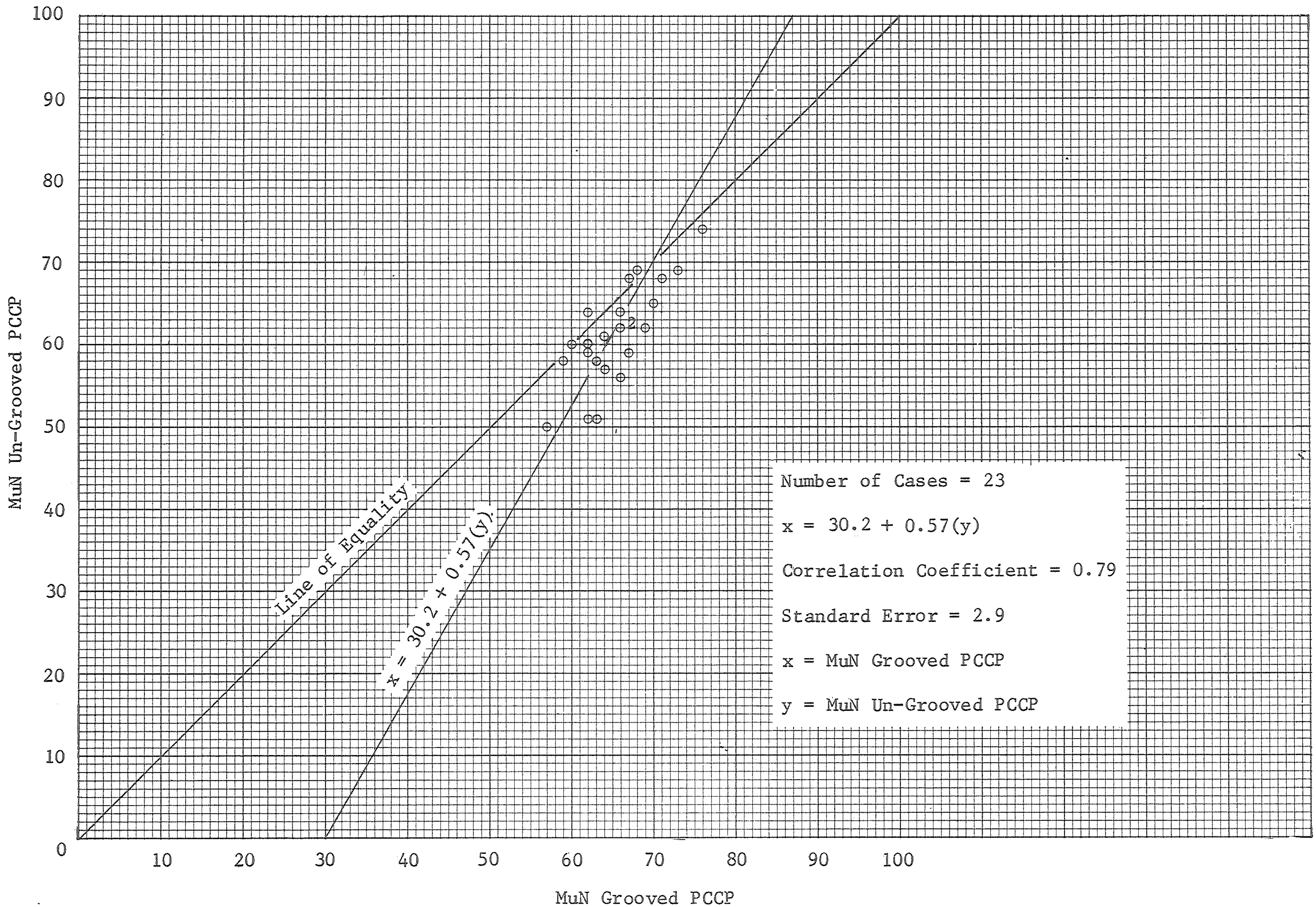
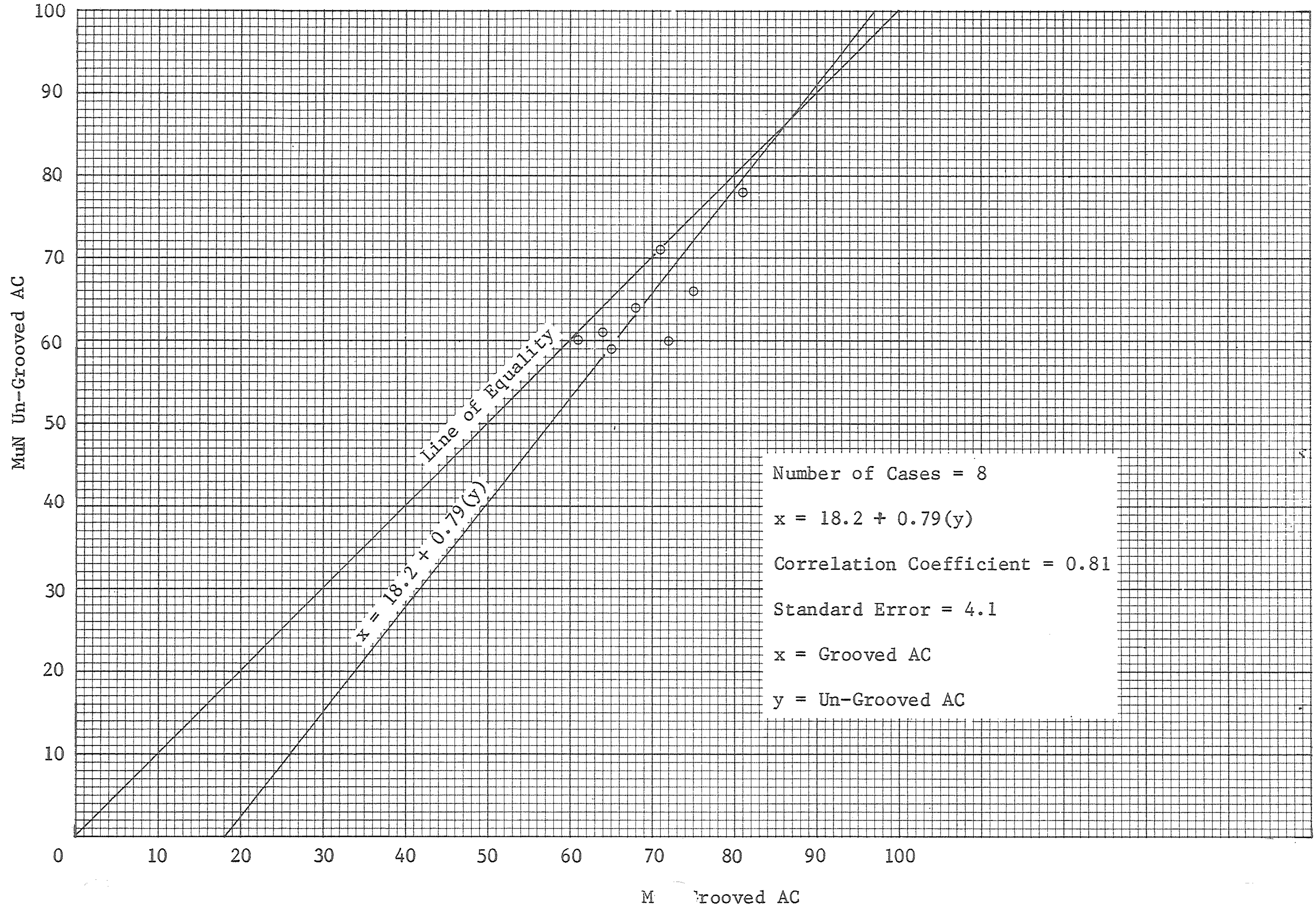


Figure 24

COMPARISON OF GROOVED AND UNGROOVED ASPHALTIC CONCRETE



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FRICTIONAL EFFECTS OF GROOVING PORTLAND CEMENT CONCRETE

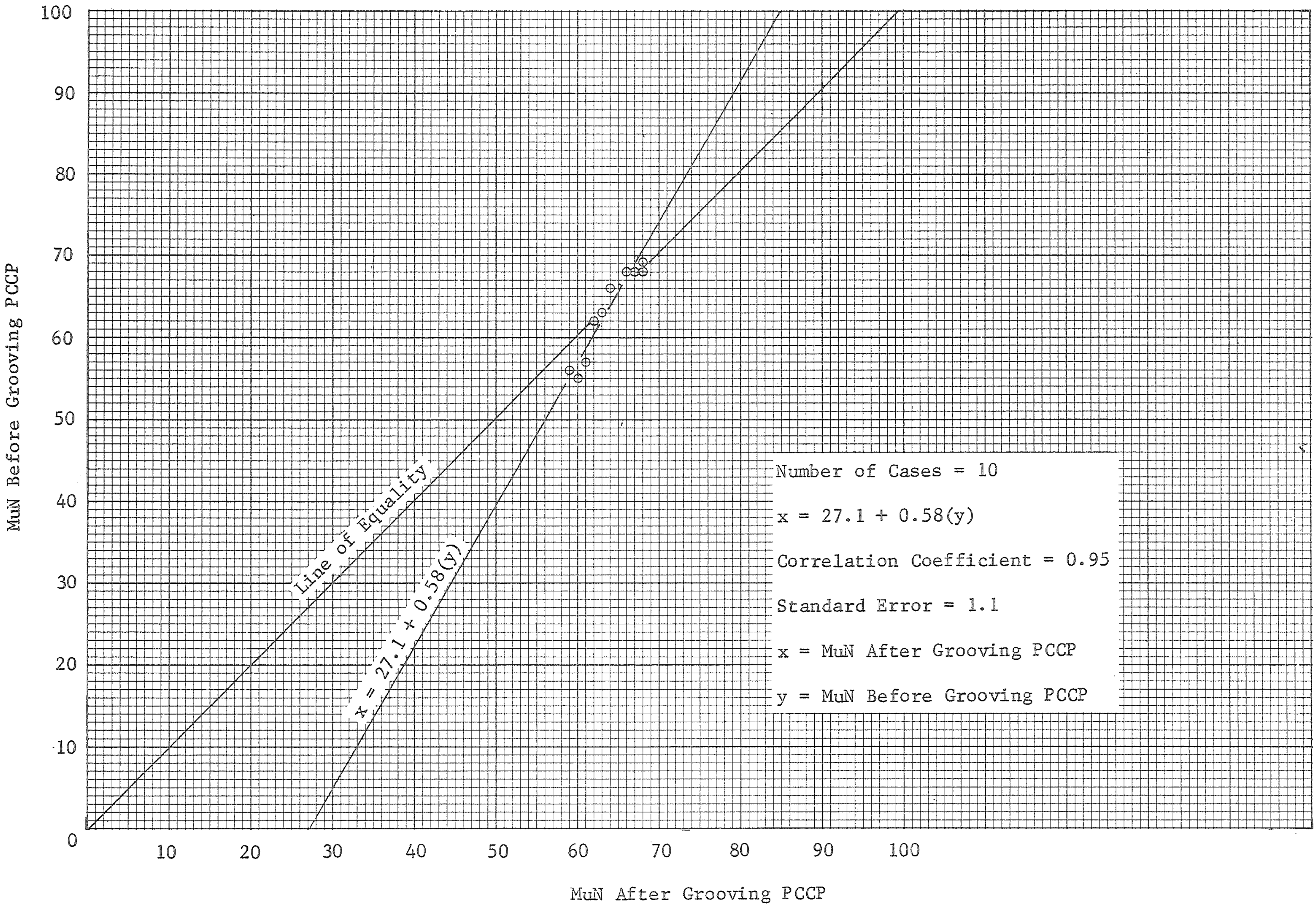


Figure 26

Since these equations are almost identical, it is probable that equation "A" would become equivalent to equation "B" if sufficient time, such as several weeks, were allowed for traffic to carry off some of the slurry dust. For this reason, it is suggested that the following equation be used for the prediction of the MuN after grooving.

For PCCP (1) Grooved MuN = 30.2 + 0.57 x (Un-grooved MuN)

and

For AC Pavement (2) Grooved MuN = 18.2 + 0.79 x (Un-grooved MuN)

As equation 2 is based on only a few samples with limited range, it is presented here only to show that there is a different equation necessary for AC pavements. This is because greater initial benefits apparently may be derived from grooving AC.

It was unfortunate that a wider range of friction levels were not available for these tests. Since the pavement was very clean, and probably at its highest level, even areas of low friction had higher readings. It would be of interest to test these surfaces again when the untreated surface was at its lowest friction level to see if it would effect the derived equations. Due to limited time, the greatest range of available friction was tested and used in this study. Further research is necessary to determine the accuracy of these equations over a wider range of initial friction levels.

It is of significant importance to note that the equations shown in Figure 26 reveal that the increase in sideforce friction after grooving is related to the friction level before grooving. If the general equation is expanded, it can be seen that theoretically if the original MuN was 20 before grooving, then the MuN after grooving would be approximately 42, or a 22 unit increase. Using the same premise, a MuN of 50 before grooving would have an expected MuN of 59, or a 9 unit increase, after grooving. At approximately a MuN of 70, no benefit from grooving would be obtained. From this we can see that when the initial value is above 70, an actual decrease in friction would occur if the pavement was grooved. For example, if the un-grooved MuN were 90, the grooved MuN would be 82, or an 8 unit decrease. Thus the benefits derived from grooving will depend on the original friction value. Grooving may increase, not change, or reduce the side-force friction, depending on the original un-grooved value. Since, for the most part, there is a significant increase for low friction pavements, which are usually the ones grooved, it appears that grooving will normally be expected to improve the surface friction.

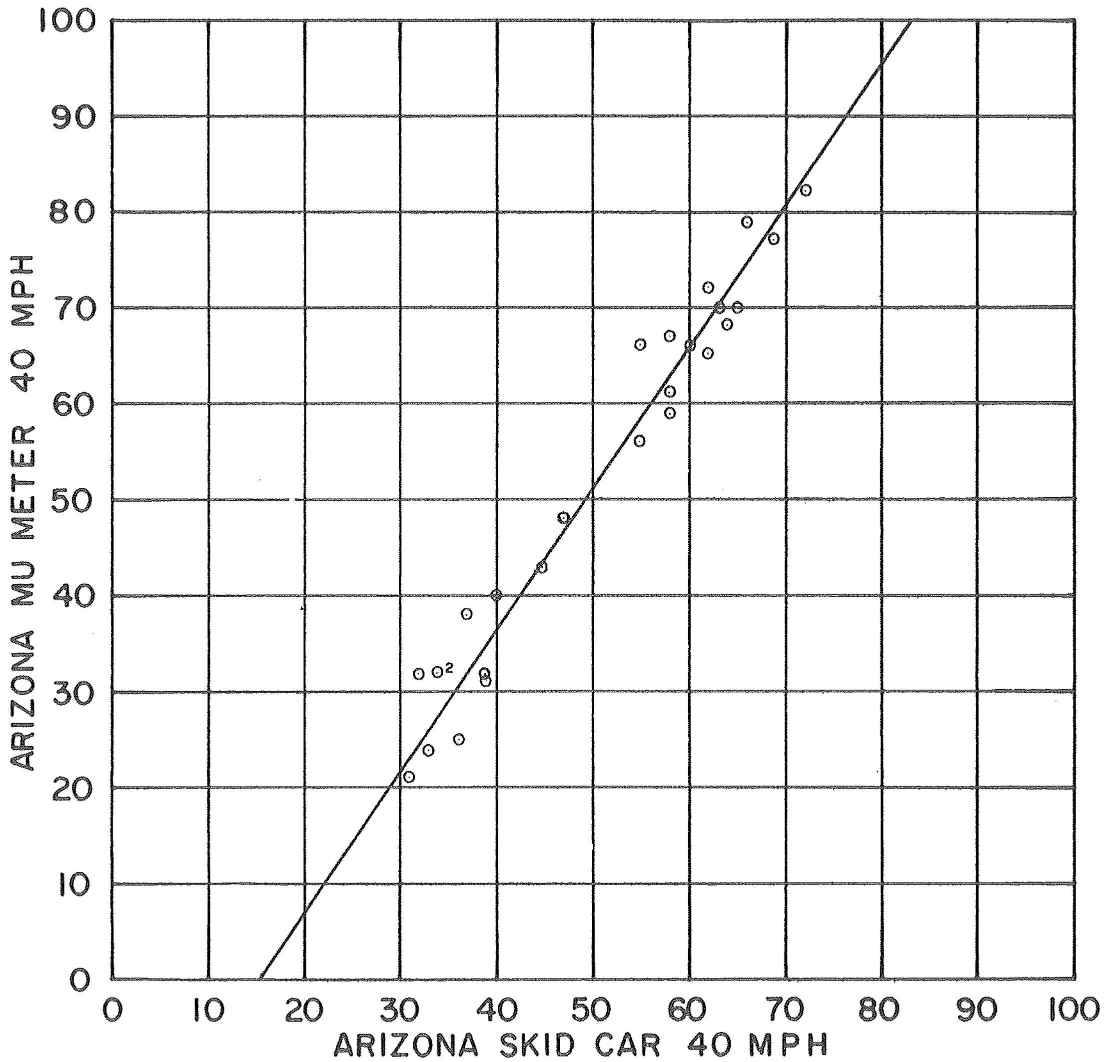
In any analysis, the cost benefit ratio should be considered in establishing a priority program for grooving. The above relationships should be considered when establishing priorities.

Figure 27 is reprinted from an earlier report[1], and is presented so that the reader may have a means of comparing the Mu-Meter numbers presented here with those of a common reference.

SUMMARY

From these tests, it can be seen that the Mu-Meter is apparently capable

SKID CAR CORRELATION
JANURARY 1972



Number of cases = 26
Skid Car = 15.219 + 0.6777(Mu-Meter)
Mu Meter = −22.456 + 1.4755(Skid Car)
Correlation coefficient = 0.9802
Standard error = 2.72 Car values

Figure 27

of detecting variations in the coefficient of friction between an un-grooved and grooved pavement.

Although the range of data was limited by physical conditions, satisfactory equations were derived, over the friction range studied, to predict the Mu-Meter number for a grooved pavement when the Mu-Meter number of the un-grooved pavement is known.

The equations derived for PCCP pavements to calculate the MuN after grooving are different from that of the AC pavement. The results from this study indicate that AC pavements tend to achieve a greater increase after grooving than do PCCP pavements.

For PCCP pavements, the friction after grooving may increase, decrease, or remain unchanged depending on the original friction level. In the normal friction ranges of polished PCCP pavements there can be a significant increase in side force friction after grooving. It must be remembered, however, that the amount of increase will decrease as the original friction level becomes higher.

The general trends established in this section should be considered when establishing priority programs for grooving. As decreasing benefits will be derived from grooving higher friction pavements, consideration must be given to cost benefit ratios. The equations in this chapter should allow the reader to obtain a general idea of the benefits to be derived by grooving. Further research over a wider range of friction levels might produce equations which would allow an accurate prediction of the cost benefit ratios associated with grinding and grooving.

PART FOUR

WET PAVEMENT ACCIDENT ANALYSIS FOR ARID CLIMATES

INTRODUCTION

Friction level alone does not always indicate whether a pavement meets the frictional demands of the driving public. Other variables that must be considered include geometrics, signing, cross section superelevation, skid resistance, obstacles, traffic volumes, percent of time the pavement is wet, likelihood of sudden vehicular maneuvers, and accident analysis. The last of these variables, accident analysis, will normally give an excellent indication if the pavement was hazardous or skid prone in the past. Unfortunately, accident analysis alone will not always detect a location that is presently skid prone, or may be skid prone in the future. Due to the time required to process accident data, a considerable period of time may pass before such information is available to the engineer. For this reason, friction data and accident analysis should both be considered in the initial evaluation of the frictional properties of a pavement. Once potentially hazardous locations are determined through this method, a more detailed analysis, which includes the previously mentioned variable, can be conducted. A priority programming of corrective actions may then follow.

In arid climates, rainfall is low and extremely variable. It is important that any accident analysis in such areas consider these two variables, otherwise, very misleading results will be obtained. In an effort to establish a system which will effectively weigh the importance of these unique rainfall conditions, the following analysis was conducted.

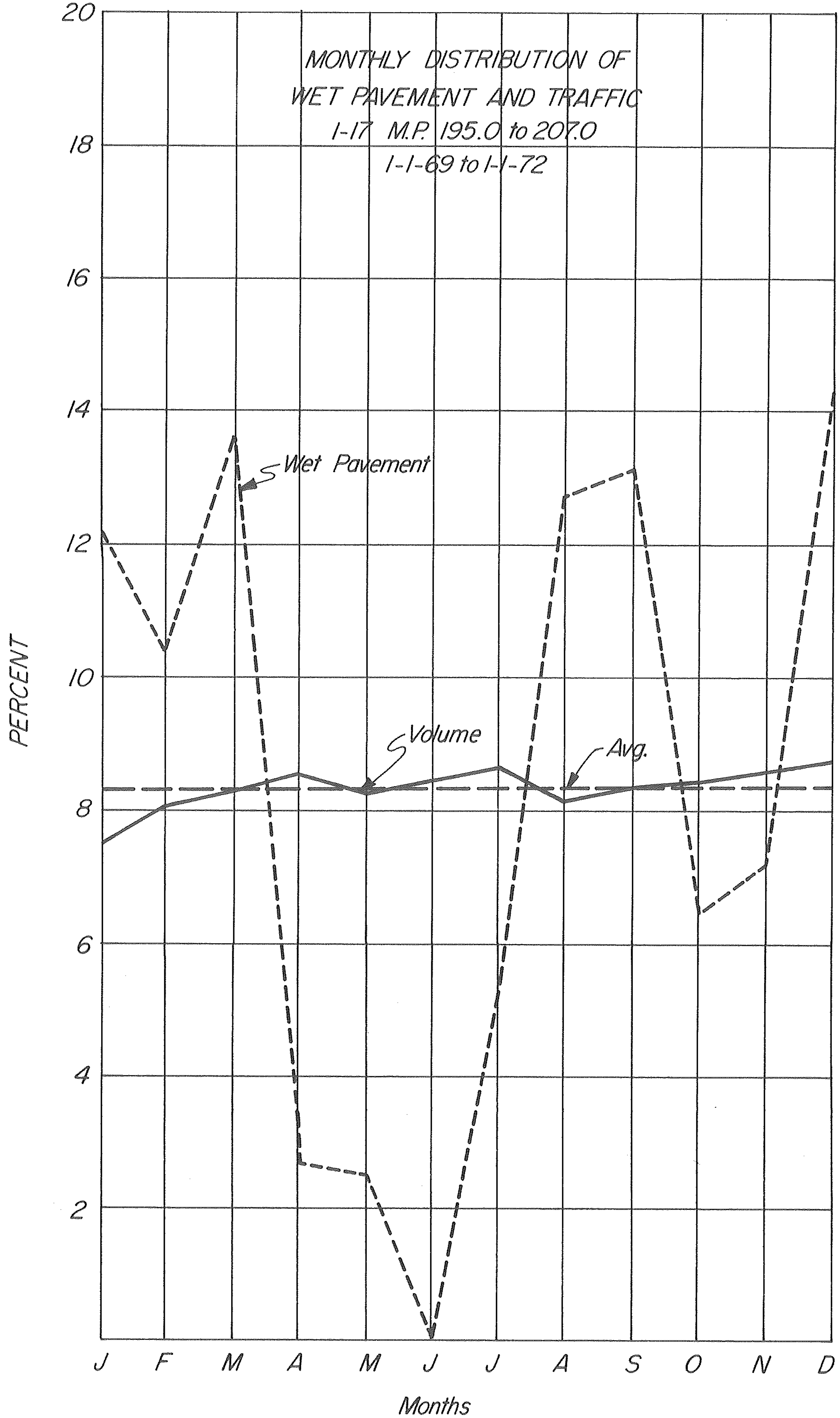
DETERMINATION OF VEHICULAR EXPOSURE TO WET PAVEMENT

When analyzing wet pavement accidents, the most certain method of obtaining the true locations of areas of poor skid resistance and high probability of wet pavement accident occurrence is to use rates of vehicular exposure to wet pavement. Wet exposure is the number of vehicles traveling on wet pavement, or wet pavement ADT. In a state with large differences in annual precipitation measurements such as Arizona, the need for an exposure rate-based analysis of wet pavement accidents is a definite necessity. Areas of the southwestern deserts average less than 5 inches of precipitation yearly, while the central mountain areas may receive 30 or more inches of precipitation. The times, duration and magnitudes of precipitation statewide vary as widely as the annual precipitation averages (Figure 28).

Our approach of determining values for amounts of wet pavement traffic volume which will most closely approximate the actual amount of wet pavement traffic is discussed in the following paragraphs.

It was determined in a report published by the California Division of Highways[7] that .01 of an inch (.025 cm) of rain falling within one hour is the minimum amount sufficient to "wet" pavement. In addition to this, we added one hour each time .01 inch (.025 cm) or more precipitation was measured for one or more continuous hours. For example, one hour of precipitation measuring .01 inch (.025 cm) or more is equivalent to two hours of wet pavement.

Figure 28


MONTHLY DISTRIBUTION OF
WET PAVEMENT AND TRAFFIC
I-17 M.P. 195.0 to 207.0
1-1-69 to 1-1-72
20
18
16
14
12
10
8
6
4
2
PERCENT
Wet Pavement
Volume
Avg.
J
F
M
A
M
J
J
A
S
O
N
D
Months

Two consecutive hours each reporting .01 inch (.025 cm) or more precipitation was equal to three hours of wet pavement.

A check of the assumptions of wet pavement hours was made in the following method. Accidents reported as having occurred on wet pavement for 72 lane mile section (115.9 lane km) of highway during a three year period were checked against the tabulation of dates and hours in which wet pavement was assumed possible, using the reported climatological data. The results showed 77% of all reported wet pavement accidents occurred within the assumed hours of wet pavement. Additionally, nine percent of the wet pavement accidents occurred one hour before the assumed hours of wet pavement and nine percent occurred one hour after. This difference can be attributed to the fact that precipitation may have fallen at the recording station then moved westerly to the Freeway, or in a reverse manner. If this logic is accepted, a 95% accuracy in determining the hours of wet pavement by the method used is possible. Using available climatological and traffic volume data, the following methods can be used to determine a value of wet Average Annual Daily Traffic (AADT):

1. <u>Actual Rainfall and Traffic Volume Method</u>

   Reported precipitation for each given hour of the year is noted and the traffic volume, as recorded for the corresponding hour of precipitation plus one hour, is recorded as the hours of wet pavement. This corresponding traffic volume for one year divided by 365 determines the wet AADT.

2. Average Hourly Rainfall and Average Hourly Traffic

   Reported precipitation adjusted to the number of wet pavement hours are averaged for the time period to derive a percent of a 24 hour day each hour is wet. Also hourly volume counts are averaged and an Average Annual Hourly Traffic (AAHT) is derived. The hourly percent of wet pavement multiplied by the AAHT yields the wet AAHT. The wet AAHT totaled for 24 hours, is the Wet AADT.

3. Ratio of Wet Hours of Pavement to Total Hours and AADT

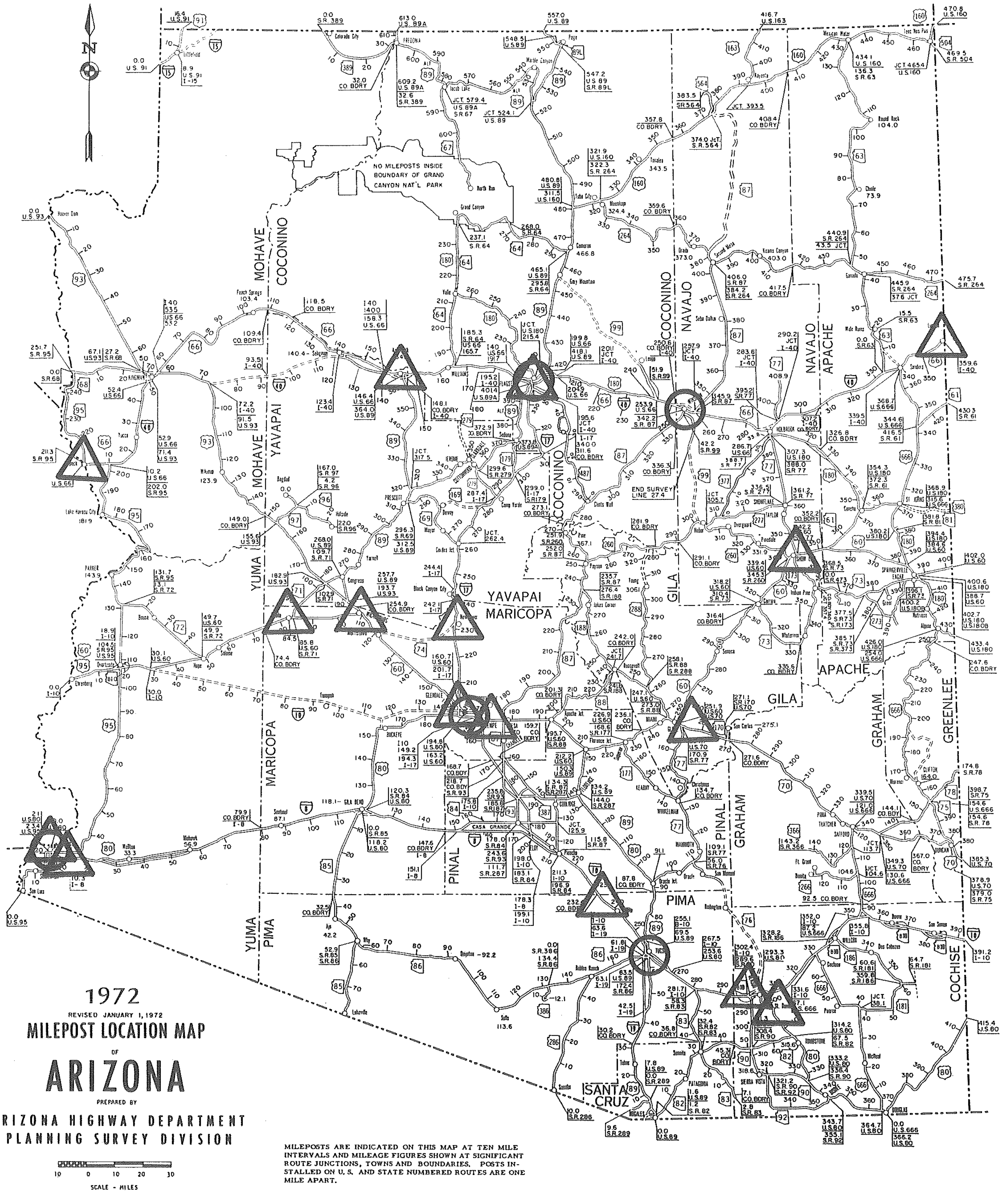
   The number of hours of wet pavement divided by total hours, yields percentage of time of wet pavement hours. This multiplied by the AADT yields a wet AADT.

Using the climatological data from the U.S. Department of Commerce National Oceanic and Atmospheric Administration Environmental Data Service and the Sky Harbor Airport in conjunction with Traffic Volumes from the Arizona Department of Transportation, Planning Survey Division, for the years of 1972 and 1973 at the 16th Street and I-17 permanent counting station, the following AADT's were developed:

	Method	Wet AADT	Wet % of Total AADT	Total Vehicles Exposed to Wet Pavement for Year
	1	1015	1.70%	371,571
1972	2	2239	3.74%	819,474
	3	1131	1.89%	413,946
	1	1087	1.62%	230,472
1973*	2	2433	3.62%	515,796
	3	1757	2.61%	372,484

*To 8-1-73

In method one, precipitation and wet pavement determine the hourly reported vehicle count. For example, if rain was reported for the hour ending at 9 a.m. October 21, 1972, the total volume counted during the hour ending at 9 a.m. October 21, 1972, was added at "wet hour's" volume. This method is, however, impractical to use. It requires a constant 24-hour recording traffic volume count and a constant 24-hour precipitation recorder located within reasonable proximity of one another. In the State of Arizona, there are only five permanent 24-hour precipitation recording stations. This, in combination with 16 different 24-hour traffic counting stations, gives us only four possible locations where both types of recording devices can be correlated (Figure 29). The mathematical operation is also quite lengthy, matching wet pavement times to corresponding volumes and summing, since to date the traffic volumes are not immediately available for manipulation with the computer.



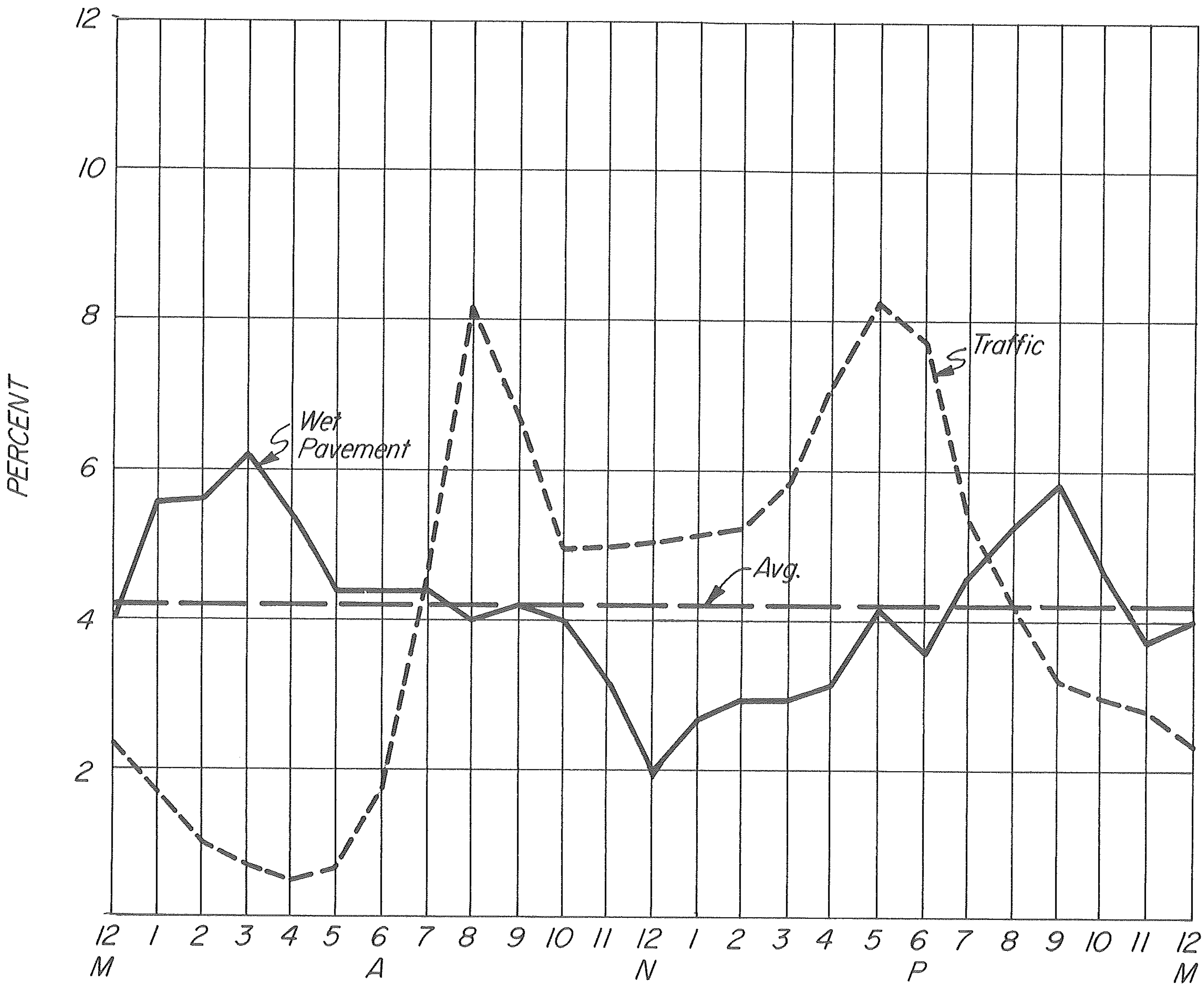
○ 24 hr. Precipitation Recording Station

△ 24 hr. Traffic Volume Recording Station

Figure 29

Method 2 yields unreasonable high wet pavement rates and is considered not as reliable as method 3 which appears to give results which most nearly approximate the "best" estimate of wet ADT. This method is also the simplest of the three. It is also substantiated by the fact that although there may be no correlation between hourly distribution of rainfall and traffic (Figure 30), that there is a definite relationship between hourly distribution of wet and dry pavement accidents (Figure 31). The method seems to give approximations which are higher than actual. Use of a higher than actual AADT in the wet accident rate calculations yields conservative results - or lower than had the actual wet AADT been used.

California Division of Highways, Traffic Department, developed a method of determining a statewide approximation of wet ADT's[8]. This method is similar to Method 3 with the addition of some factors which they felt would more closely approximate the wet traffic volumes considering seasonal and area factors. A map for the State of California was developed, similar to an isohyetal map, which shows lines of equal percentage of time the pavement is wet. This map was made by using precipitation reports from 350 scattered recording stations for the years 1957 - 1967 and is used as a standard for any year. This data most likely gives a fair approximation of wet pavement time. However, when percentage wet times are taken from this map and applied to 1969, 1970, 1971, or 1972 ADT's and any abnormal precipitation times, durations, or magnitude had occurred in the later years not included by the map, a large error would exist in the approximated wet ADT. It is on this

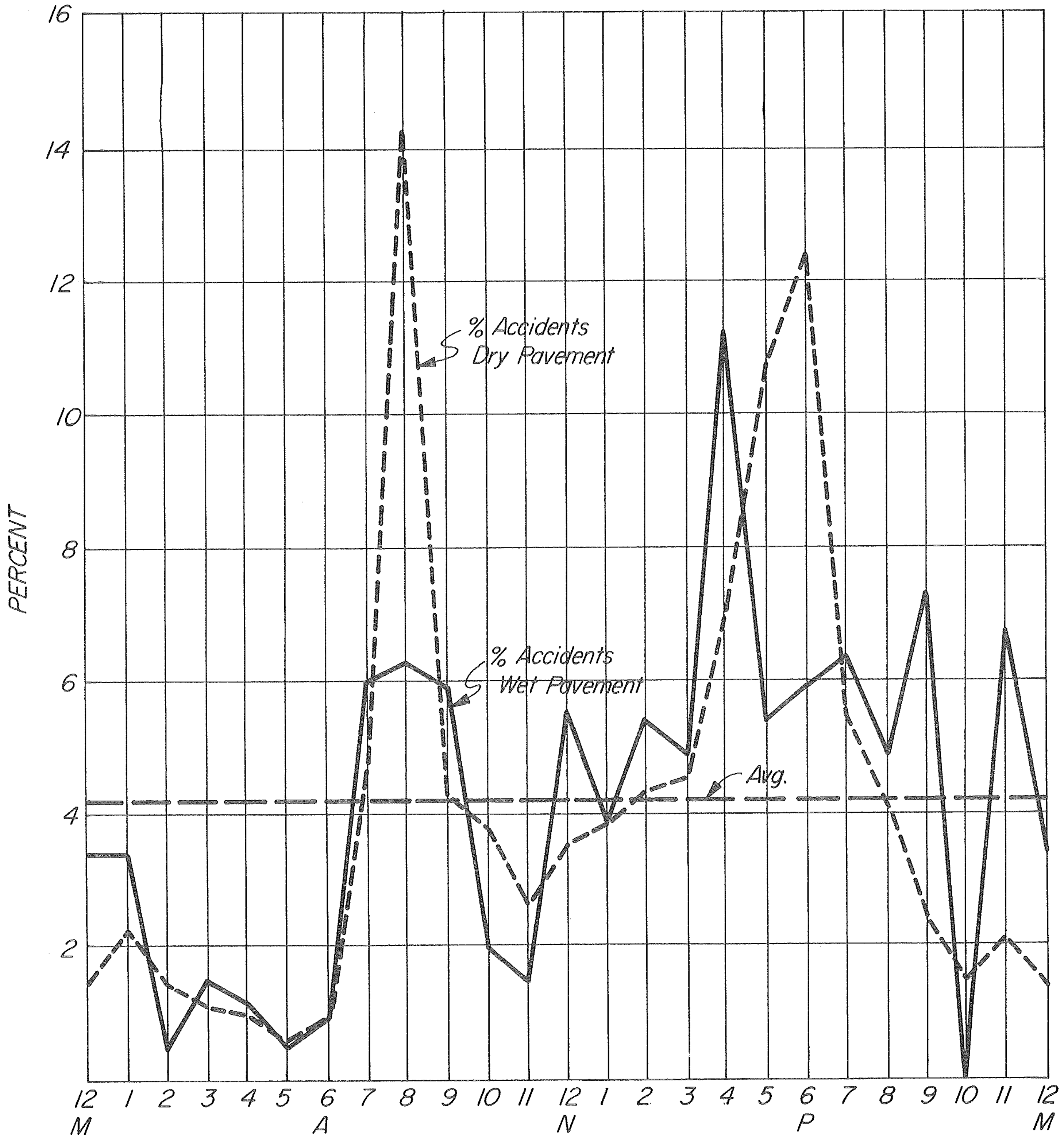


HOURLY DISTRIBUTION OF WET PAVEMENT AND TRAFFIC

I-17

M.P. 195.0 to M.P. 207.0

1-1-69 to 1-1-72



HOURLY DISTRIBUTION OF ACCIDENTS
I-17
M.P. 195.0 to M.P. 207.0
1-1-69 to 1-1-72

basis than an isohyetal type map based on climatological data for years identical to the accident and traffic volume data is proposed.

To substantiate this theory, the following observations are offered. Data taken from the Sky Harbor recording station in Phoenix produces the following number of hours that pavement can be considered wet:

1969 - 235 hours or 2.68% wet time

1970 - 135 hours or 1.54% wet time

1971 - 112 hours or 1.28% wet time

1972 - 166 hours or 1.70% wet time

1973 thru July 31 - 133 hours or 2.6.% wet time

Average - 1.96% wet time

Had an average wet time of 1.96% been used to calculate wet accident rates from the "dryer" years of 1970, 1971 and 1972, a 32% error would have been made in the Wet ADT and the accident rate because the actual percent wet time for these years was only 1.51%. It would appear imperative that due to the extremely variable amounts of precipitation, together with varying numbers of traffic volumes and accidents, percentage of wet pavement times must be derived using climatological data from the same years corresponding to traffic volume and accident data.

Using this theory as a basis, a map of Arizona was designed with isohyetal lines, or lines of equal percentages of time the pavement is wet.

This map was designed using the following:

1. 24-Hour recorded precipitation amounts for the years 1970, 1971, and 1972 from weather recording stations at Flagstaff, Winslow, Yuma, Phoenix and Tucson, Arizona.

2. Daily recorded precipitation amounts from the following Arizona cities for the years 1970, 1971 and 1972.

Show Low	Casa Grande
Williams	Carefree
Cordes Junction	Winkleman
Prescott	San Manuel
Seligman	Parker
Globe	Salome
Payson	Quartzsite
Fredonia	Wellton
Cameron	Clifton
Kayenta	Safford
Page	San Simon
Sanders	Willcox
St. Johns	Douglas
Kingman	Ft. Huachuca
Apache Junction	Nogales
Wickenburg	Sells
Buckeye	Ajo
Gila Bend	Benson
Cortaro	

3. 2-Year and 100-year maximum intensity precipitation maps for the State of Arizona, prepared by the U.S. Weather Service for the Soil Conservation Service.

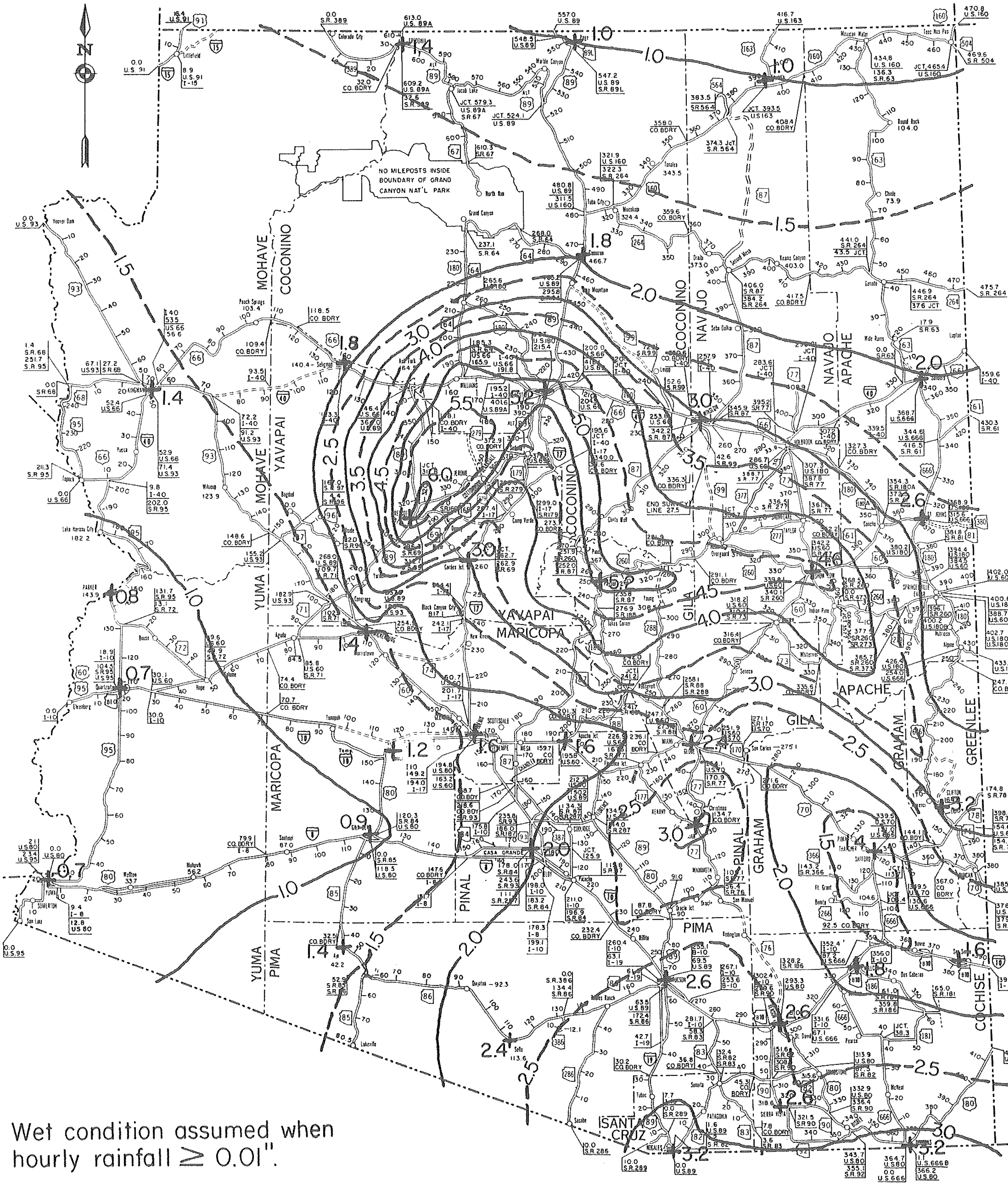
The development procedure is described as follows:

The five 24-hour precipitation recording stations were used to determine actual percentage of wet pavement time at their specific locations. The State was then divided into five sectors by use of the precipitation intensity maps to gather locations of similar precipitation characteristics near the five 24-hour recorders. Thirty-seven daily reporting precipitation stations were used to get good statewide coverage. Daily precipitation reports from the 37 stations were compared to those from the five 24-hour recording stations for the years 1970-1972. Using these comparisons and the 2-year and 100-year intensity maps, approximations were developed for percent of time the pavement was wet at each location. With now 42 locations of know times of wet pavement, the statewide map was constructed using straight line approximations to determine locations of isohyetal lines. Approximations can be obtained from the map (Figure 32) for percentage of wet pavement times to the nearest one-tenth of a percent.

ACCIDENT ANALYSIS

Using the "Percent of Time Pavement is Wet" and the AADT's for the State System, wet pavement accident rates can be calculated for each state route or location.

PERCENT OF TIME PAVEMENT IS WET



Wet condition assumed when hourly rainfall ≥ 0.01".

Values based on data for years 1970 - 1972.

Figure 32

The methodology used to calculate wet and dry accident rates involves the following procedure. Accidents are divided into two groups, those occurring on wet pavement and those occurring on dry pavement. These values are used in the following equations:

$$\text{Wet Accident Rate (per wet million vehicle miles)} = \frac{A_w \times 10^6}{L \times ADT_w \times T}$$

where:

A_w = Accidents reported occuring on wet pavement in a particular period.

L = Length of section considered in miles (1.6093 Km).

ADT_w = Average vehicular daily volume on wet pavement (obtained by multiplying % time pavement is wet from map by AADT) for same period.

T = Number of days in period.

This can be compared with:

$$\text{Dry Accident Rate (per day million vehicle miles)} = \frac{A_d \times 10^6}{L \times ADT_d \times T_d}$$

A_d = Accidents occurring on dry pavement for a particular period.

L = Length of section considered in miles.

ADT_d = Average vehicular volume traveling on dry pavement for same period [ADT (Total) - ADT_w].

T_d = Number of dry days in period.

The number of wet accidents per million vehicle miles is very important when comparing accident rates on an overall state basis. There is, however, a faster method of obtaining a general indication of a possible wet pavement problem at a particular location. This method is called the Wet Accident Index. To calculate this value, the number of wet and dry pavement accidents should be known for the area in question. A quick analysis of wet accident potential can be determined by the following equation for the Wet Accident Index (WAI).

$$WAI = \frac{A_w \times P_d}{A_d \times P_w} \quad \text{where } P_d = 100 - P_w$$

where: A_w = Number of accidents on wet pavement.

A_d = Number of accidents on dry pavement.

P_w = % time pavement is wet.

P_d = % time pavement is dry.

A Wet Accident Index (WAI) of one (1) would indicate that there is no difference between the number of accidents occurring when the pavement is wet, then when it is dry. A WAI of less than one (1) indicates that there are less accidents when the pavement is wet, then when it is dry. A WAI of greater than unity indicates that there are that number of times more accidents when the pavement is wet, then when it is dry. For example, a WAI of 4.0 would indicate there are four times as many accidents when the pavement is wet, then when it is dry. It follows that a WAI of greater than unity indicates a pavement has a greater accident potential when wet. The greater the WAI above unity, the greater the wet accident potential.

EXAMPLE

In a particular one mile section of highway, 32 wet and 157 dry pavement accidents occurred over a three year period. The AADT for this section was 49,050 and the site was in an area where the map in Figure 32 indicated that the pavement was wet 1.6 percent (.016) of the time.

If the actual number of accidents are compared, it is found that only seventeen percent of the accidents were on wet pavement, or that there were approximately six times as many dry pavement accidents as wet pavement accidents. Unfortunately, this is not a true picture of the real condition of the pavement.

The actual wet and dry accident rates must be calculated as follows:

$$\text{Wet Accident Rate} = \frac{32 \times 10^6}{1 \times 49,050 \times 0.016 \times 3 \times 365} = 37.24$$

or 37.2 accidents per wet million vehicle miles.

$$\text{Dry Accident Rate} = \frac{157 \times 10^6}{1 \times 49,050 \times .984 \times 3 \times 365} = 2.97$$

or 2.97 accidents per dry million vehicle miles.

This shows that actually, there are 12.5 times more accidents when the pavement is wet than when it is dry. A similar ratio could have been obtained by just considering the accidents themselves and calculating the Wet Accident Index.

$$WAI = \frac{A_w}{A_d} \times \frac{\%d}{\%w} = \frac{32}{157} \times \frac{0.984}{0.016} = 12.5$$

Since the WAI equals 12.5, then it indicates that there are 12.5 times as many accidents when the pavement is wet than when it is dry. This is equivalent to the value reached in the first part of this example. It does not, however, indicate the actual accident rate, and thus a low WAI does not necessarily indicate a safe pavement.

From this example it can be seen that it is extremely important to consider the magnitude of the time the pavement is wet in any accident analysis. To do otherwise, will yield completely misleading and incorrect results.

The systems we have devised here are not one hundred percent accurate, but are close, conservative straight-forward approximations which can be used and derived without large amounts of time consuming calculations.

In this study a light snow condition was considered, as it is recorded as precipitation. The effects of heavy snow melts were also considered for input, however, eliminated since highways are cleared rapidly. Sections with an excessive problem are infrequent and should be considered individually. Such a section needs special attention and the engineer should modify our system to meet extremely localized conditions.

SUMMARY

Using the "Percent of Time Pavement is Wet" and the AADT's for the State System, wet pavement accident rates can be calculated for each state route. These rates can then be compared by road category, to determine the high wet pavement accident locations. These locations when compared with the pavement friction log will develop a correlation of high wet pavement accident frequency with low pavement friction. Priorities for pavement surface treatment, to increase pavement friction, lower skidding potential and thereby reduce wet pavement accidents, can then be determined.

The WAI is an alternate system which can be used as a quick, reliable method of estimating wet accident potentials at spot locations throughout the state, when only a record of wet and dry pavement accidents is available.

CONCLUSIONS

PART ONE

Differential friction can cause an extremely hazardous condition for a braking vehicle. As this problem can occur at high as well as low friction levels, it should be given major consideration in any pavement friction evaluation.

For any particular combination of differential friction numbers and speed, the number of degrees rotation can be calculated by the equations given in this report. Thus the magnitude of the problem at various locations can be predicted by normal testing techniques, without the hazards of actual vehicle rotation tests.

Construction and maintenance operations can cause, or contribute to, differential friction. For this reason, consideration should be given to this problem during design, construction and maintenance.

There are numerous causes of differential friction. In order to properly understand the nature of this phenomena, an educational program may be necessary, otherwise corrective actions may not be effective and may actually make the condition worse.

In addition to visual observations and friction tests, accident records should be examined for locations where there are a high number of accidents in which loss of control was attributed to the rotation of the vehicle while braking.

There is a strong indication that differential friction may be as important in the causation of accidents as low friction level. If this is the case, a major re-evaluation of present pavement friction analysis, design and corrective techniques may be necessary. For this reason, friction inventories and studies where only one wheeltrack is tested may not define actual wet pavement problems and thus produce misleading results.

PART TWO

Proper texturing of portland cement concrete pavement is essential if a durable, high friction surface is to be obtained. In this study, the longitudinal nylon bristle broom yielded, by far, the most easily obtained, uniform, durable and skid resistant surface texture. All the burlap drag textures produced poor, non-uniform textures that were extremely dependent on time of texturing and operator technique. The burlap drag textures were also significantly less durable.

Experimental fluted float textures produced unusual surfaces that were very uniform and definitely have merit. The texture yields a high coefficient of friction and is durable. If equipment was produced to apply such a texture, it is believed that excellent results could be obtained.

Results from this study indicate that transverse textures are rapidly worn away, and thus may be undersirable, in areas where snow and ice removal operations are regularly conducted.

PART THREE

Unlike the ASTM locked wheel skid trailer, the Mu-Meter, which measures side-force friction, is capable of detecting the frictional effects of grooving. Although the range of data was limited, satisfactory equations were derived over the range studied to predict the coefficient of friction after grooving, when the friction level of the ungrooved surface is known. Separate grooving equations are necessary for PCCP and AC pavements.

The coefficient of friction after grooving may increase, remain the same, or decrease depending on the original friction level. As the original friction level increases, the rise in friction level after grooving is reduced. Since decreasing benefits may be derived from grooving higher friction pavements, the trends established in this study should be considered when establishing priority programs.

PART FOUR

In arid climates where rainfall is low and extremely variable, an exposure rate-based analysis of wet pavement accidents is a necessity. Without such an analysis, completely erroneous and misleading results will be obtained.

Thru this study, a viable system for predicting wet exposure accident rates has been developed for Arizona. The system described can be used for any accident analysis in an arid climate or other location where rainfall is highly variable. The study also describes the Wet Accident Index which is a value which will allow a quick evaluation of wet accident potentials for a particular area.

Using these two systems and the surface friction inventory, a priority system for pavement surface treatment to increase pavement friction, reduce skidding potential and thus reduce wet pavement accidents can be achieved.

RECOMMENDATIONS

1. Differential friction should be given major consideration in any pavement friction evaluation.
2. During friction inventories, locations of visual differential friction should be recorded.
3. As differential friction can occur at high as well as low friction levels, the entire highway system should be evaluated for wet accident potential caused by this problem.
4. During construction, the ribbons should be placed so that all longitudinal joints fall outside of each lane at, or very near, the location of the lane stripes.
5. During construction, it would be desirable that the distress lane and shoulder lane be constructed of the same surface type as the travel lane. When this is done, a well maintained stripe at the edge of the travel lane is required for visual guidance.
6. During maintenance and construction operations, it is imperative that some operations be full lane width.
7. Heavy crack patching or corrective actions in only one wheel-track should be avoided.
8. Educational programs should be provided to discuss the effect and causes of differential friction with maintenance and

construction personnel. Without a complete knowledge of the subject, numerous differential friction problems are likely to be created by such forces.

9. A large scale accident analysis should be made to determine if, in actuality, differential friction is as important in the causation of wet pavement accidents, as low friction level. If this is the case, a major re-evaluation of present friction analysis, design and corrective techniques may be necessary.
10. Use of the longitudinal nylon bristle broom texture is recommended. The use of various forms of burlap drag textures should be avoided.
11. Future development of equipment to produce the longitudinal fluted float texture should be considered.
12. Transverse textures should possibly be avoided in areas where snow and ice removal operations are anticipated.
13. Rubber molded samples of the "desired texture" should be given to the contractor and inspector. Such a sample in the field allows "on the spot" comparison of the texture being achieved and the desired texture. Instructions for producing such molds are presented in this paper.
14. An evaluation of the expected increase in friction should be considered before grooving. The trends established in this report should be considered when establishing priority programs,

as decreasing monetary and frictional benefits are achieved as the original friction level (before grooving) increases.

15. A wet accident rate, such as described in this report, should be made for the entire state highway system. This information should then be used as an input for a portion of a pavement surface treatment priority program.

IMPLEMENTATION

For the State of Arizona, the Arizona Department of Transportation has already implemented Item Numbers 1, 2, 3, 8, 10, 12, 13 and 15 of the recommendations listed in this report.

The State is presently contemplating the implementation of Number 4, 5, 6, 7 and 14 of these recommendations and the approach which will be taken concerning these matters.

Our research indicates that differential friction may be as important in the causation of accidents as low friction level. As differential friction should be an immediate nationwide concern, consideration should be given to emphasizing and educating highway and maintenance engineers in regards to this phenomena. This problem should be given serious consideration as a demonstration project.

ACKNOWLEDGEMENTS

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REFERENCES

1. Burns, J.C. and Peters, R.J. "Surface Friction Study of Arizona Highways", HPR 1-9 (162), Arizona Department of Transportation, Materials Services, Research Branch, August 1972, 75 pages.

2. "Stopping Distance on Paved Surfaces Using a Passenger Automobile Equipped with Full-Scale Tires", ASTM Tentative Method E 445-71T, 1974.

3. Zuk, William, "The Dynamics of Vehicle Skid Deviation as Caused by Road Conditions", Paper Presented to the First International Skid Prevention Conference, September 1958.

4. Emery, Jr., D.K., "Paved Shoulder Encroachment and Transverse Lane Displacement for Design Trucks on Rural Freeways", a Preliminary Report, Georgia Department of Transportation, January 1974.

5. Chamberline, W.P. and Amsler, D.E., "Pilot Field Study of Concrete Pavement Texturing Methods", New York State Department of Transportation, Highway Research Record No. 389, 1972.

6. "Interim Recommendations for the Construction of Skid Resistant Concrete Pavement", A.C.P.A. Technical Bulletin No. 6, August 1969.

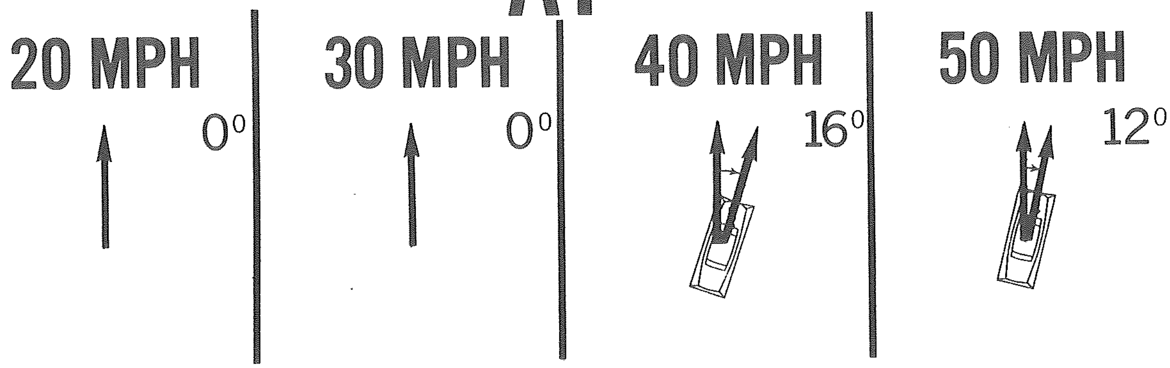
7. Farnsworth, E.E. and Johnson, M.H., "Reduction of Wet Pavement Accidents on Los Angeles Metropolitan Freeways", California Division of Highways, June 1971, 20 pages.

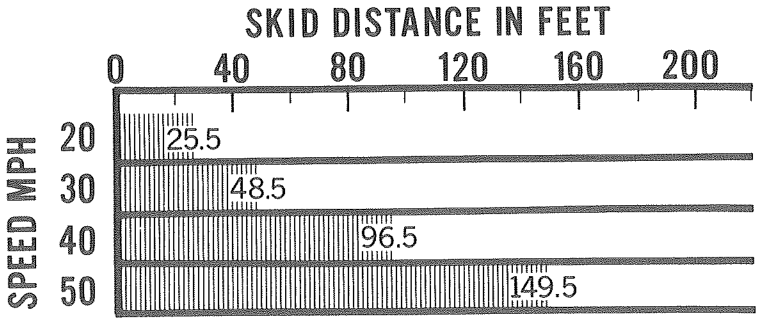
8. Karr, J.I. and Guillory, M., "A Method to Determine the Exposure of Vehicles to Wet Pavements", California Division of Highways, January 1972, 28 pages.

APPENDIX A

SITE NO. 1

DEGREES OF ROTATION AT





ROTATION IN °/FT = 0.17

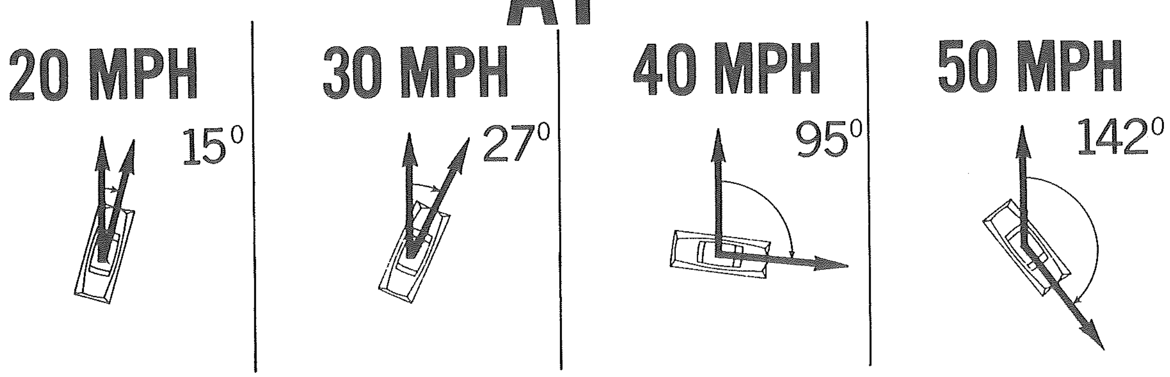
DIFFERENTIAL 40 = 5

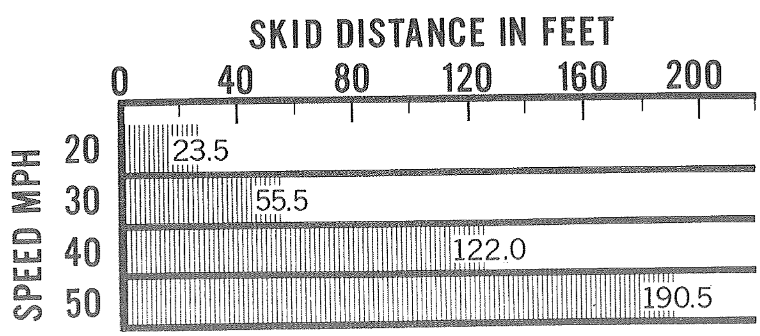
SKID NUMBER

	LT.	RT.
20 MPH	64	70
40 MPH	57	62

SITE NO. 2

DEGREES OF ROTATION AT





ROTATION IN °/FT = 0.78

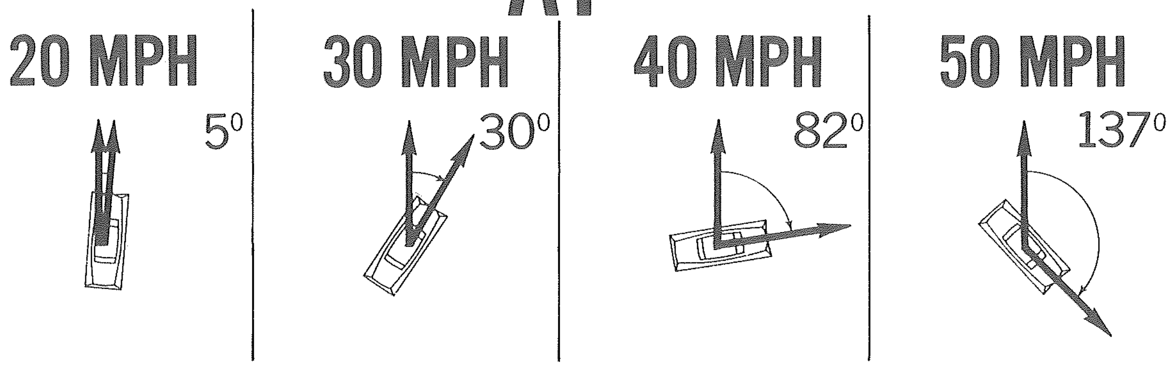
DIFFERENTIAL 40 = 21

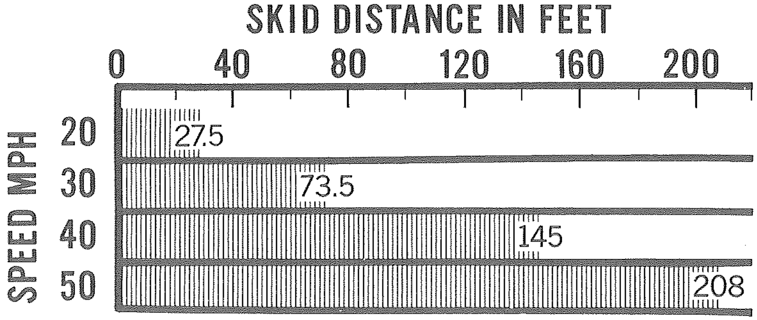
SKID NUMBER

	LT.	RT.
20 MPH	44	64
40 MPH	36	57

SITE NO. 3

DEGREES OF ROTATION AT





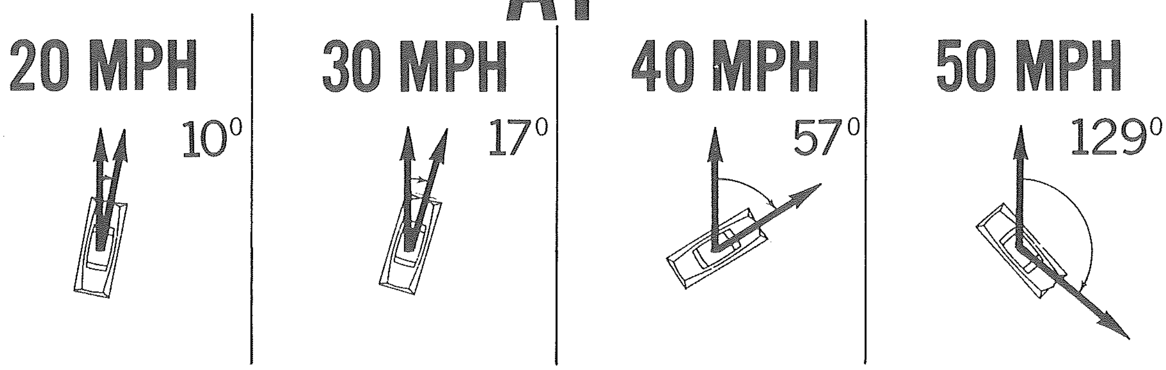
ROTATION IN °/FT: 0.57
DIFFERENTIAL 40 : 14

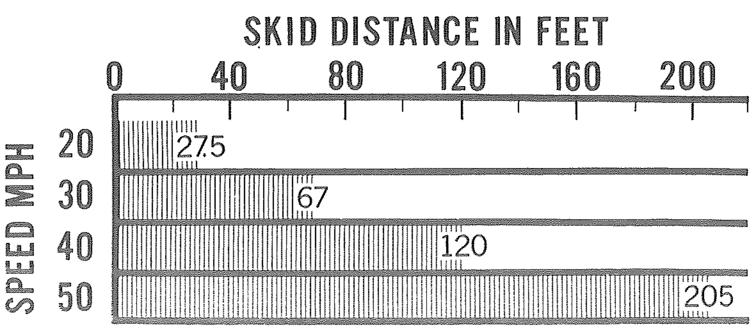
SKID NUMBER

	LT.	RT.
20 MPH	41	56
40 MPH	31	45

SITE NO. 4

DEGREES OF ROTATION AT





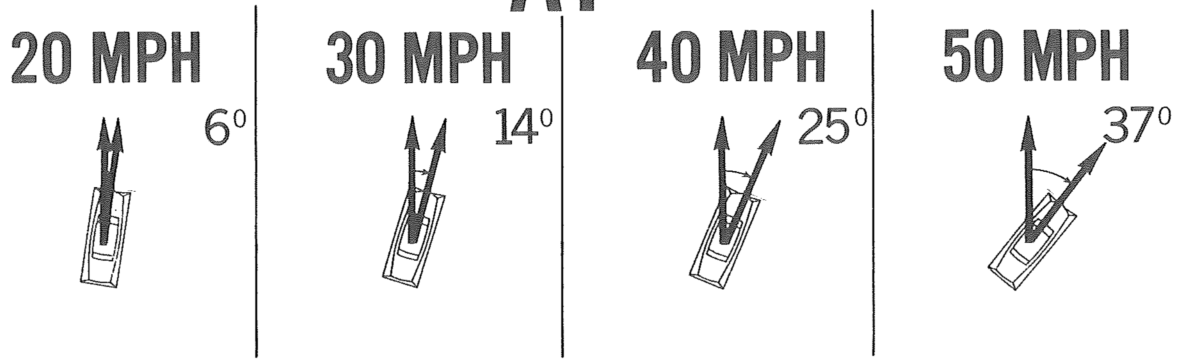
ROTATION IN °/FT: 0.48
DIFFERENTIAL 40 : 9

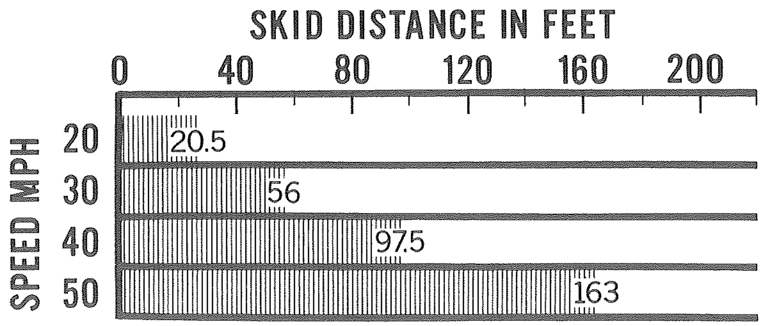
SKID NUMBER

	LT.	RT.
20 MPH	56	62
40 MPH	40	49

SITE NO. 5

DEGREES OF ROTATION AT





ROTATION IN °/FT = 0.26

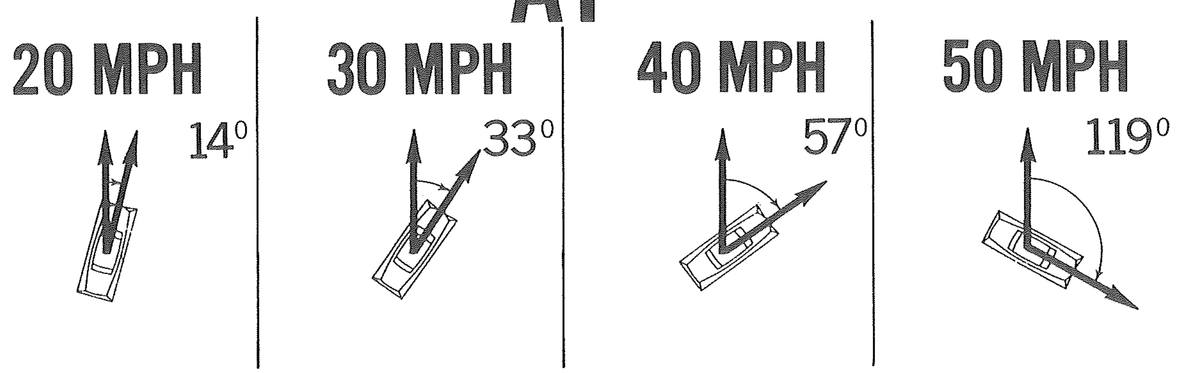
DIFFERENTIAL 40 = 9

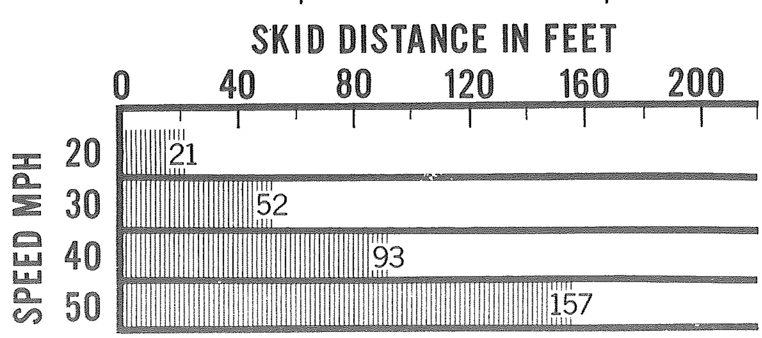
SKID NUMBER

	LT.	RT.
20 MPH	61	63
40 MPH	52	61

SITE NO. 6

DEGREES OF ROTATION AT





ROTATION IN °/FT = 0.62

DIFFERENTIAL 40 = 26

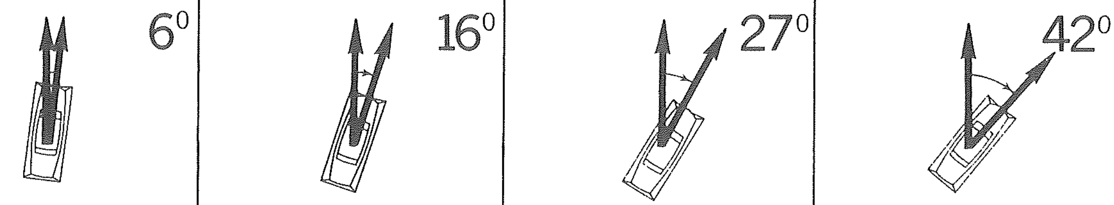
SKID NUMBER

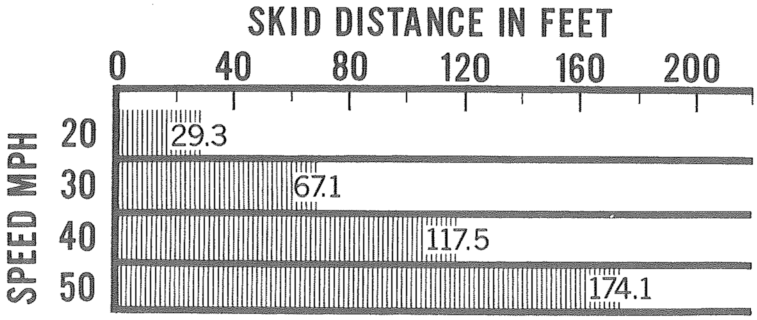
	LT.	RT.
20 MPH	51	76
40 MPH	47	73

SITE NO. 7

DEGREES OF ROTATION AT

20 MPH	30 MPH	40 MPH	50 MPH





ROTATION IN °/FT: 0.23
DIFFERENTIAL 40 : 6

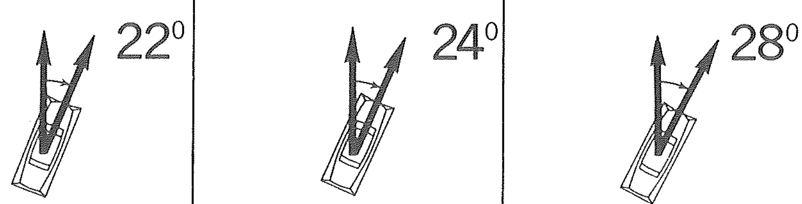
SKID NUMBER

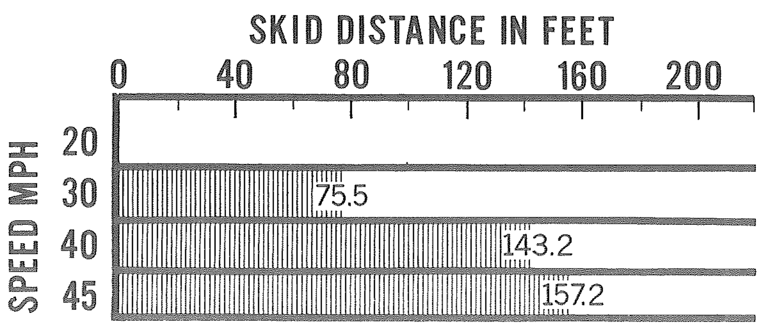
	LT.	RT.
20 MPH	42	44
40 MPH	42	48

SITE NO. 8

DEGREES OF ROTATION AT

20 MPH	30 MPH	40 MPH	45 MPH





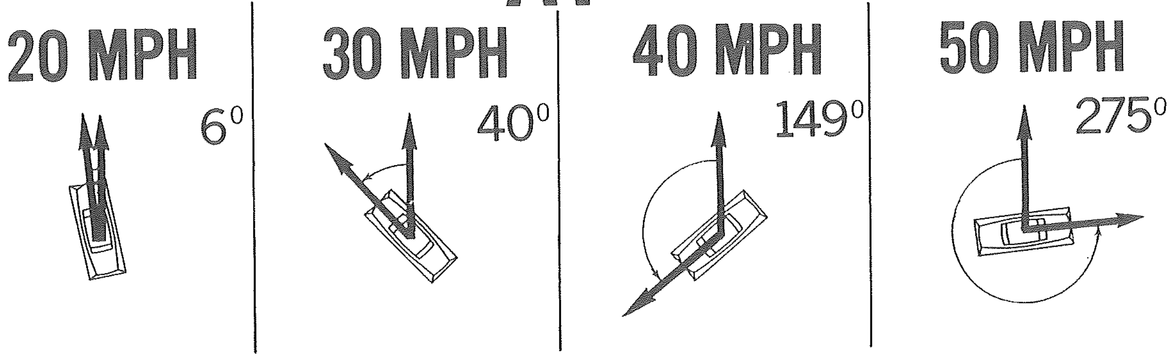
ROTATION IN °/FT: 0.17
DIFFERENTIAL 40 : 8

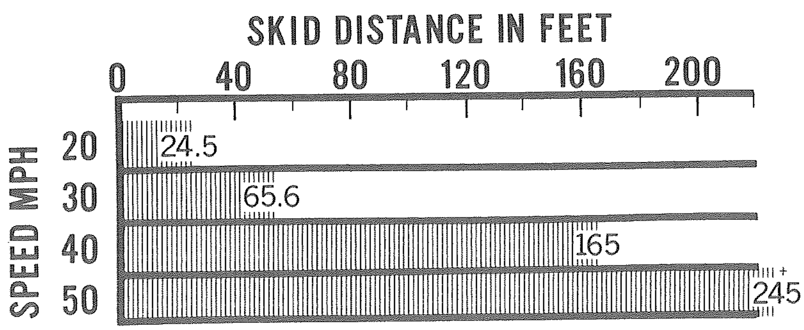
SKID NUMBER

	LT.	RT.
20 MPH	49	61
40 MPH	36	44

SITE NO. 9

DEGREES OF ROTATION AT





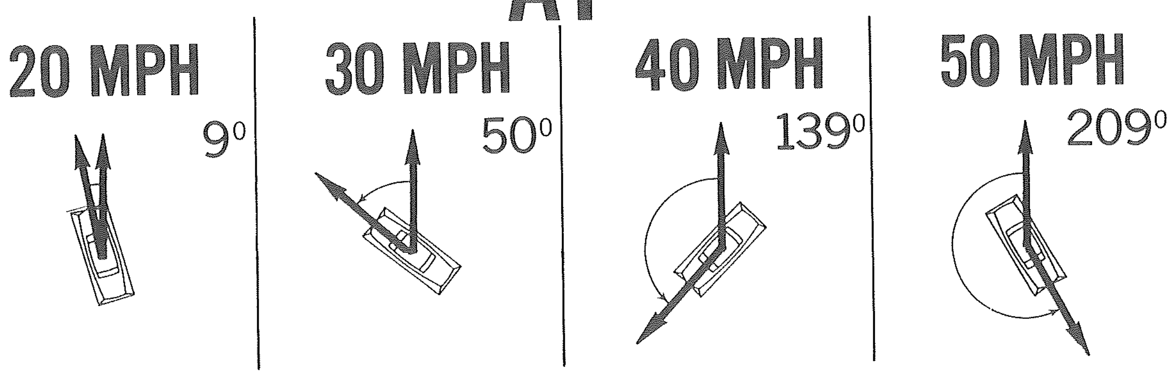
ROTATION IN °/FT = 0.90
DIFFERENTIAL 40 = 22

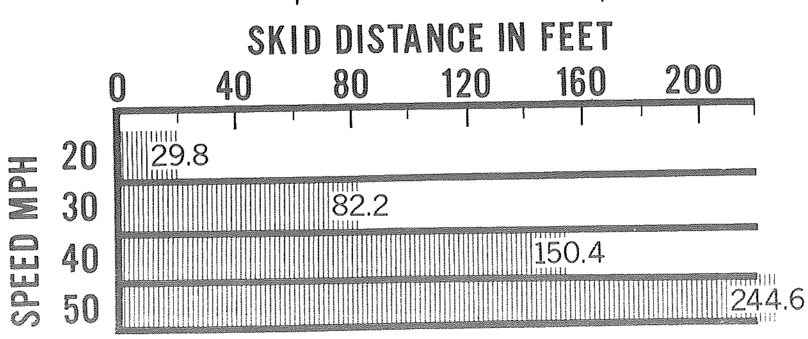
SKID NUMBER

	LT.	RT.
20 MPH	60	38
40 MPH	47	25

SITE NO. 10

DEGREES OF ROTATION AT





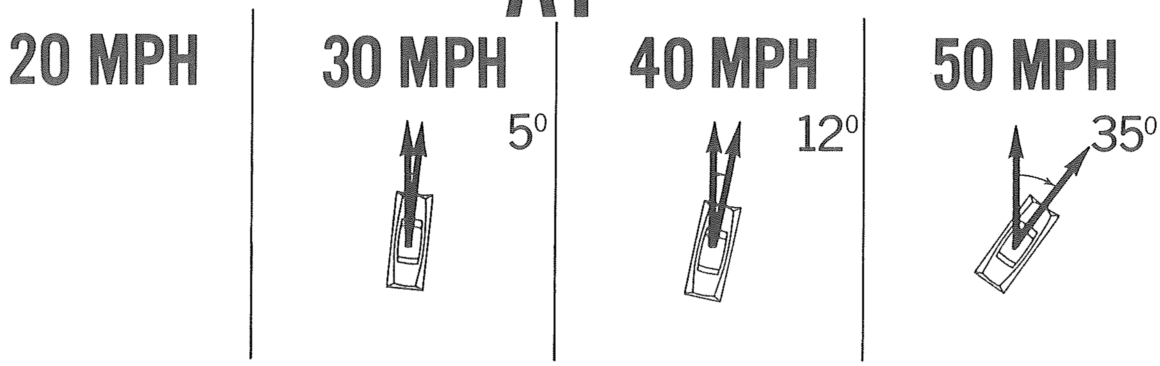
ROTATION IN °/FT = 0.92
DIFFERENTIAL 40 = 17

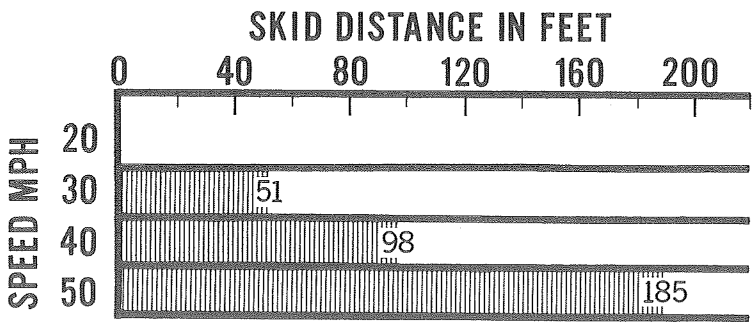
SKID NUMBER

	LT.	RT.
20 MPH	58	40
40 MPH	49	32

SITE NO. 11

DEGREES OF ROTATION AT





ROTATION IN °/FT = 0.12

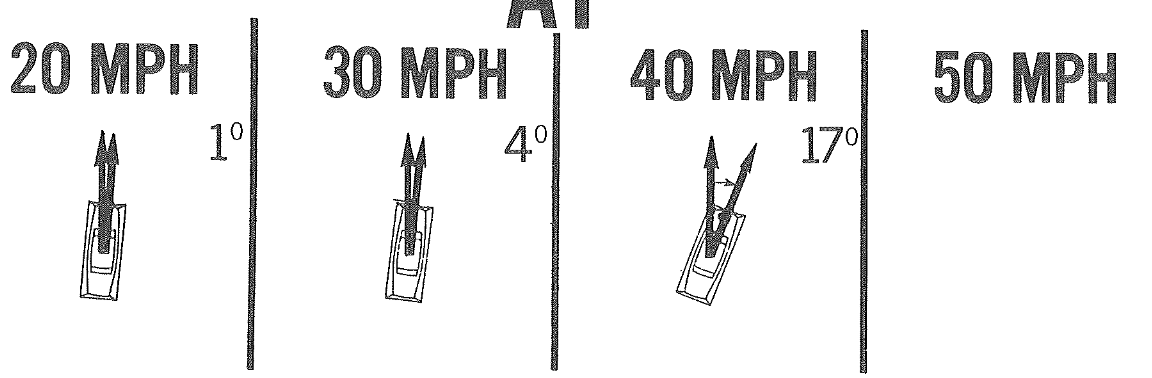
DIFFERENTIAL 40 = 4

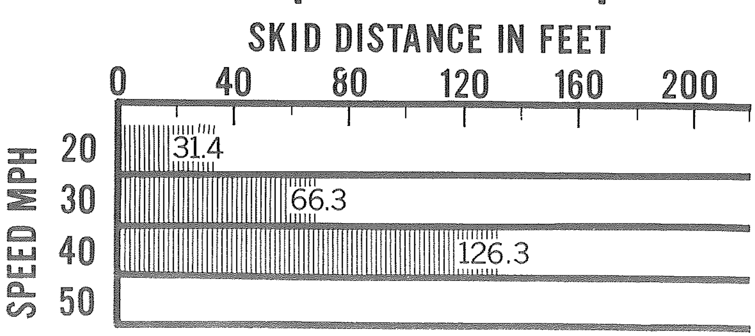
SKID NUMBER

	LT.	RT.
20 MPH		
40 MPH	53	57

SITE NO. 12

DEGREES OF ROTATION AT





ROTATION IN °/FT = .14

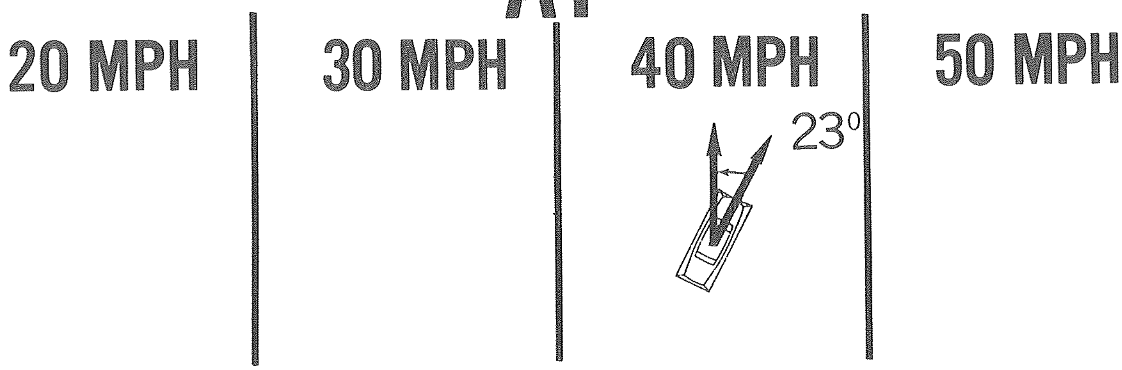
DIFFERENTIAL 40 = 4

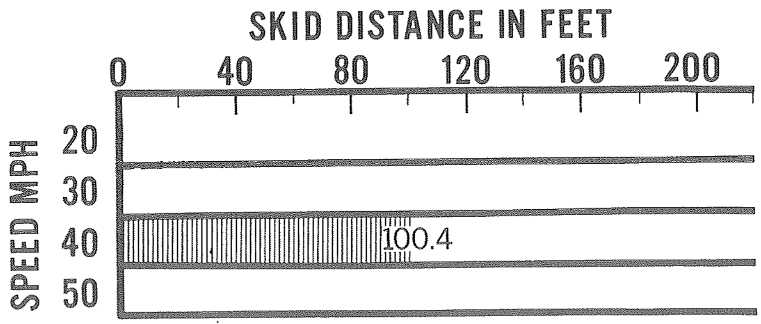
SKID NUMBER

	LT.	RT.
20 MPH	58	56
40 MPH	45	49

SITE NO. 13

DEGREES OF ROTATION AT





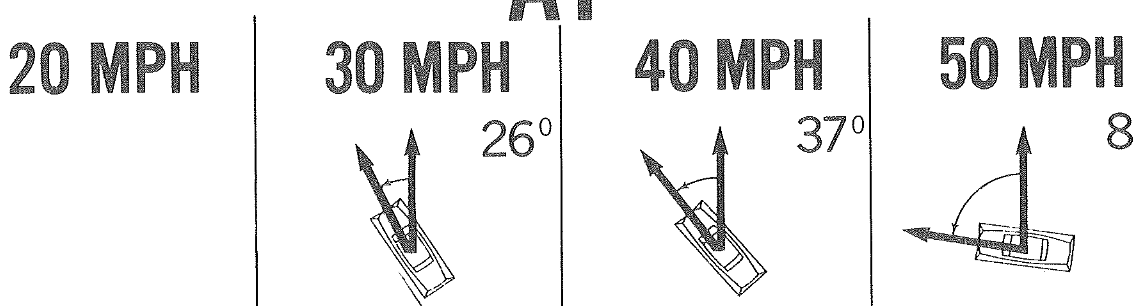
ROTATION IN °/FT: .23
DIFFERENTIAL 40 : 11

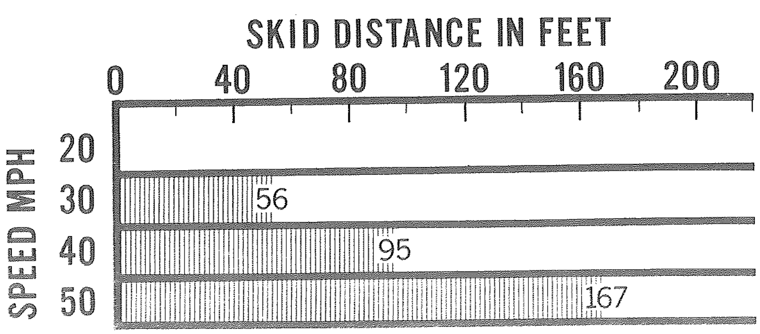
SKID NUMBER

	LT.	RT.
20 MPH		
40 MPH	60	49

SITE NO. 14

DEGREES OF ROTATION AT





ROTATION IN °/FT: 0.39
DIFFERENTIAL 40 : 12

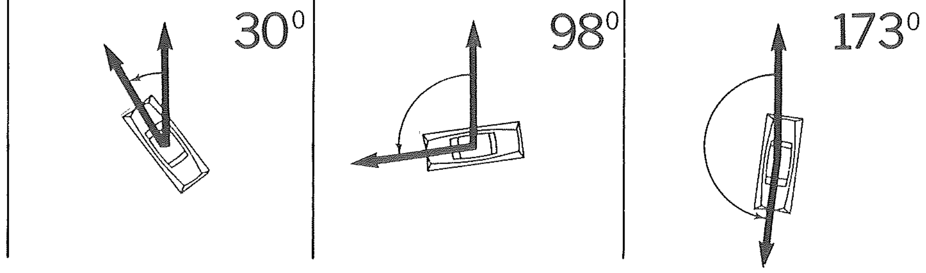
SKID NUMBER

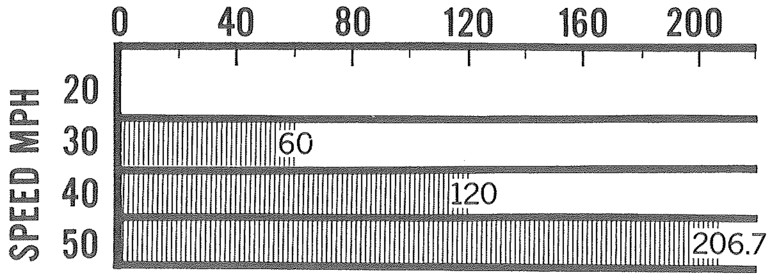
	LT.	RT.
20 MPH		
40 MPH	58	46

SITE NO. 15

DEGREES OF ROTATION AT

20 MPH	30 MPH	40 MPH	50 MPH





ROTATION IN °/FT = 0.82
DIFFERENTIAL 40 = 19

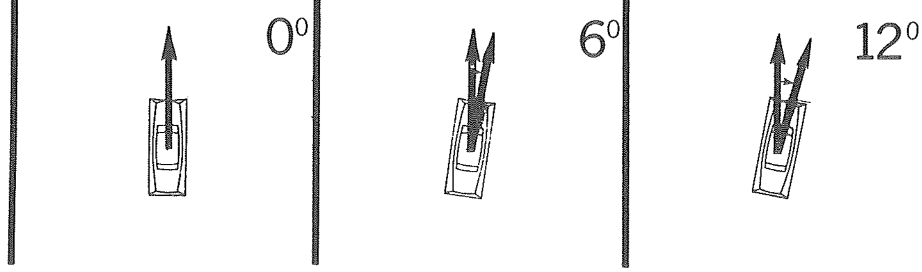
SKID NUMBER

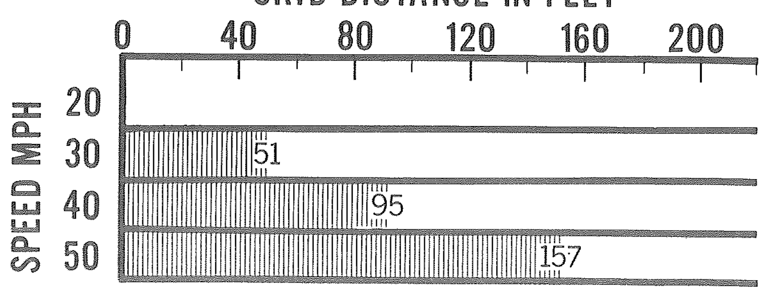
	LT.	RT.
20 MPH		
40 MPH	57	38

SITE NO. 16

DEGREES OF ROTATION AT

20 MPH	30 MPH	40 MPH	50 MPH





ROTATION IN °/FT = .06
DIFFERENTIAL 40 = 4

SKID NUMBER

	LT.	RT.
20 MPH		
40 MPH	51	55

APPENDIX B

APPENDIX B

No. I 17-2 (35) Cordes Junction - Flagstaff (Munds Park - Flagstaff Airport)

Gradation:

Fine Aggregate:

Passing 3/8" Sieve	100%
Passing No. 4 Sieve	95 - 100%
Passing No. 16 Sieve	45 - 80%
Passing No. 50 Sieve	10 - 30%
Passing No. 100 Sieve	2 - 10%
Passing No. 200 Sieve	0 - 4%

Coarse Aggregate No. 1		Specification Change
Passing 2-1/2" Sieve	100%	
Passing 2" Sieve	95 - 100%	
Passing 1-1/2" Sieve	35 - 70%	50 - 80%
Passing 1" Sieve	1 - 15%	5 - 20%
Passing 1/2" Sieve	0 - 5%	

Coarse Aggregate No. 2

Passing 1-1/2" Sieve	100%
Passing 1" Sieve	95 - 100%
Passing 1/2" Sieve	25 - 60%
Passing No. 4 Sieve	0 - 10%
Passing No. 8 Sieve	0 - 5%

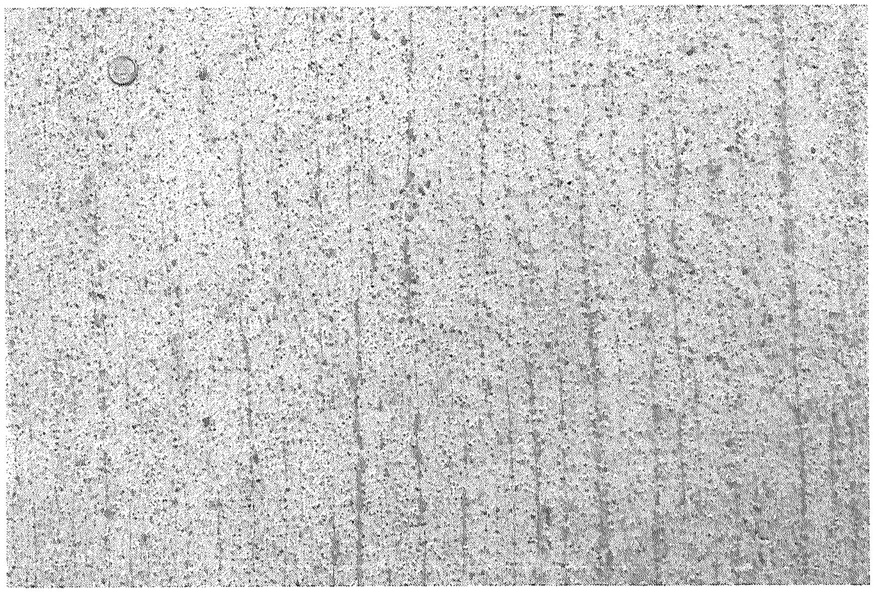
Composition of Coarse Aggregates No. 1 and No. 2

Passing 2-1/2" Sieve	100%
Passing 2" Sieve	95 - 100%
Passing 1" Sieve	35 - 70%
Passing 1/2" Sieve	10 - 10%
Passing No. 4 Sieve	0 - 5%

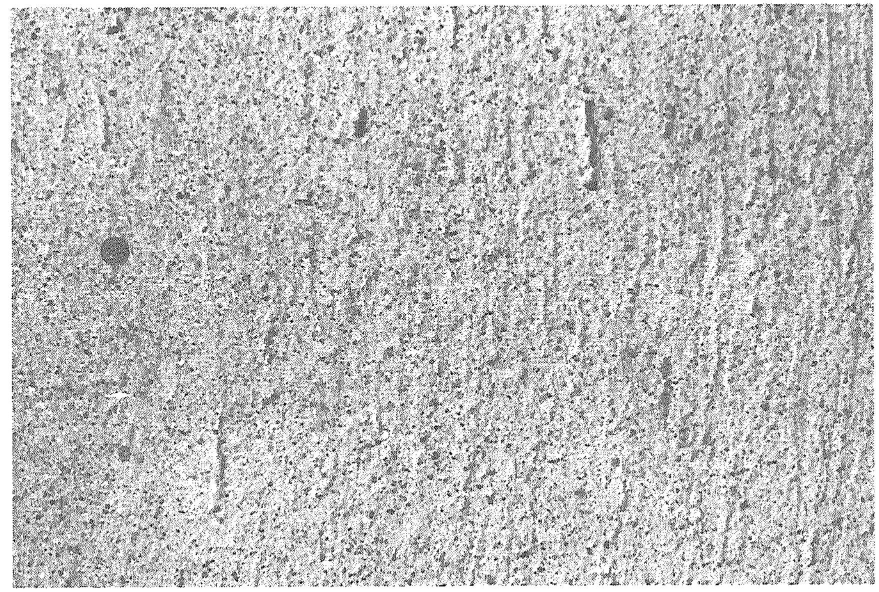
The 1-1/2" and 1" screen specification for coarse aggregate No. 1 were changed about the middle of the project, however, the composite coarse aggregate was to conform to the requirements indicated above.

APPENDIX C

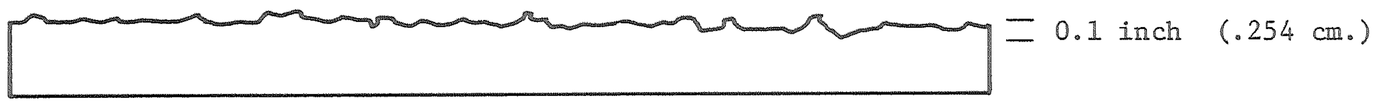
NORMAL BURLAP DRAG TEXTURE



11-7-74



6-17-75

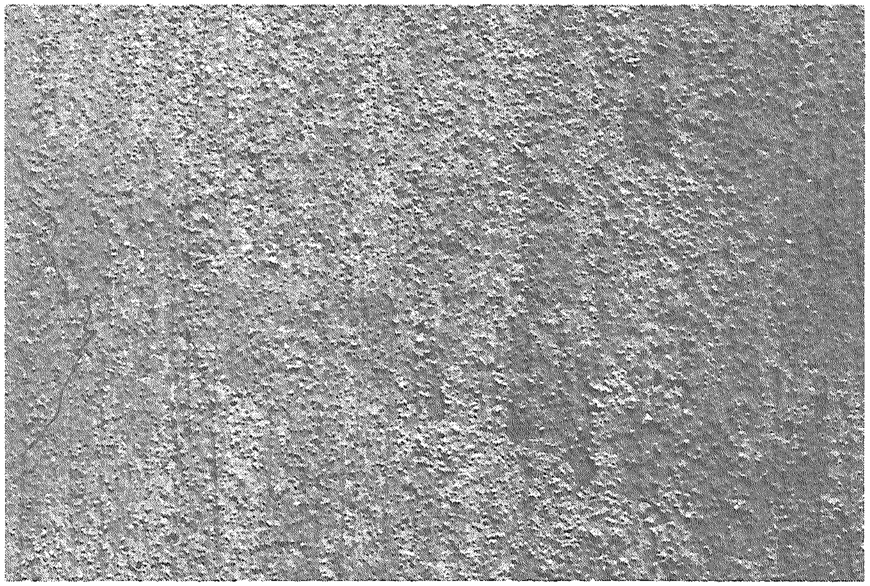


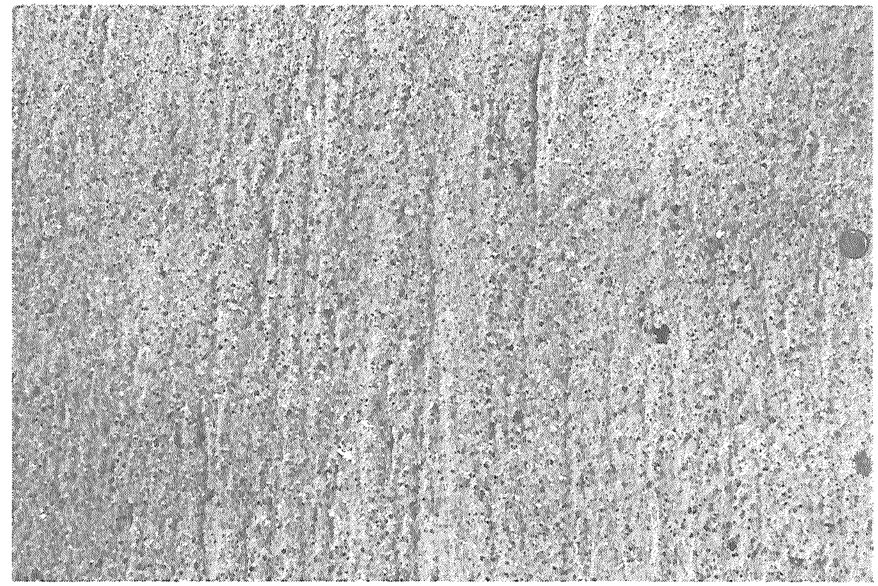
Typical Original Cross Section

Original Average Texture Depth --	High	Low
	.051 in.	.037 in.
	.123 cm.	.094 cm.

Figure C-1

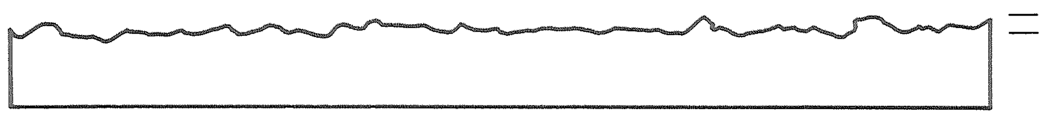
BURLAP DRAG TEXTURE (4 ply, 10 oz (283.5g))





11-7-74

6-17-75



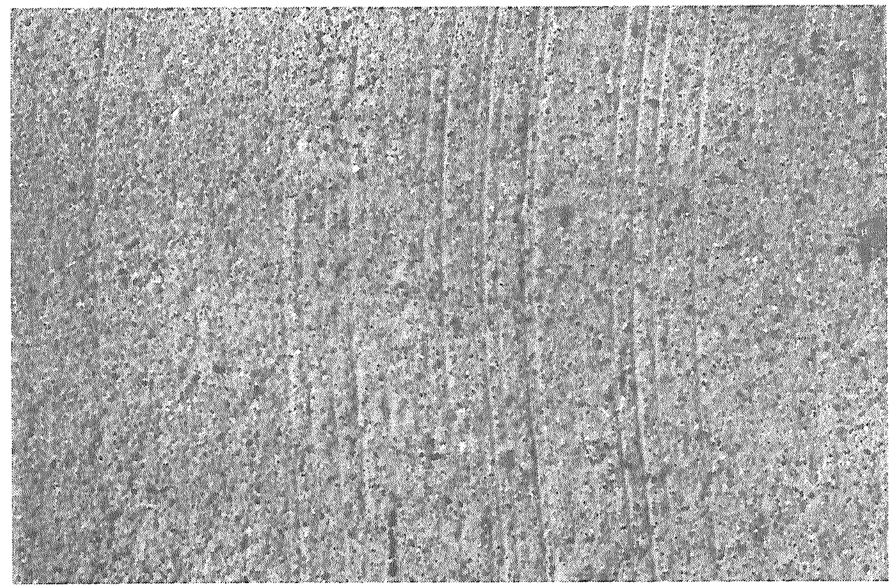
Typical Original Cross Section

Original Average Texture Depth --	High	Low
	.036 in.	.028 in.
	.091 cm.	.071 cm.

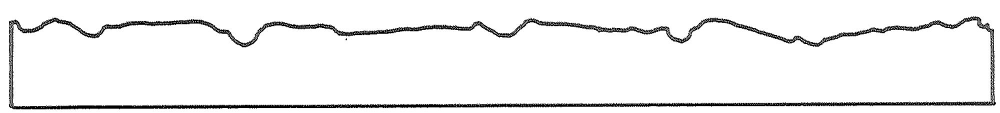
Figure C-2

BURLAP DRAG WITH NAILS TEXTURE

11-7-74



6-17-75

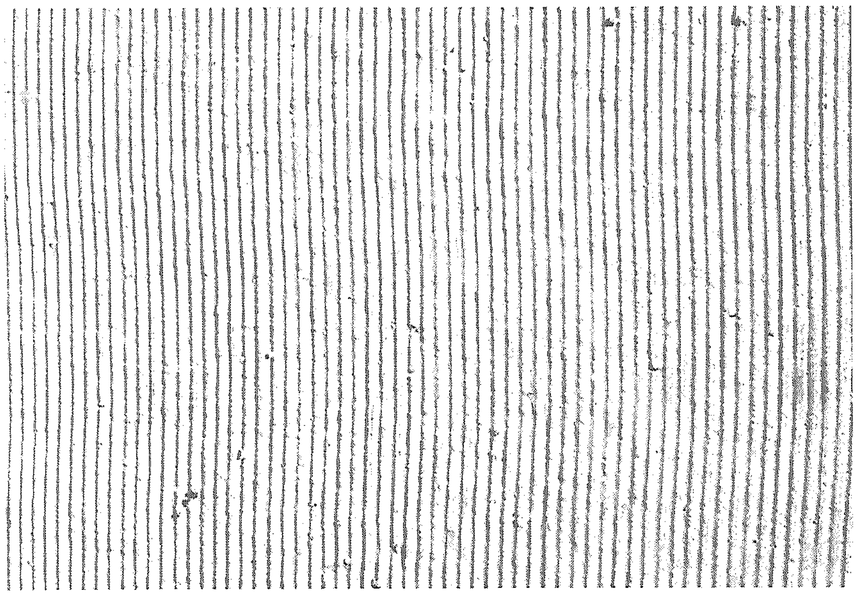


Typical Original Cross Section

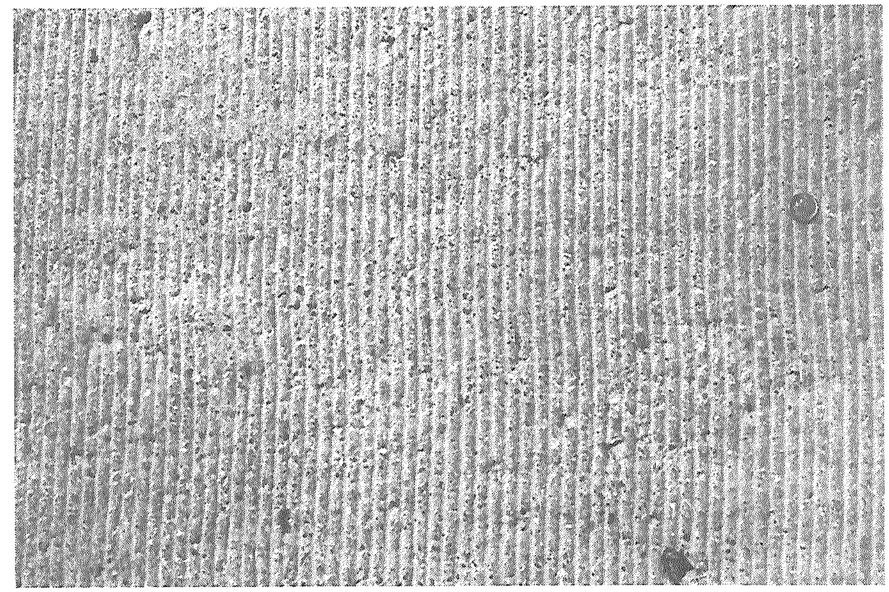
Original Average Texture Depth --	High	Low
	.056 in.	.036 in.
	.142 cm.	.091 cm.

Figure C-3

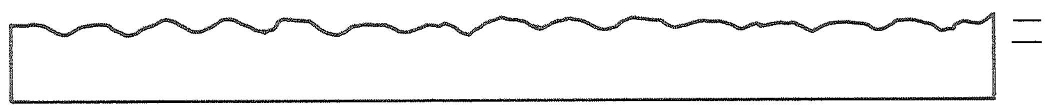
FLUTED FLOAT TEXTURE



11-7-74



6-17-75

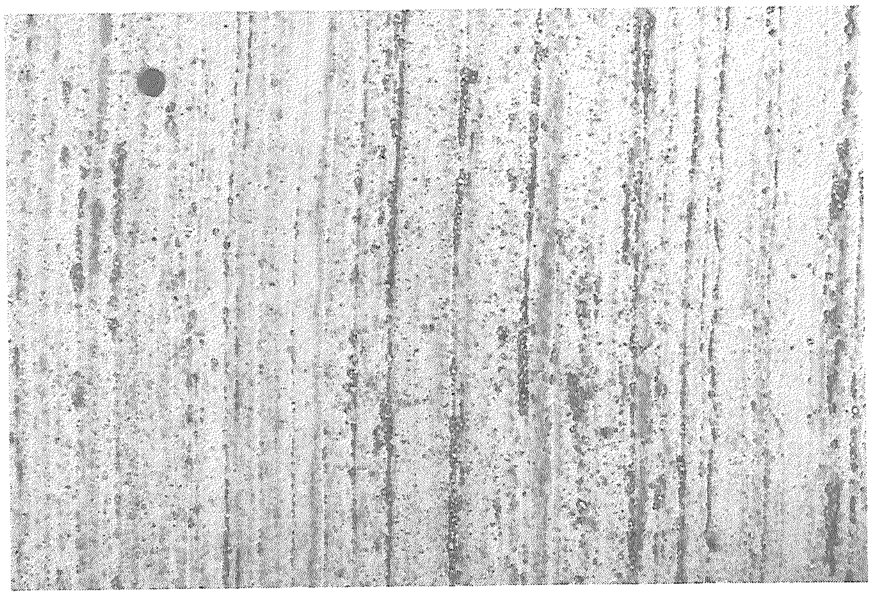


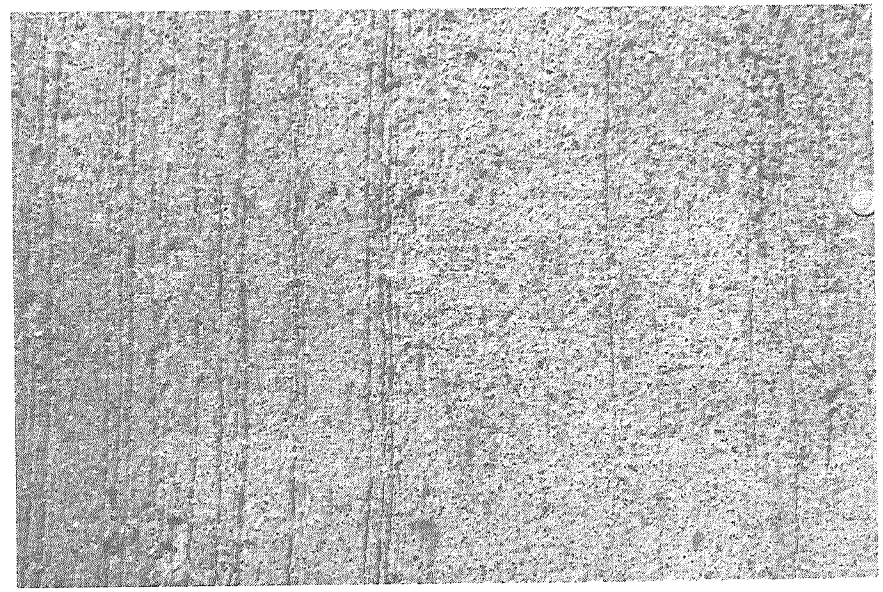
Typical Original Cross Section

Original Average Texture Depth --	High	Low
	.041 in.	.036 in.
	.104 cm.	.091 cm.

Figure C-4

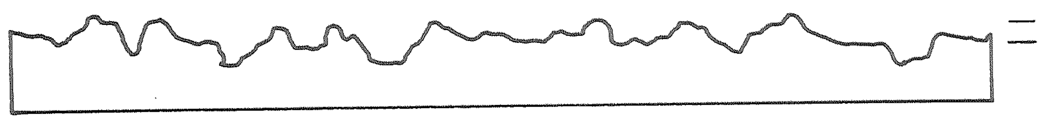
WIRE BROOM TEXTURE





11-7-74

6-17-75

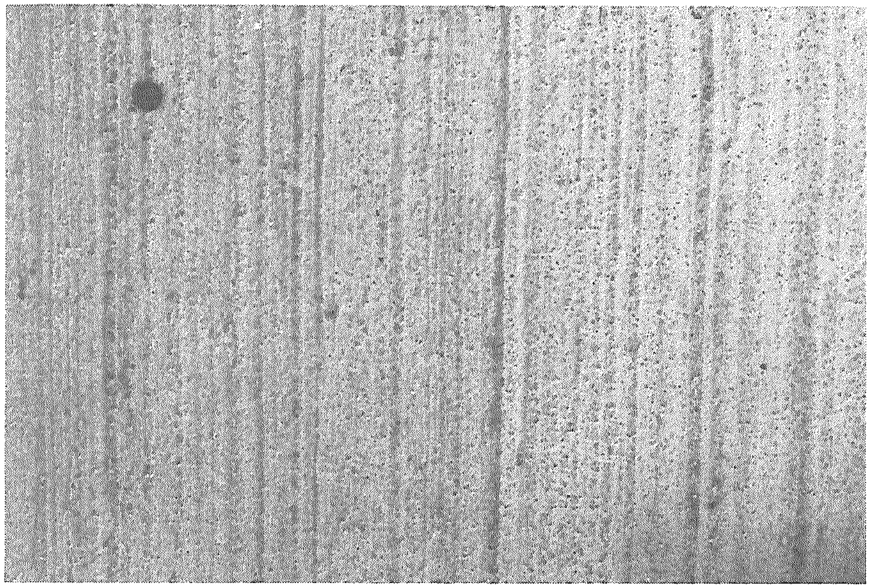


Typical Original Cross Section

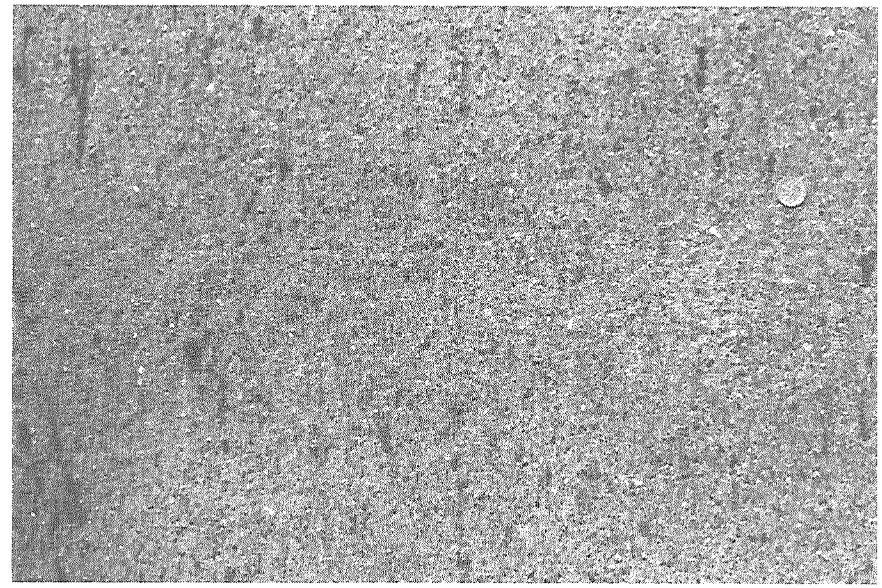
Original Average Texture Depth -- High	Low
.078 in.	.069 in.
.198 cm.	.175 cm.

Figure C-5

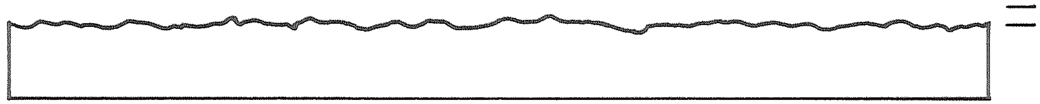
BURLAP DRAG TEXTURE (Threads Removed)



11-7-74



6-17-75

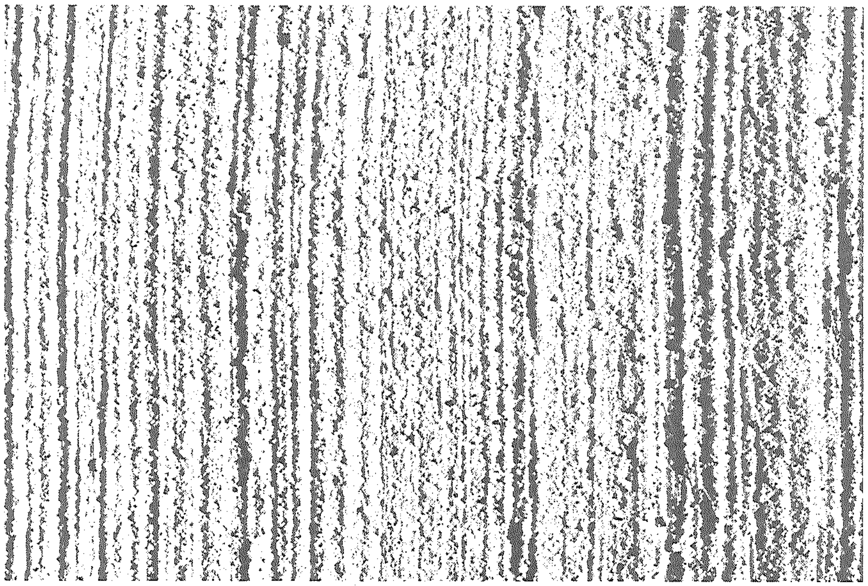


Typical Original Cross Section

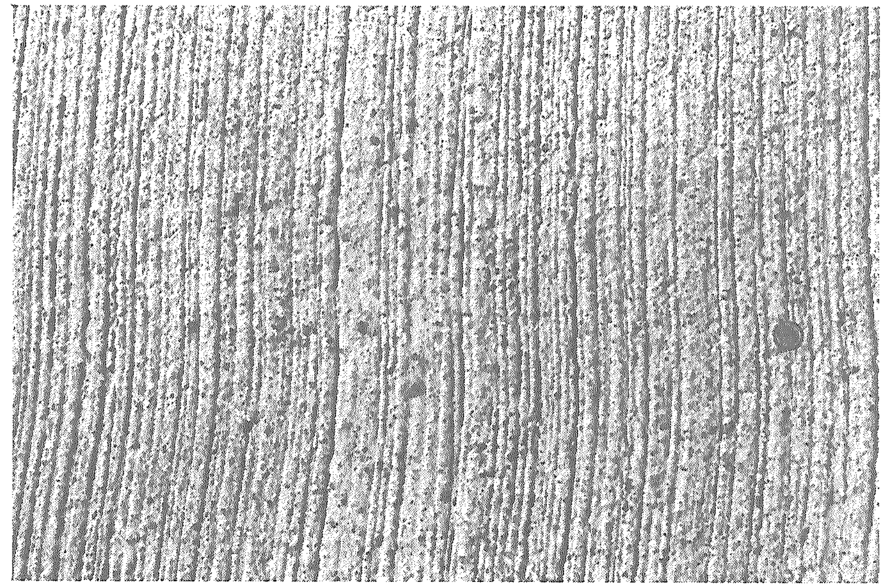
Original Average Texture Depth --	High	Low
	.041 in.	.035 in.
	.104 cm.	.089 cm.

Figure C-6

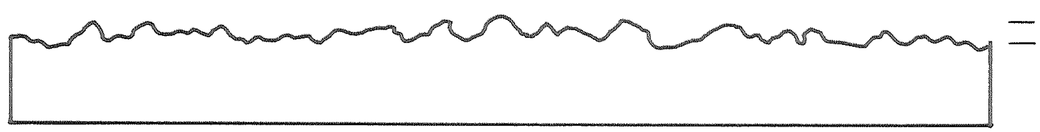
NYLON BRISTLE BROOM TEXTURE



11-7-74



6-17-75



Typical Original Cross Section

Original Average Texture Depth --	High	Low
	.096 in.	.060 in.
	.244 cm.	.152 cm.

Figure C-7