STATE OF THE ART RELATED TO SAFETY CRITERIA FOR HIGHWAY CURVE DESIGN

by

John C. Glennon

Research Report Number 134-4

Highway Design Criteria

Research Study Number 2-8-68-134

Sponsored by

The Texas Highway Department
In Cooperation With the
U. S. Department of Transportation
Bureau of Public Roads

November 1969

TEXAS TRANSPORTATION INSTITUTE
Texas A&M University
College Station, Texas

FOREWORD

This report is one phase of Research Study No. 2-8-68-134 entitled "An Examination of the Basic Design Criteria as They Relate to Safe Operation on Modern High Speed Highways." Other phases of research within the scope of this study are: (1) an evaluation of design criteria for critical lengths of grade and truck climbing lanes; (2) an evaluation of passing sight distance design criteria; and (3) an evaluation of stopping sight distance design criteria. Separate reports have been prepared for each phase of the study.

DISCLAIMER

ABSTRACT

An examination of the state of knowledge was conducted for the purpose of evaluating the validity of design criteria for horizontal highway curves. The evaluation was specifically concerned with the design equation (centripetal), assumed levels of tire-pavement side friction capability, safe side friction factors, maximum degree of curvature, maximum super elevation, and design factors of safety.

The evaluation was addressed to design criteria as presented in " A Policy on Geometric Design of Rural Highways, 1965," by the AASHO. The major findings were:

- It appears that minimum curve design standards (those employed by most state highway departments) do not provide an adequate factor of safety for the range in operational conditions encountered on our highways.

- The standard centripetal force equation is reasonably valid if the curve radius is large relative to the dimensions of the vehicle. It's validity has not been substantiated for curves greater than 4-degrees.

- The "typical" relationship between tire-pavement friction capability and vehicle speed employed by the AASHO Policy has no objective relation to actual highway conditions. Measurements made in one state indicate that only 55 percent of that state's pavements satisfy this "typical" friction capability level.

- The use of locked-wheel skid trailers to measure the side friction capability of a pavement is a questionable practice.

- The use of friction demand design values that correspond to that point at which side forces cause driver discomfort has no objective factor of safety relationship to the side friction capability of the tire-pavement interface.

- The AASHO Policy employs the explicit assumption that vehicles will follow the designed path of the highway curve with geometric exactness. This assumption does not account for corrective maneuvers that are occasionally found necessary when drivers have misjudged the degree of the highway curve.

- There are several other variables, not explicity designed for, that will reduce the assumed factor of safety.

- The report recommends upgrading minimum curve design standards on a provisional basis and concurs with the values recently adopted by the Texas Highway Department.

TABLE OF CONTENTS

SUMMARY

The study reported here was undertaken in response to an increasing concern by highway design engineers regarding the validity of current horizontal highway curve design standards. The report presents a comprehensive review of the current AASHO design standards (2) and an evaluation of these standards based on the existing state-of-the-art.

From the evaluation presented in the report, it appears that minimum curve design standards (those employed by most state highway departments) do not provide an adequate factor of safety for the range in operational conditions which are encountered on our highways. More specifically, the following findings may be drawn from the report:

1. The standard centripetal force equation employed as the basis for all highway curve design is reasonably valid if the curve radius is large relative to the dimensions of the vehicle. The report indicates that the equation is a relatively good predictive tool for highway curves of 4-degrees or less. For highway curves greater than 20-degrees, the equation appears to yield incorrectly low values of friction demand due to the inherent "point-mass" assumption. For the region between 4° and 20°, the equation explains less as the degree of curve increases. This relationship,

however, cannot be explained without further experimentation.

2. The "typical" relationship between tire-pavement friction capability and vehicle speed employed by the AASHO Policy has no objective relation to actual highway conditions. This relationship was taken from selected skid test measurements conducted in 1933. Measurements of 500 pavements randomly dispersed throughout one state, conducted in 1964, indicate that only 55 percent of that state's pavements satisfy this "typical" friction capability level.

The measurements discussed above refer to stopping friction capability. The report also discusses some inconclusive results of tests conducted to measure cornering friction capability. Although it is not readily apparent how cornering capability and stopping capability relate, if at all, it was surmised that the cornering friction capability (for a given tire-pavement combination and vehicle speed) could possibly be lower than the stopping friction capability.

There are several variables, not explicitly designed for, that will reduce the cornering friction capability dependent upon temporal conditions. Some of these are: (a) excessive water depth on the pavement; (b) excessive tire temperature; (c) faulty vehicle conditions, i.e., bald tires, low tire pressure, uneven tire pressures, uneven tire loads, faulty suspension, and poor wheel alignment; and (d) foreign material on the pavement such as snow, ice, oil, loose aggregate, or a heavy dust.

3. The use of friction demand design values that correspond to that point at which side forces cause driver discomfort has no objective factor of safety relationship to the cornering friction capabilities of the tire-pavement interface. The assumption in using this criterion is that very few drivers will corner with friction demands that are uncomfortable. This assumption does not account for corrective maneuvers that drivers find necessary when they have misjudged the degree of highway curve and find themselves encroaching on opposing lanes or on the highway shoulder.

The AASHO Policy employs the explicit assumption that vehicles will follow the designed path of a highway curve with geometric exactness. This is exemplified by the substitution of the highway curve radius for the vehicle path radius into the standard centripetal force equation. The report concludes the possibility of a vehicle traversing a more severe cornering maneuver, thereby increasing the friction demand above the design level. These friction demands may be quite large for corrective maneuvers, especially when a vehicle performs a curved path opposite to the geometry of the highway curve. In this case, the friction demand would be increased rather than decreased by the amount of the superelevation.

There are other variables, not explicitly designed for, that will increase the cornering friction demand above the design level. These are: (a) vehicle speed higher than design speed; (b) acceleration or braking of the vehicle; (c) short crest vertical curves; (d) high steering reversal rates; (e) gusty winds; and (f) severe cornering maneuvers due to over-reaction with power steering.

The findings of this research may be summarized as shown in Figure S-1. This graph treats the cornering situation as one of supply and demand. It shows that the safety margin assumed by AASHO is not always available on the road. For one state, the friction "supply" is not as great as that assumed, creating a margin for error which is relatively small. Any of the temporal conditions, previously mentioned, can lower the friction "supply" or increase the frictional "demand" so that the margin of error is reduced or eliminated.

Recommendations for Implementation

The report sufficiently substantiates that the current AASHO minimum curve design standards as previously employed by the Texas Highway Department are marginal. Although objective criteria cannot be established without further research, it is recommended that minimum curve design standards be upgraded on a provisional basis. The curve design standards presented in the recently published "Operations and Procedures Manual" (31) of the Texas Highway Department appear to adequately satisfy this provisional need. These values are shown in Table S-1.

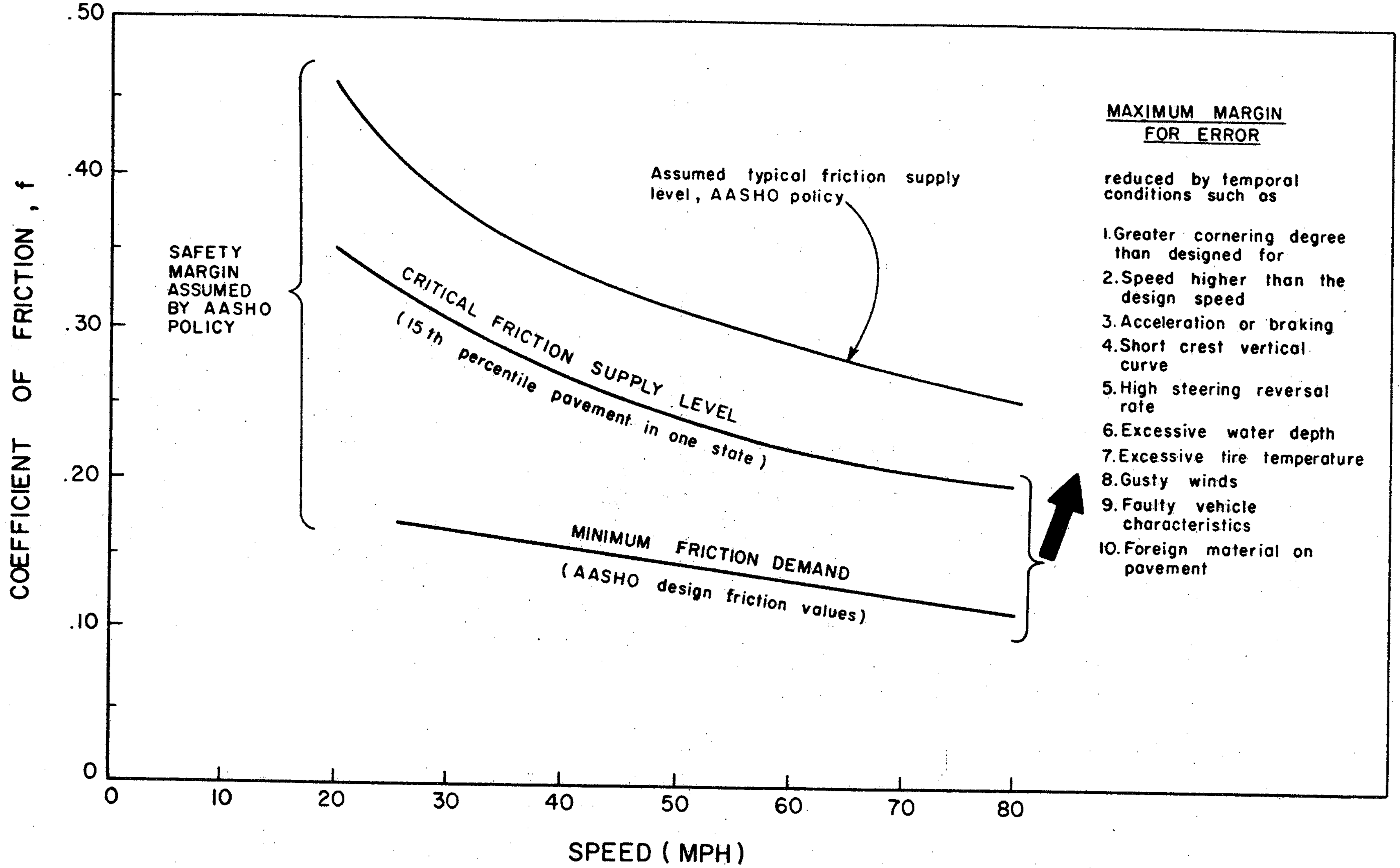


Figure S-1. Evaluation of the Factor of Safety Employed in Horizontal Curve Design.

TABLE S-1

TEXAS HIGHWAY DEPARTMENT
STANDARDS FOR CURVATURE (31)

DESIGN SPEED (MPH)	RECOMMENDED MAX. DEGREE OF CURVE[1]	ABSOLUTE MAX. DEGREE OF CURVE[2]	ABSOLUTE MIN. RADIUS (FEET)
30	1° 15	12° 00	480
40	1° 00	8° 00	715
50	0° 30	5° 00	1150
60	0° 25	3° 00	1910
70	0° 20	2° 00	2865
80	0° 15	1° 00	5730

1. Normal crown maintained without superelevation.

2. For general use (max. e = 0.06) - exceptions may be considered in case of unusual conditions.

Every effort should be made to exceed these minimum values. Minimum radii should be used only when the cost of realizing a higher standard is inconsistent with the benefits.

Recommendations for Further Research

Figure S-2 is a flow diagram which illustrates the technology necessary to arrive at an objective basis for horizontal curve design. Based on this process, the following information is required to develop objective criteria:

(1) Measurement of critical paths followed by vehicles on highway curves;

(2) Development of a critical friction demand model based on a sophisticated vehicle dynamics model (such as the TTI single vehicle computer simulation model);

(3) Development of a realistic method for measuring the cornering friction capability of a tire-pavement combination.

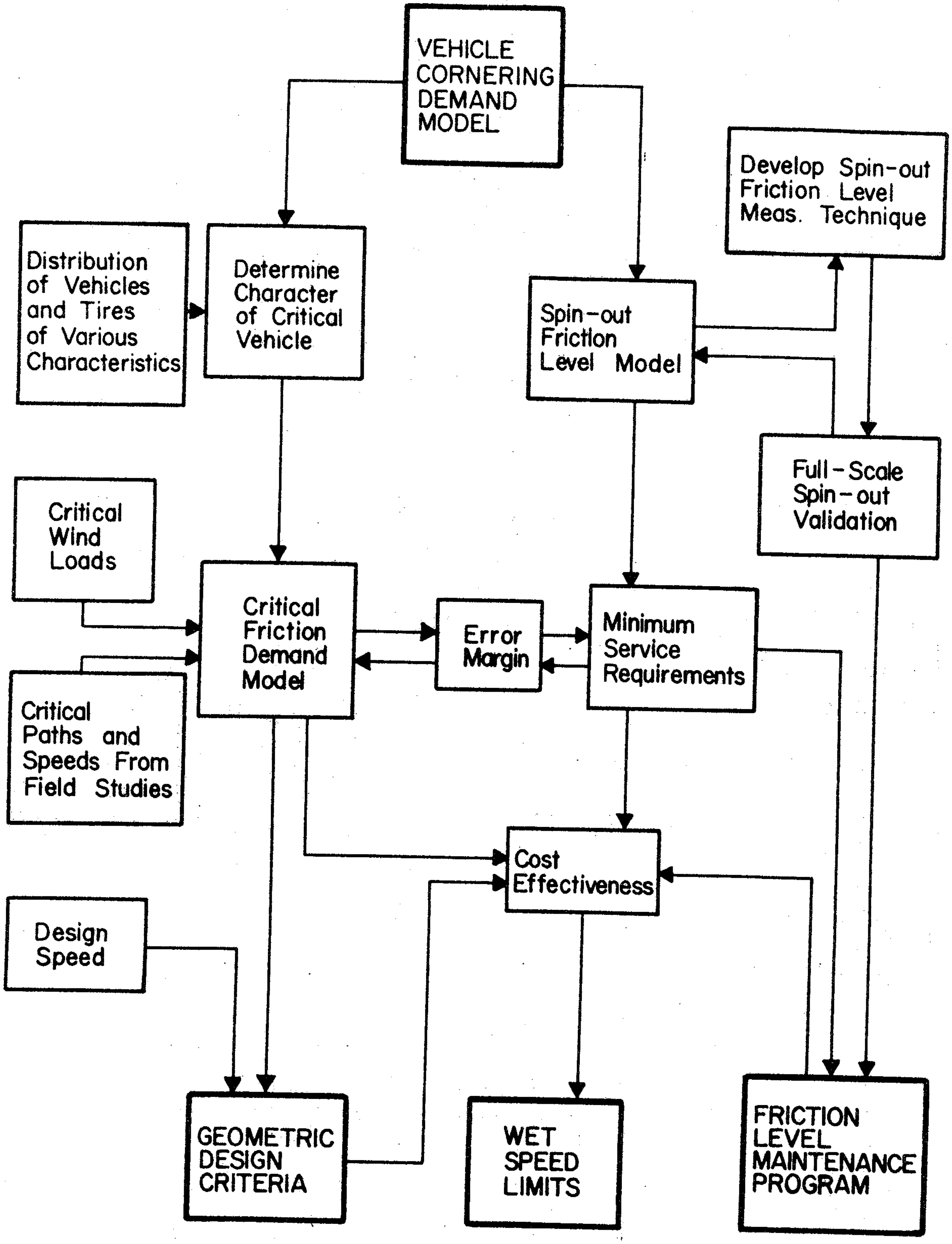


Figure S-2. Technology Necessary for an Objective Basis in Designing Horizontal Curves.

A. INTRODUCTION

Slippery pavements have been known to exist for many years, but the causes of slipperiness, its measurement, and its effect on the safety of vehicular traffic were not regarded with great concern prior to 1950. Although reliable skidding accident data are difficult to obtain, those in existence suggest that the skidding accident rate is increasing and has reached proportions which may no longer be ignored. This trend may be partly attributed to improved accident reporting, but it is also undoubtedly a reflection of increased vehicle speeds and traffic volumes. (1)*

More rapid acceleration, higher travel speeds, and faster deceleration made possible by modern highway and vehicle design have raised the frictional demands placed on the tire-pavement interface because larger forces are required to maintain the vehicle on an intended path. On the other hand, for wet pavements, the frictional capability of the tire-pavement combination decreases with increasing speed (1). In addition, higher traffic volumes, speeds, and cornering requirements promote pavement wear and thereby increase the time rate of degradation in pavement frictional capability. Figure 1 illustrates how these parameters interact to produce a higher loss of control potential.

The upward trend of vehicle speeds and traffic volumes will undoubtedly continue through the next decade. Therefore, the skidding problem will become a more serious limitation to safe high-speed

---

*(1) denotes number of reference listed in the Bibliography

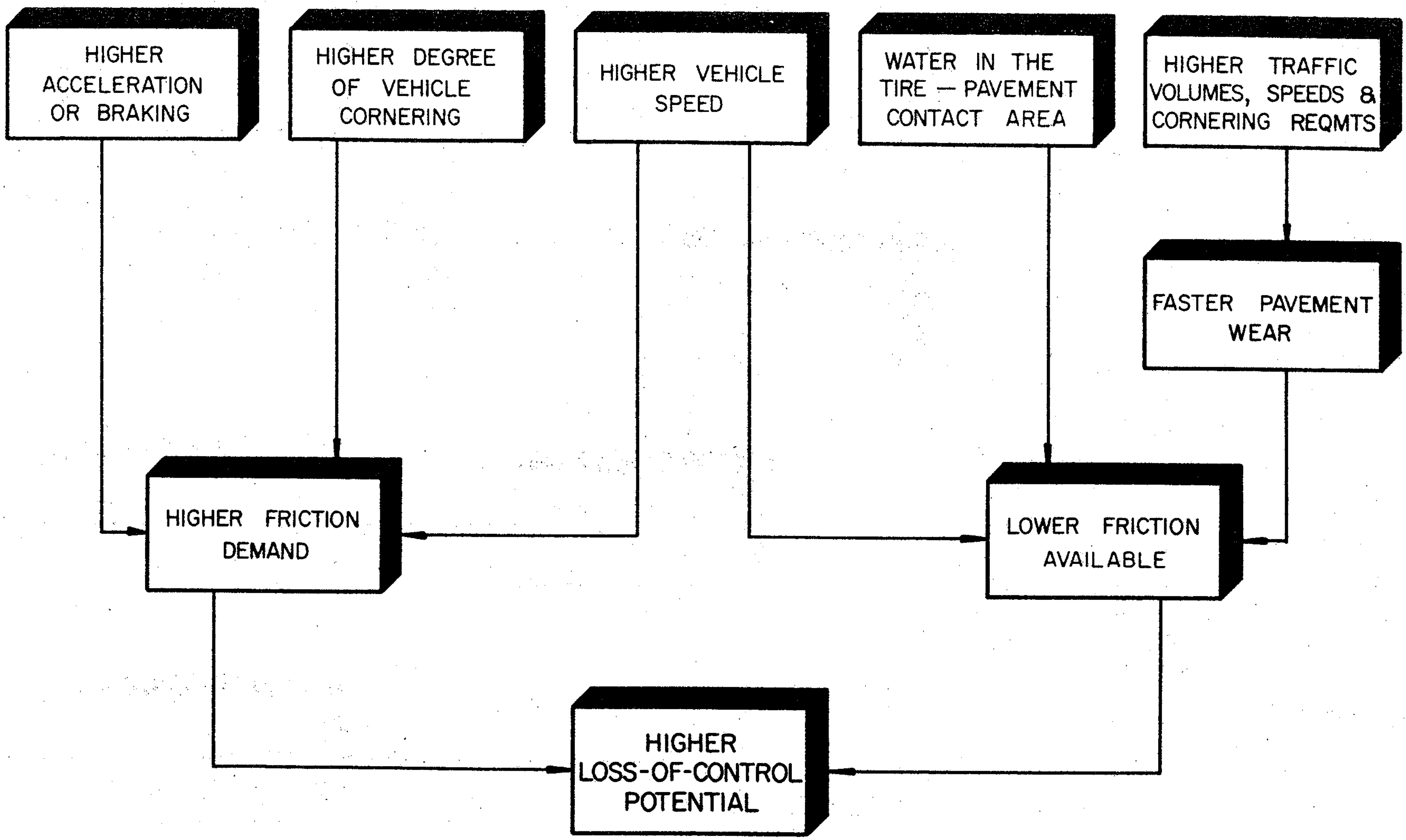


Figure 1. Circumstances Leading to Higher Loss of Control Potential.

travel on wet highways. From the technological standpoint, the slipperiness problem appears amenable to solutions which either reduce the frictional demand (improved geometric design, and adoption of different speed limits for dry and wet conditions) or improve the frictional capability (improved pavement surface design and maintenance procedures, improved tire design, and improved inspection procedures that identify and correct faulty vehicle conditions).

In terms of highway safety, there has been an increasing concern by highway and traffic engineers regarding the validity of the basic criteria that are fundamental to current geometric design standards. A review of references reveals that most of the data which led to the establishment of these criteria were developed from 20 to 35 years ago.

The design standards for highway curves in AASHO's "A Policy on Geometric Design of Rural Highways, 1965" (2) are based on studies conducted between 1933 and 1940. As such, they may no longer be representative because vehicle, roadway, and driver characteristics have changed. In addition, there are uncertainties regarding the assumptions employed in establishing safe side friction factors, maximum superelevation rates, and maximum curvature.

This research study is addressed to an evaluation of the validity of the AASHO Policy's safety criteria for highway curve design. The method of study employs a comprehensive review of current highway curve design standards and an evaluation of their validity based on an analysis of the existing state-of-the-art.

B. THE VEHICLE CORNERING PHENOMENON

When a vehicle moves in a circular path, it requires side friction on the tires to maintain that path. Physically, there is a minimum radius of curvature that a vehicle can negotiate at any given speed. A sharper turn cannot be held because the tires will not develop enough centripetal force to provide the necessary radial acceleration.

Application of the pertainent laws of mechanics yields the centripetal force equation (see derivation in Appendix A) of the cornering vehicle as:

$$f = \frac{V^2}{15R} - e \qquad (1)$$

Minimum cornering friction capability of tire-pavement combination required to prevent sliding.	=	Frictional demand of the cornering vehicle.

This equation is a very close, but conservative, approximation of the precise derivation

$$f = \frac{V^2}{15R}(1-ef) - e \qquad (2)$$

where

f = side friction factor, dimensionless
V = vehicle speed, mph
R = radius of vehicle maneuver, feet
e = superelevation rate, feet per foot

Equation 1 was originally employed for the design of railroad curves. Its first widespread recognition as a basis for highway curve design was established in "A Policy on Intersections at Grade," published in 1940 by the AASHO (3). As indicated by "A Policy on Geometric Design for Rural Highways, 1965," by the AASHO, Equation 1 has continued to be the basis for horizontal highway curve design (2).

As discussed in Appendix A, the derivation of the standard cornering equation depends upon the assumption that all points in the vehicle have the same radial acceleration. This is equivalent to assuming that the vehicle has a "point mass". There is some uncertainty regarding the validity of this assumption. Spin-out tire tests (4) conducted by the Texas Transportation Institute (see Appendix B for analysis) indicated that, for a 20-degree curve, vehicles will generally spin-out at a significantly lower speed than that computed by the standard cornering equation (using the curve radius for R and friction factors measured with a locked-wheel skid trailer). This discrepancy could possibly be related to: (a) the assumption that cornering friction factors are equivalent to stopping friction factors; (b) the accuracy of the measured friction factors; or (c) the point-mass assumption. Because of the relatively small radius for these tests, it appears that the point-mass assumption accounted for a sizeable portion of difference between measured and computed spin-out friction factors.

Stonex and Noble (5) related measured side friction demands to those computed using the cornering equation. The research was conducted in 1940 on unopened sections of the Pennsylvania Turnpike, employing late model automobiles. The tests were run on curve radii ranging from 1,400 to 3,800 feet with vehicle speeds from 76 to 100 mph. Comparing the measured side friction demands (obtained from a recording brake decelerometer) with those computed (using the radius of highway curve for R in the standard cornering equation), fairly close agreement was obtained. For seventeen test runs, the difference between measured and computed values ranged from 0.006 to 0.069, with an average difference of 0.024.

From the above discussion, it appears that the point-mass assumption is reasonably valid if the curve radius is large relative to the dimensions of the vehicle. In other words, the assumption is questionable for curve radii less than 300 feet (20 degrees) but appears reasonably valid for curve radii in excess of 1,400 feet (4 degrees). Because of the lack of available data, no specific inferences may be drawn regarding the validity of the point-mass assumption for radii between 300 and 1,400 feet.

C. CORNERING FRICTION CAPABILITY

The side friction factor at which a cornering skid is imminent depends principally upon the speed of the vehicle, the degree of the cornering path, the condition of the tires, and the characteristics of the road surface. On wet pavements, vehicle speed is perhaps the most significant parameter not only because the frictional demand increases directly with the square of the speed but also because the frictional capability of the tire-pavement combination decreases with increasing speed (1). Figure 2 depicts a generalization of this relationship which illustrates for a given degree of cornering how the factor of safety against skidding decreases rapidly with increasing speed, until the skid is imminent.

The 1965 AASHO Policy in considering a typical cornering friction capability level assumed that cornering capability and stopping capability are equivalent. Major consideration was apparently given to studies conducted in 1933 by Moyer, as reported in Highway Research Bulletin 27 (6). Figure 3 is a reproduction of the graph of stopping friction measurements reported in Bulletin 27. The AASHO Policy states "for normal wet pavements and smooth tires the [friction] value is about 0.35 at 45 mph." Apparently, therefore, the AASHO Policy employed the stopping friction curve for "portland cement concrete-smooth tires," shown in Figure 3, as a typical cornering friction capability level, and extrapolated this curve for speeds higher than 40 mph.

Although there are many standardized methods for measuring the stopping friction capability of a tire-pavement combination, very

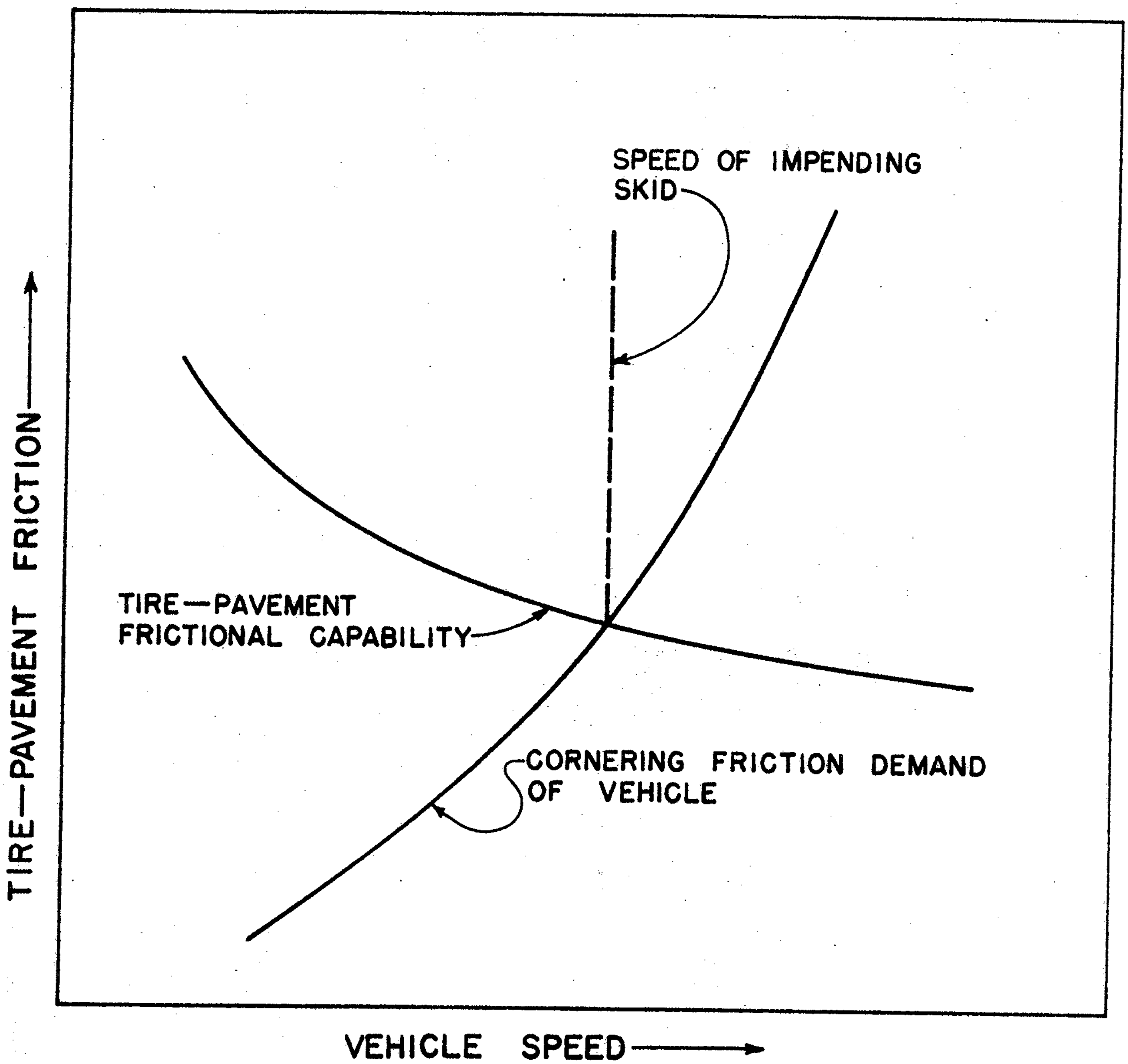


Figure 2. Friction Demand Relative to Friction Capability for Wet Pavements.

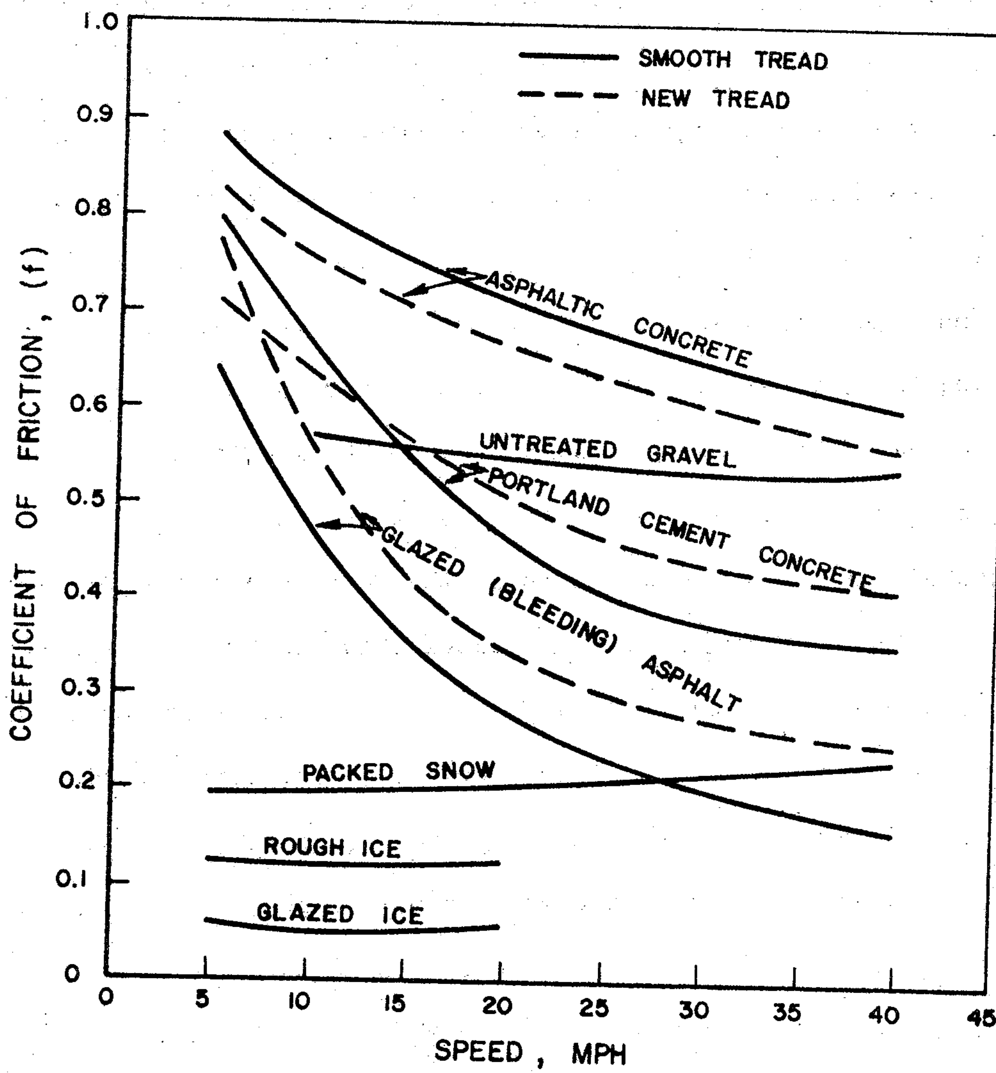


Figure 3. Frictional Capability Measurements on Wet Pavements (HRB Bulletin 27).

little has been done to establish a method for measuring cornering friction capability. In 1933, Moyer (7) conducted tests in which he attempted to measure cornering friction capability by dragging a skid trailer at a 15-degree angle. Moyer indicated that the cornering friction capability is somewhat higher than the stopping friction capability for a particular tire-pavement combination. Similarly, Kummer and Meyer (1) discussed the measurements on a single pavement indicating that, at higher speeds, a cornering slip tester yields higher values of friction capability than do the locked-wheel skid trailer or the locked-wheel stopping distance vehicle.

Unpublished tests (8) conducted by the Texas Highway Department show a very close agreement between values obtained by their standard skid trailer and by a cornering slip tester. On the other hand, the spin-out tire tests conducted by the Texas Transportation Institute (4) on 20-degree curves (see Appendix B) indicate that vehicles spin-out with less apparent cornering friction demand (computed using degree of curve and spin-out speed in standard cornering equation) than the friction capability measured with the locked-wheel skid trailer.

Summarizing the above, it is not readily apparent how the cornering friction capability and stopping friction capability relate, if at all. When a vehicle performs a cornering maneuver, there is a load transfer to the outer tires. It appears that this load transfer would reduce the cornering friction capability of the tire-pavement interface to a level below that measured by a skid trailer adapted to measure cornering slip.

To establish highway curve design requirements for safety, it is necessary to know the cornering friction capabilities of the tire-pavement combination in order to provide an adequate factor of safety against skidding. As stated above, the AASHO Policy utilized the stopping capability curve for "portland-cement concrete - smooth tires" shown in Figure 3 to represent a typical cornering friction capability. Assuming for the moment that the stopping friction capability and the cornering friction capability are the same, the validity of this curve as a typical representation is questionable. Figure 4 shows a percentile distribution of skid numbers (the relation being that Skid Number = 100f) at various speeds computed for a random sampling of 500 pavements in one state (9). These measurements were taken in 1964 employing a modified version of the ASTM standard trailer with standard ASTM test tires.

The typical cornering capability assumed by the AASHO Policy is also plotted on Figure 4. It is noted that about 45 percent of the pavements in this one state do not satisfy the typical capability level assumed by the AASHO Policy.

Figure 5 shows a similar percentile plot of skid numbers for 600 pavements in Germany (10).

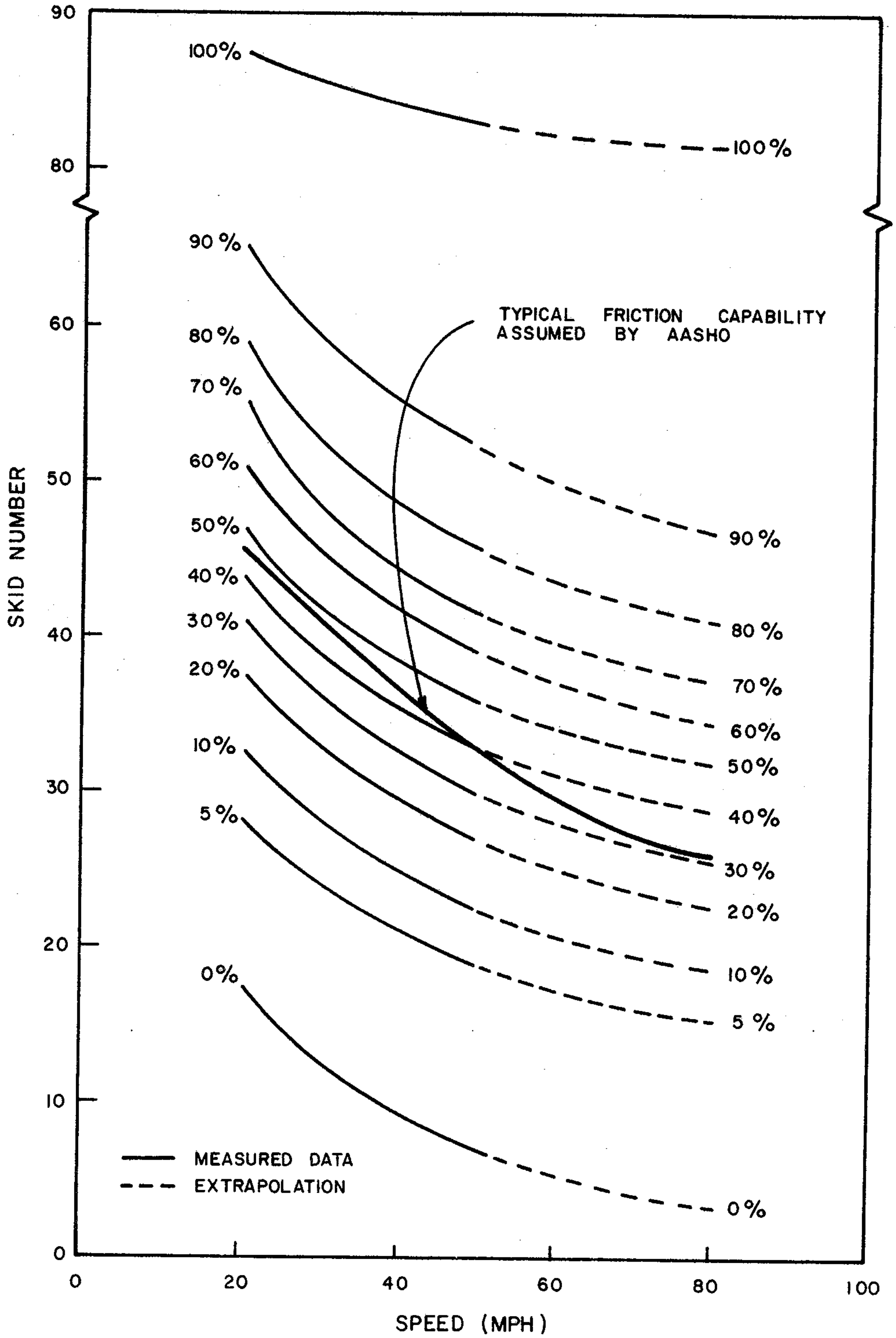


Figure 4. Percentile Distribution of Relation Between Friction Capability and Vehicle Speed for 500 Pavements in One State (9).

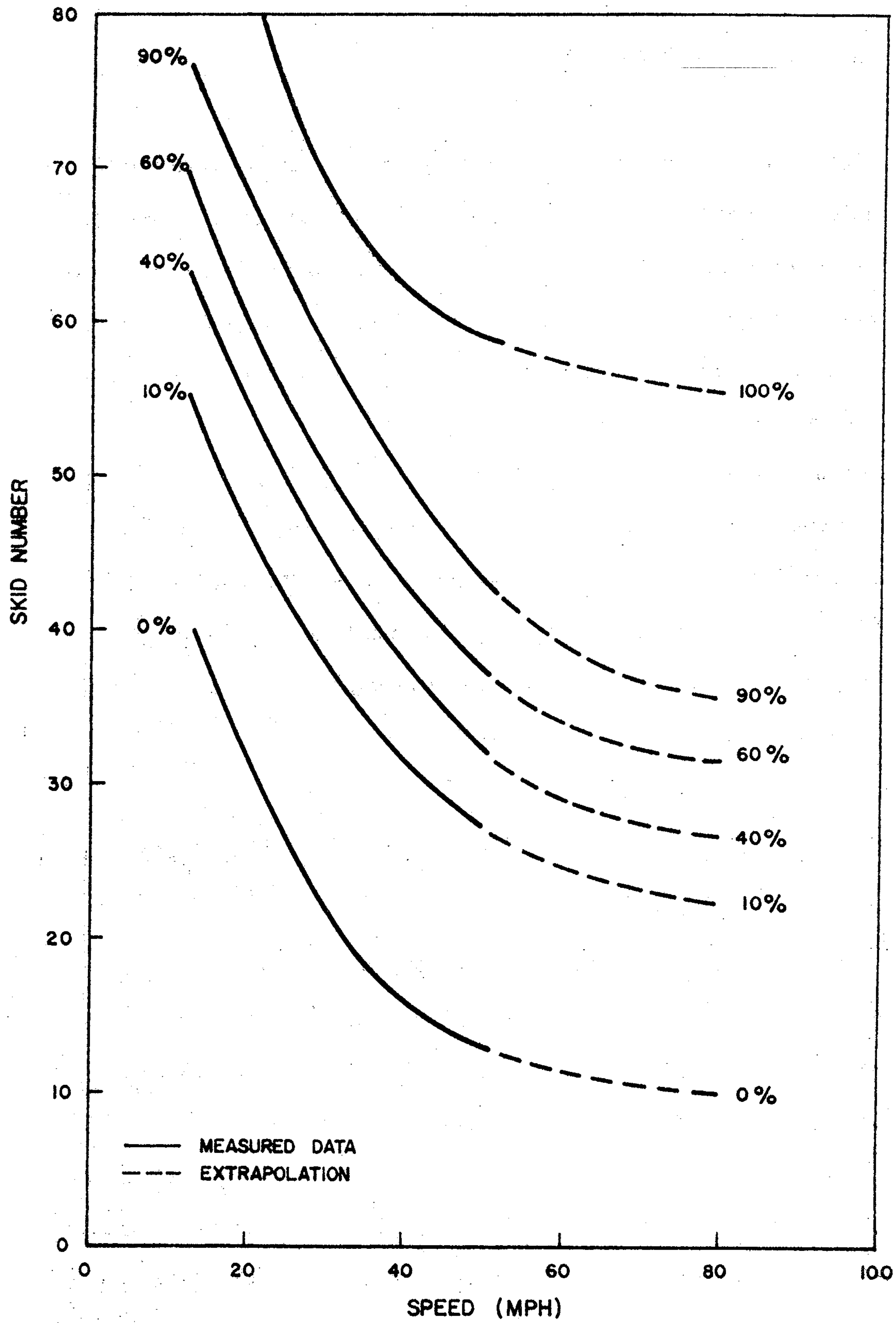


Figure 5. Percentile Distribution of Relation Between Friction Capability and Speed for 600 Pavements in Germany (10).

D. SAFE SIDE FRICTION FACTORS

According to the 1965 AASHO Policy, for a given curve radius, superelevation, and vehicle speed, the standard cornering equation yields the side friction demand by a cornering vehicle. As speed is increased for a particular curve design, the cornering friction demand increases until the frictional capability of the tire-pavement combination is reached and a cornering skid results.

Highway curves cannot, of course, be designed directly on basis of the maximum frictional capability of the tire-pavement combination. As in all engineering work, safety factors must be introduced. That portion of the frictional capability that can be utilized with safety by the vast majority of drivers then becomes the value for design. The AASHO Policy expands on this thought by stating:

> Values which are properly related to pavements that are largely deteriorated or poorly maintained - glazed, bleeding or oil slicked - should not control design because these conditions are avoidable and design should be based on acceptable structures attainable with reasonable cost.

Over the years, various researchers have recommended different values for safe side friction factors to be used in the design of highway curves. In a 1936 paper, Barnett (11) reported the results of 900 observations on highway curves. The speed at which the driver "felt a side pitch outward" was recorded and related to the side friction calculated using the standard cornering equation (the same as derived in Appendix A). Assuming that the minimum speed at which "side pitch is felt" is the maximum safe speed and employing a "best fit" curve of the recorded data, Barnett recommended a safe side friction factor of 0.16 for speeds up to 60 mph. For higher speeds, it was recommended to reduce this value by 0.01 for each 5-mph increment over 60 mph.

In selecting safe side friction factors for design, another criterion suggested by various authors is the point at which side force causes the driver to recognize a feeling of discomfort and act instinctively to avoid higher speed. The ball bank indicator has been widely used by research groups and highway departments as a uniform measure for the point of discomfort to determine safe speed on curves (2). With this device mounted in a moving vehicle, the reading at any time indicates the combined effect of body roll angle, side force angle, and superelevation angle. Moyer and Berry (12) reported analyses of the relation between these angles. As a result of their study, which also included an analyses of driver reaction to posted speeds on curves, the authors arrived at the following conclusions:

> To obtain the driver's respect for the speed on the sign over a wide range of speed, the following ball bank angles are recommended: 14° for speeds below 20 mph, 12° for speeds of 25 and 30 mph, and 10° for speeds of 35 mph and higher.
>
> For speeds up to 50 or even 60 mph, a ball bank angle of 10° has been found to be quite satisfactory, but for speeds above 60 mph a lower value should be used.
>
> The safe speed can be computed using the standard curve formula with a friction factor of f = 0.21 for speeds below 20 mph, f = 0.18, for speeds of 25 and 35 mph, and f = 0.15 for speeds of 35 mph and higher.

Meyer (13) derived a curvilinear relationship between speed and side friction factor ranging from 0.21 at 20 mph to 0.14 at 50 mph. He developed this relationship by employing (a) the recommended ball bank readings of Moyer and Berry (12), (b) the relationship between roll angle, side force angle, and superelevation angle derived by Moyer and Berry (8), and (c) the values of body roll angle reported by the General Motors Proving Ground (14).

The Arizona Highway Department prescribed a relationship similar to that derived by Meyer and noted that these side friction factors are the values at which comfort ends and discomfort begins (2). Kummer and Meyer (1) reported relationships used by the Montana Highway Department and the New Jersey Highway Department as shown in Figure 5. The explanation for the unusual relationship for New Jersey, is that they use the same minimum curve (1000 foot radius, 0.06 superelevation) for design speeds of 50 through 70 mph. In another study (5), high speed vehicle stability tests using 1940 automobiles led to a subjective conclusion that the side friction factor should not exceed 0.10 for design speeds of 70 mph and higher.

Figure 6 shows all of the relationships described above. In addition, Figure 6 also shows the AASHO (2) recommendation for design. The authors of the AASHO Policy considered the recommendations of references 5, 11, 12, and 13 and the Arizona practice in choosing their recommendations for design. In arriving at their design recommendations the authors of the Policy state:

> While some variation [between the five stated references] is noted, all are in agreement that the side friction factor for high speed design should be lower than for low speed design. A recommended straight line relation, shown solid, is superimposed on the analyses curves. It provides a reasonably good margin of safety at the higher speeds and gives somewhat lower rates for the low design speeds than

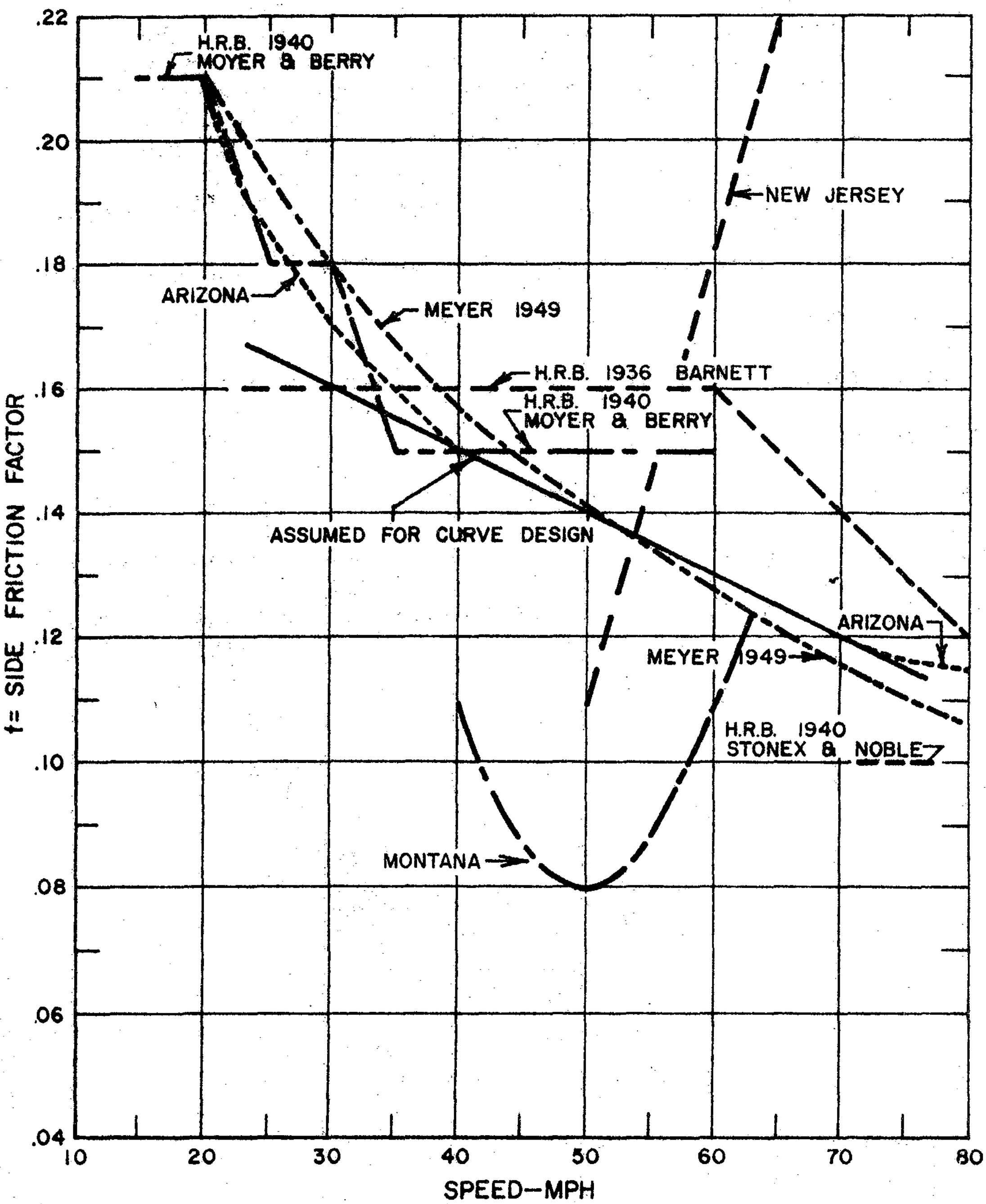


Figure 6. AASHO Policy's Assumed Safe Side Friction Factors and Recommendations From Other Sources

some of the other curves. The lower rates at the low speeds are desirable since drivers tend to over-drive low design speed highways. From the above data it is concluded that safe side friction factors for the use in highway curve design should be as shown by the solid straight line, varying directly with the design speed from 0.16 at 30 mph to 0.11 at 80 mph.

The use of friction demand levels that correspond to that point at which side forces cause discomfort to the driver has no objective factor of safety relationship with wet pavement cornering friction capabilities. It has not been proven that these values are the maximum demand levels accepted by most drivers. Further discussion of cornering friction demands will be presented in the last section of this report dealing with the factor of safety in horizontal curve design.

E. MAXIMUM SUPERELEVATION RATES

For a particular design speed, the maximum superelevation rate and the assumption for the safe side friction factor in combination determine the maximum curvature. The maximum rates of superelevation useable on highway curves are controlled by several factors: (a) climatic conditions, i.e., frequent snow and ice; (b) type of area, i.e., rural or urban; (c) frequency of slow-moving vehicles; and (d) terrain conditions, i.e., flat versus mountainous for drainage considerations. Consideration of these factors jointly has led to different conclusions by the various State Highway Departments as to maximum superelevation for design. The maximum superelevation rate for open highway in common use is 0.12. Where ice and snow are factors, experience has indicated that a superelevation rate of about 0.08 is a logical maximum to minimize slipping across the pavement when stopped or when attempting to gain momentum from a stopped position. Some agencies have adopted a maximum rate of 0.10, based on the avoidance of excessive outward friction forces required to drive slowly around the curve, a condition resulting in erratic operation. In urban areas, where it is difficult to warp crossing pavements for drainage without introducing negative superelevation for some turning movements, a maximum rate of 0.06 is commonly used. In summarizing the above considerations, the AASHO Policy concludes: (2)

> ...that (a) several rates rather than a single rate of maximum superelevation should be recognized in establishing design controls for highway curves, (b) a rate of 0.12 should not be exceeded, and (c) at the other extreme a rate of 0.06 is

applicable for urban design. Accordingly, four maximum rates - 0.06, 0.08, 0.10, and 0.12 - are used herein. Consistent with current practice, values for the 0.10 rate are referred to as generally desirable or nationally representative. For actual design use in a State or region, only one of the above maximum rates will apply, although there is no inhibition against the use of more than one, say for different road systems.

F. MAXIMUM DEGREE OF CURVATURE

The maximum degree of curvature, or the minimum radius, is a limiting value for a given design speed determined from the maximum rate of superelevation and the safe side friction factor. Use of sharper curvature for that design speed would call for superelevation beyond the limit considered practical, or for a cornering friction demand greater than that assumed safe, or both.

Minimum safe radius, R, is calculated directly from the standard centripetal force equation:

$$R = \frac{V^2}{15(e + f)}$$

Using D as the degree of circular curve (arc definition), D = 5729.6/R, the standard equation becomes:

$$D = \frac{85{,}900\ (e + f)}{V^2}$$

Employing these equations and the values for maximum superelevation and safe side friction presented, the AASHO Policy presents a table (Table III-5, AASHO Policy) for maximum degree of curve and minimum radius. These values are shown in Table 1 herein. It is important to note that although this table was not directly intended to be the recommended AASHO design policy, several State Highway Departments have adopted this table or a portion thereof as their basis for horizontal curve design. (15,16,17,18,19,20,21,22)

TABLE 1

MAXIMUM DEGREE OF CURVE AND MINIMUM RADIUS
DETERMINED FOR LIMITING VALUES OF e and f - AASHO Policy (2)

Design speed	Maximum e	Maximum f	Total (e+f)	Minimum radius	Max. degree of curve	Max. degree of curve, rounded
30	.06	.16	.22	273	21.0	21.0
40	.06	.15	.21	508	11.3	11.5
50	.06	.14	.20	833	6.9	7.0
60	.06	.13	.19	1263	4.5	4.5
65	.06	.13	.19	1483	3.9	4.0
70	.06	.12	.18	1815	3.2	3.0
75	.06	.11	.17	2206	2.6	2.5
80	.06	.11	.17	2510	2.3	2.5
30	.08	.16	.24	250	22.9	23.0
40	.08	.15	.23	464	12.4	12.5
50	.08	.14	.22	758	7.6	7.5
60	.08	.13	.21	1143	5.0	5.0
65	.08	.13	.21	1341	4.3	4.5
70	.08	.12	.20	1633	3.5	3.5
75	.08	.11	.19	1974	2.9	3.0
80	.08	.11	.19	2246	2.5	2.5
30	.10	.16	.26	231	24.8	25.0
40	.10	.15	.25	427	13.4	13.5
50	.10	.14	.24	694	8.3	8.5
60	.10	.13	.23	1043	5.5	5.5
65	.10	.13	.23	1225	4.7	4.5
70	.10	.12	.22	1485	3.9	4.0
75	.10	.11	.21	1786	3.2	3.0
80	.10	.11	.21	2032	2.8	3.0
30	.12	.16	.28	214	26.7	26.5
40	.12	.15	.27	395	14.5	14.5
50	.12	.14	.26	641	8.9	9.0
60	.12	.13	.25	960	6.0	6.0
65	.12	.13	.25	1127	5.1	5.0
70	.12	.12	.24	1361	4.2	4.0
75	.12	.11	.23	1630	3.5	3.5
80	.12	.11	.23	1855	3.1	3.0

G. AASHO STANDARDS FOR MAINLINE HORIZONTAL CURVES

In arriving at a design basis for combinations of curvature and superelevation, the AASHO Policy states that the superelevation should vary between zero and the maximum allowable as degree of curvature varies between zero and the maximum allowable for a given design speed. After a lengthy discussion of how this variation may be distributed, the AASHO Policy concluded that the relation should be a parabolic one. The result of this retionale is presented in four design tables in the AASHO Policy. These tables are reproduced in Tables 2 through 5 herein. From the State Highway Department design manuals available to the project staff, it was found that four States employ this design concept (23,24,25).

Considering the assumptions of the AASHO Policy that the standard cornering equation is valid and applicable and that the AASHO values for safe side friction factors are valid, it would appear that the values of Table 1 are the critical values for design. The rationale behind the AASHO curve design standards as presented in Tables 2 through 5 is not apparent. An interesting feature that occurs with these design tables is that the highway department which employs the higher maximum superelevation rate obtains a higher design factor of safety when using less than the maximum superelevation than the highway department which uses a lower maximum superelevation rate. For example, when employing Table 5 (max. e = 0.12) for a design speed of 70-mph, if 0.06 superelevation is used, the degree of curve assigned would be about 1.5-degrees; whereas, when employing Table 2 (max. e = 0.06) for a design speed of 70-mph, if 0.06 superelevation is used, the degree of curve assigned would be 3-degrees. Obviously then, the highway department employing the higher maximum superelevation rate would have a higher design factor of safety (a design f value of 0.03 compared with a value of 0.11 for the lower maximum superelevation rate).

These design tables do provide more conservative values (higher factors of safety against skidding) when considering superelevation less than the maximum rate. In other words, the less the superelevation the less the degree of curve and the less the side friction demand. However, if the assumed safe side friction factors were considered valid, it would appear that a more logical standard (one which would have universal application) would be to consider all superelevation rates up to the maximum and assign the appropriate maximum curvature to each superelevation rate. This standard could be extended to consider that flatter than maximum curves should be employed where feasible. This, in essence, is the standard employed by the several state highway departments mentioned earlier in this report.

TABLE 2

RATE OF SUPERELEVATION (e) AND MINIMUM LENGTH OF RUNOFF OR SPIRAL CURVE (L) FOR e max. = 0.06 - AASHO POLICY (2)

D	R	V=30 mph			V=40 mph			V=50 mph			V=60 mph			V=65mph			V=70 mph			V=75 mph			V=80mph		
		e	L-Feet		e	L-Feet		e	L-Feet		e	L-Feet		e	L-Feet		e	L-Feet		e	L-Feet		e	L-Feet	
			2-lane	4-lane		2-lane	4-lane		2-lane	4-lane		2-lane	4-lane		2-lane	4-lane		2-lane	4-lane		2-lane	4-lane		2-lane	4-lane
0° 15′	22918′	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	RC	240	240
0° 30′	11459′	NC	0	0	NC	0	0	NC	0	0	RC	175	175	RC	190	190	RC	200	200	.021	220	220	.023	240	240
0° 45′	7639′	NC	0	0	NC	0	0	RC	0	0	.021	175	175	.023	190	190	.026	200	200	.030	220	220	.033	240	240
1° 00	5730′	NC	0	0	RC	125	125	.020	150	150	.027	175	175	.029	190	190	.033	200	200	.037	220	220	.041	240	240
1° 30′	3820′	RC	100	100	.020	125	125	.028	150	150	.036	175	175	.040	190	190	.044	200	200	.050	220	240	.053	240	260
2° 00′	2865′	RC	100	100	.026	125	125	.035	150	150	.044	175	180	.048	190	210	.052	200	230	.057	220	270	.059	240	290
2° 30′	2292′	.020	100	100	.031	125	125	.040	150	150	.050	175	200	.053	190	230	.057	200	260	.060	220	290	.060	240	300
3° 00′	1910′	.023	100	100	.035	125	125	.044	150	160	.054	175	220	.057	190	250	.060	200	270	.060	220	290	.060	240	300
3° 30′	1637′	.026	100	100	.038	125	125	.048	150	170	.057	175	230	.059	190	250	.060	200	270	D max=2.5°			D max=2.5°		
4° 00′	1432′	.029	100	100	.041	125	130	.051	150	180	.059	175	240	.060	190	260	D max=3.0°								
5° 00′	1146′	.034	100	100	.046	125	140	.056	150	200	.060	175	240	.060	190	260									
6° 00′	955′	.038	100	100	.050	125	160	.059	150	210	D max=4.5°			D max=4.0°											
7° 00′	819′	.041	100	110	.054	125	170	.060	150	220															
8° 00′	716′	.043	100	120	.056	125	180	.060	150	220															
9° 00′	637′	.046	100	120	.058	125	180	D max=7.0°																	
10° 00′	573′	.048	100	130	.059	125	190																		
11° 00′	521′	.050	100	140	.060	130	190																		
12° 00′	477′	.052	100	140	.060	130	190																		
13° 00′	441′	.053	100	140	D max=11.5°																				
14° 00′	409′	.055	100	150																					
16° 00′	358′	.058	100	160																					
18° 00′	318′	.059	110	160																					
20° 00′	286′	.060	110	160																					
		.060	110	160																					
		D max=21.0°																							

emax=0.06

D—Degree of curve
R—Radius of curve
V—Assumed design speed
e—Rate of superelevation
L—Minimum length of runoff of spiral curve
NC—Normal crown section
RC—Remove adverse crown, superelevate at normal crown slope

Spirals desirable but not as essential above heavy line.

Lengths rounded in multiples of 25 or 50 feet permit simpler calculations.

TABLE 3

RATE OF SUPERELEVATION (e) AND MINIMUM LENGTH OF RUNOFF OR SPIRAL CURVE (L) FOR e max. = 0.08 - AASHO POLICY (2)

D	R	V=30 mph			V=40 mph			V=50 mph			V=60 mph			V=65 mph			V=70 mph			V=75 mph			V=80 mph		
		e	L-Feet 2-lane	L-Feet 4-lane	e	L-Feet 2-lane	L-Feet 4-lane	e	L-Feet 2-lane	L-Feet 4-lane	e	L-Feet 2-lane	L-Feet 4-lane	e	L-Feet 2-lane	L-Feet 4-lane	e	L-Feet 2-lane	L-Feet 4-lane	e	L-Feet 2-lane	L-Feet 4-lane	e	L-Feet 2-lane	L-Feet 4-lane
0° 15'	22918'	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	RC	240	240
0° 30'	11459'	NC	0	0	NC	0	0	NC	0	0	RC	175	175	RC	190	190	RC	200	200	.022	220	220	.024	240	240
0° 45'	7639'	NC	0	0	NC	0	0	RC	150	150	.022	175	175	.025	190	190	.029	200	200	.032	220	220	.036	240	240
1° 00'	5730'	NC	0	0	RC	125	125	.021	150	150	.029	175	175	.033	190	190	.038	200	200	.043	220	220	.047	240	240
1° 30'	3820'	RC	100	100	.021	125	125	.030	150	150	.040	175	175	.046	190	200	.053	200	240	.060	220	290	.065	240	320
2° 00'	2865'	RC	100	100	.027	125	125	.038	150	150	.051	175	210	.057	190	250	.065	200	290	.072	230	340	.076	250	380
2° 30'	2292'	.021	100	100	.033	125	125	.046	150	170	.060	175	240	.066	190	290	.073	220	330	.078	250	370	.080	260	400
3° 00'	1910'	.025	100	100	.038	125	125	.053	150	190	.067	180	270	.073	210	320	.078	230	350	.080	250	380	.080	260	400
3° 30'	1637'	.028	100	100	.043	125	140	.058	150	210	.073	200	300	.077	220	330	.080	240	360	.080	250	380	D max=2.5°		
4° 00'	1432'	.032	100	100	.047	125	150	.063	150	230	.077	210	310	.079	230	340	.080	240	360	D max=3.0°					
5° 00'	1145'	.038	100	100	.055	125	170	.071	170	260	.080	220	320	.080	230	350	D max=3.5°								
6° 00'	955'	.043	100	120	.061	130	190	.077	180	280	.080	220	320	D max=4.5°											
7° 00'	819'	.048	100	130	.067	140	210	.079	190	280	D max=5.0°														
8° 00'	716'	.052	100	140	.071	150	220	.080	190	290															
9° 00'	637'	.056	100	150	.075	160	240	D max=7.5°																	
10° 00'	573'	.059	110	160	.077	160	240																		
11° 00'	521'	.063	110	170	.079	170	250																		
12° 00'	477'	.066	120	180	.080	170	250																		
13° 00'	441'	.068	120	180	.080	170	250																		
14° 00'	409'	.070	130	190	D max=12.5°																				
16° 00'	358'	.074	130	200																					
18° 00'	318'	.077	140	210																					
20° 00'	286'	.079	140	210																					
22° 00'	260'	.080	140	220																					
		.080	140	220																					
		D max=23.0°																							

$e$max=0.08

D—Degree of curve
R—Radius of curve
V—Assumed design speed
e—Rate of superelevation
L—Minimum length of runoff of spiral curve
NC—Normal crown section
RC—Remove adverse crown, superelevate at normal crown slope
Spirals desirable but not as essential above heavy line.
Lengths rounded in multiples of 25 or 50 feet permit simpler calculations.

TABLE 4

RATE OF SUPERELEVATION (e) AND MINIMUM LENGTH OF RUNOFF OR SPIRAL CURVE (L) FOR e max. = 0.10

D	R	V=30mph			V=40mph			V=50mph			V=60mph			V=65mph			V=70mph			V=75mph			V=80mph		
		e	L-Feet 2-lane	L-Feet 4-lane	e	L-Feet 2-lane	L-Feet 4-lane	e	L-Feet 2-lane	L-Feet 4-lane	e	L-Feet 2-lane	L-Feet 4-lane	e	L-Feet 2-lane	L-Feet 4-lane	e	L-Feet 2-lane	L-Feet 4-lane	e	L-Feet 2-lane	L-Feet 4-lane	e	L-Feet 2-lane	L-Feet 4-lane
0° 15'	22918'	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	RC	240	240
0° 30'	11459'	NC	0	0	NC	0	0	NC	0	0	RC	175	175	RC	190	190	RC	200	200	.022	220	220	.024	240	240
0° 45'	7639'	NC	0	0	NC	0	0	RC	150	150	.024	175	175	.027	190	190	.029	200	200	.033	220	220	.036	240	240
1° 00'	5730'	NC	0	0	RC	125	125	.023	150	150	.032	175	175	.035	190	190	.039	200	200	.044	220	220	.048	240	240
1° 30'	3820'	RC	100	100	.021	125	125	.033	150	150	.046	175	190	.052	190	220	.058	200	260	.065	220	310	.071	240	350
2° 00'	2865'	RC	100	100	.028	125	125	.042	150	150	.058	175	230	.066	190	290	.074	220	330	.082	260	390	.089	290	440
2° 30'	2292'	.021	100	100	.034	125	125	.051	150	180	.069	190	280	.077	220	330	.086	260	390	.094	300	450	.099	330	490
3° 00'	1910'	.025	100	100	.040	125	125	.059	150	210	.079	210	320	.087	250	380	.094	280	420	.100	320	480	.100	330	500
3° 30'	1637'	.029	100	100	.046	125	140	.067	160	240	.087	230	350	.093	270	400	.099	300	450	.100	320	480	.100	330	500
4° 00'	1432'	.033	100	100	.051	125	160	.073	180	260	.093	250	380	.098	280	420	.100	300	450	D max=3.0°			D max=3.0°		
5° 00'	1146'	.040	100	110	.061	130	190	.084	200	300	.099	270	400	.100	290	430	.100	300	450						
6° 00'	955'	.046	100	120	.070	150	220	.092	220	330	.100	270	410	D max=4.5°			D max=4.0°								
7° 00'	819'	.053	100	140	.077	160	240	.098	240	350	D max=5.5°														
8° 00'	716'	.059	110	160	.084	180	260	.100	240	360															
9° 00'	637'	.064	120	170	.089	190	280	.100	240	360															
10° 00'	573'	.068	120	180	.093	200	290	D max=8.5°																	
11° 00'	521'	.073	130	200	.097	200	310																		
12° 00'	477'	.077	140	210	.099	210	310																		
13° 00'	441'	.080	140	220	.100	210	320																		
14° 00'	409'	.083	150	220	.100	210	320																		
16° 00'	358'	.089	160	240	D max=13.5°																				
18° 00'	318'	.093	170	250																					
20° 00'	286'	.097	170	260																					
22° 00'	260'	.099	180	270																					
24° 00'	239'	.100	180	270																					
		.100	180	270																					
		D max=25.0°																							

e max=0.10

D—Degree of curve
R—Radius of curve
V—Assumed design speed
e—Rate of superelevation
L—Minimum length of runoff of spiral curve
NC—Normal crown section
RC—Remove adverse crown, superelevate at normal crown slope
Spirals desirable but not as essential above heavy line.
Lengths rounded in multiples of 25 or 50 feet permit simpler calculations.

TABLE 5

RATE OF SUPERELEVATION (e) AND MINIMUM LENGTH OF RUNOFF OR SPIRAL CURVE (L) FOR e max. = 0.12 - AASHO POLICY (2)

D	R	V = 30 mph			V = 40 mph			V = 50 mph			V = 60 mph			V = 65 mph			V = 70 mph			V = 75 mph			V = 80 mph		
		e	L-Feet		e	L-Feet		e	L-Feet		e	L-Feet		e	L-Feet		e	L-Feet		e	L-Feet		e	L-Feet	
			2-lane	4-lane		2-lane	4-lane		2-lane	4-lane		2-lane	4-lane		2-lane	4-lane		2-lane	4-lane		2-lane	4-lane		2-lane	4-lane
0° 15′	22918′	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	NC	0	0	RC	240	240
0° 30′	11459′	NC	0	0	NC	0	0	NC	0	0	RC	175	175	RC	190	190	RC	200	200	.022	220	220	.024	240	240
0° 45′	7639′	NC	0	0	NC	0	0	RC	150	150	.024	175	175	.026	190	190	.029	200	200	.033	220	220	.036	240	240
1° 00′	5730′	NC	0	0	RC	125	125	.023	150	150	.031	175	175	.035	190	190	.039	200	200	.043	220	220	.048	240	240
1° 30′	3820′	RC	100	100	.022	125	125	.034	150	150	.047	175	190	.053	190	230	.059	200	270	.065	220	310	.072	240	360
2° 00′	2865′	RC	100	100	.030	125	125	.045	150	160	.062	175	250	.070	200	300	.078	230	350	.087	280	410	.095	310	470
2° 30′	2292′	.022	100	100	.037	125	125	.055	150	200	.076	210	310	.085	240	370	.095	290	430	.105	330	500	.113	370	560
3° 00′	1910′	.026	100	100	.044	125	140	.065	160	230	.088	240	360	.097	240	420	.108	320	490	.117	370	560	.120	400	600
3° 30′	1637′	.030	100	100	.050	125	160	.074	180	270	.098	260	400	.107	310	460	.116	350	520	.120	380	570	.120	400	600
4° 00′	1432′	.034	100	100	.057	125	180	.082	200	300	.106	290	430	.114	330	490	.120	360	540	.120	380	570	D max=3.0°		
5° 00′	1146′	.042	100	110	.068	140	210	.096	230	350	.117	320	470	.120	350	520	.120	360	540	D max=3.5°					
6° 00′	955′	.049	100	130	.079	170	250	.107	260	390	.120	320	490	.120	350	520	D max=4.0°								
7° 00′	819′	.055	100	150	.088	180	280	.114	270	410	.120	320	490	D max=5.0°											
8° 00′	716′	.062	110	170	.096	200	300	.119	290	430	D max=6.0°														
9° 00′	637′	.068	120	180	.103	220	320	.120	290	430															
10° 00′	573′	.074	130	200	.108	230	340	.120	290	430															
11° 00′	521′	.079	140	210	.113	240	360	D max=9.0°																	
12° 00′	477′	.084	150	230	.116	240	370																		
13° 00′	441′	.089	160	240	.119	250	370																		
14° 00′	409′	.093	170	250	.120	250	380																		
16° 00′	358′	.101	180	270	.120	250	380																		
18° 00′	318′	.108	190	290	D max=14.5°																				
20° 00′	286′	.113	200	310																					
22° 00′	260′	.117	210	320																					
26° 00′	220′	.120	220	320																					
		.120	220	320																					
		D max=26.5°																							

e_{max}=0.12

D—Degree of curve
R—Radius of curve
V—Assumed design speed
e—Rate of superelevation
L—Minimum length of runoff of spiral curve
NC—Normal crown section
RC—Remove adverse crown, superelevate at normal crown slope
Spirals desirable but not as essential above heavy line.
Lengths rounded in multiples of 25 or 50 feet permit simpler calculations.

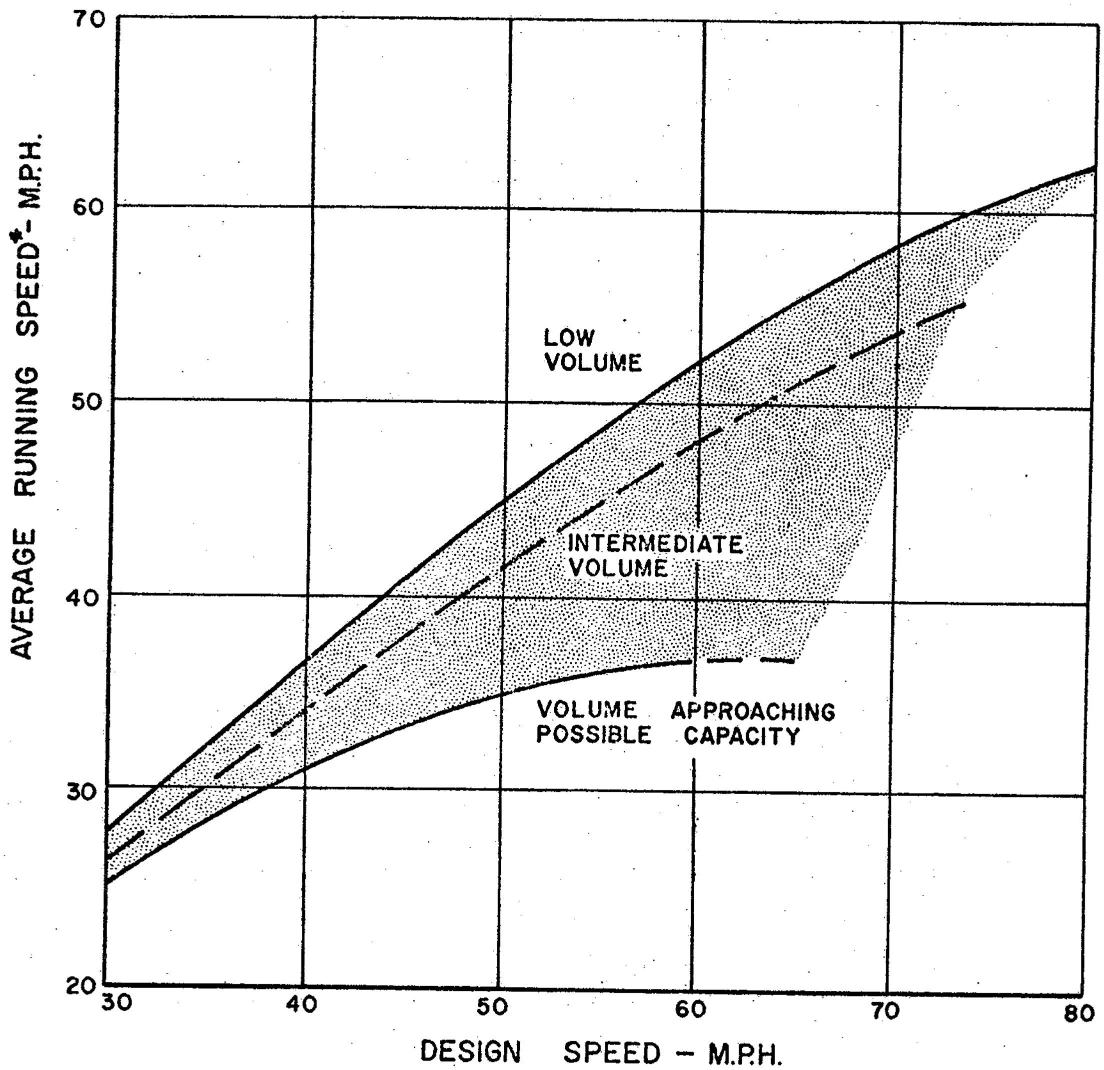
H. MINIMUM RADIUS FOR TURNING ROADWAYS

In consideration of design speeds for turning roadways at highway intersections, the AASHO Policy states the following: (2)

> While it is desirable and often feasible to design for turning vehicles operating at higher speeds, in most cases of rural intersections at grade, low turning design speeds will be necessary for safety and economy. The speeds for which intersection curves should be designed depend largely upon vehicle speeds on the approach highways, type of intersection, and volumes of through and turning traffic. Generally, a desirable turning speed for design is the average running speed (table II-6) of traffic on the highway approaching the turn. Designs at such speeds offer little hindrance to smooth flow of traffic and may be justified on some interchange ramps or on at-grade intersections for certain movements involving little or no conflict with pedestrians or other vehicular traffic.
>
> Curves at intersections need not be considered in the same category as curves on the open highway because the various warnings provided and the anticipation of more critical conditions at an intersection permit the use of less liberal design factors. Drivers generally operate on intersection curves at higher speeds in relation to the degree of curvature than on open highway curves. This is accomplished by the drivers' acceptance and use of higher side friction in operating around curves on intersections than on the through highway.

The table (II-6) mentioned in the above quote is shown graphically in Figure 7 of this report.

To arrive at design values for side friction demand, the AASHO Policy considered several studies (26, 27, 28, 29) in which the 95th percentile speed was related to the side friction demand. The results of these studies are plotted in Figure 8. For employing these data in design, the AASHO Policy states:



RUNNING SPEED IS THE SPEED (OF AN INDIVIDUAL VEHICLE) OVER A SPECIFIED SECTION OF HIGHWAY, BEING DIVIDED BY RUNNING TIME.

*AVERAGE RUNNING SPEED IS THE AVERAGE FOR ALL TRAFFIC OR COMPONENT OF TRAFFIC, BEING THE SUMMATION OF DISTANCES DIVIDED BY THE SUMMATION OF RUNNING TIMES. IT IS APPROXIMATELY EQUAL TO THE AVERAGE OF THE RUNNING SPEEDS OF ALL VEHICLES BEING CONSIDERED.

Figure 7. Relation of Average Running Speed and Volume Conditions - AASHO Policy (2).

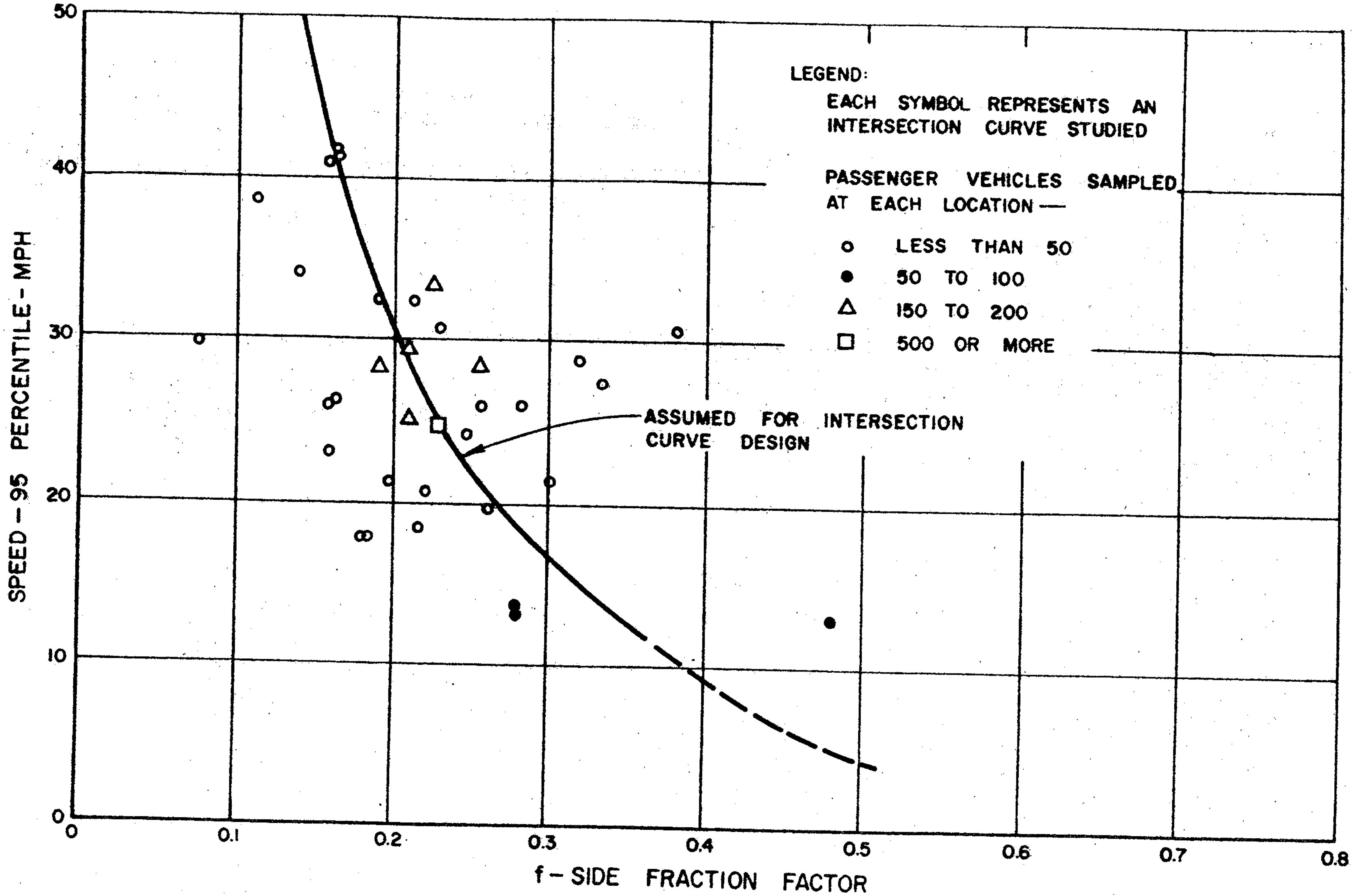


Figure 8. Relation Between Speed and Side Friction Factor of Curves at Intersections - AASHO Policy (2).

In the analyses of these data, the 95-percentile speed of traffic was assumed to be that closely representing the design speed, which generally corresponds to the speed adopted by the faster group of drivers. Side friction factors (taking superelevation into account) actually developed by drivers negotiating the curves at 95-percentile speed are indicated for 34 locations. The dashed line at the upper left shows the side friction factors used for design of curves on open highways. Use of this control limit for higher speeds, and a friction factor of about 0.5 which could be developed at a low speed as the other limit, gives an average or representative curve through the plottings of individual observations--a relation between design (95-percentile) speed and side friction factor which is considered appropriate for intersection curve design.

With this relation established and with logical assumptions for the superelevation rates that can be developed on intersection curves, minimum radii for various design speeds are derived from the standard curve formula. Obviously different rates of superelevation would produce somewhat different radii for a given design speed and side friction factor. For intersection curve design it is desirable to establish a single minimum radius for each design speed. This is done by assuming a likely minimum rate of superelevation--a conservative value--that could nearly always be obtained for certain radii. If more superelevation than this minimum is actually provided, drivers either will be able to drive the curves a little faster or drive them more comfortably due to less friction.

In selecting such a minimum rate of superelevation it is recognized that the sharper the curve, the shorter its length, and the less the opportunity for developing a large rate of superelevation. This applies particularly to at-grade intersections where the turning roadway is often close and much of its area adjacent to the through pavement, and where the complete turn is made through a total angle of about 90 degrees. Assuming the more critical conditions, and considering the lengths likely to be available for developing superelevation on curves of various radii, the minimum rate of superelevation for derivation purposes is taken as that varying from zero at 15 mph to 0.08 at 35 mph.

By substituting the superelevation rates described above and the side friction factors of Figure 8 into the standard curve equation, the AASHO Policy derived the recommended values for radius of intersection curves listed in Table 6.

In relation to the design of mainline curves, the design friction values for turning roadways appear somewhat excessive. The rationale for this difference is not apparent. The data plotted in Figure 8 does not show that drivers "accept" these high friction values but that they "experienced" them because the turning roadway was underdesigned for the prevailing operational conditions. These high friction design values are also questionable from the viewpoint that they allow friction demands which would promote rapid degradation of the pavement friction capability.

TABLE 6

MINIMUM RADII FOR INTERSECTION CURVES-
AASHO Policy (2)

Design (turning) speed (V), mph	15	20	25	30	35	40
Side friction factor (f)	0.32	0.27	0.23	0.20	0.18	0.16
Assumed min. superelevation (e)	.00	.02	.04	.06	.08	0.09
Total e + f	.32	.29	.27	.26	.26	.25
Calculated min. radius (R), feet	47	92	154	231	314	426
Suggested curvature for design:						
Radius--minimum, feet	50	90	150	230	310	430
Degree of curve--maximum	--	64	38	25	18	13
Average running speed, mph	14	18	22	26	30	34

NOTE: For design speeds of more than 40 mph, use values for open highway conditions.

I. SPIRAL TRANSITION CURVES

A vehicle which traverses from tangent alignment to a horizontal circular curve must follow a transitional path. The steering change and the consequent gain in lateral force cannot be effected instantly. This path varies depending on speed, curve radius, superelevation, and the particular driver's steering action. With moderate speed and curvature, most drivers can effect a suitable transition path within the limits of normal lane width. With high speed and sharp curvature the resultant longer transition can be traversed only by hazardous crowding or actual occupation of either adjoining lanes or the shoulder.

The AASHO Policy encourages the use of spiral transition curves based on the following stated advantages:

1. Properly designed transition curves provide a natural easy-to-follow path for drivers, such that the centrifugal force increases and decreases gradually as the vehicle enters and leaves a circular curve. This minimizes encroachment upon adjoining traffic lanes, tends to promote uniformity in speed, and results in increased safety.

2. The transition curve length provides a convenient desirable arrangement for superelevation runoff. The transition between the normal cross slope and the fully superelevated section on the curve can be effected along the length of transition curve in a manner closely fitting the speed-radius relation for the vehicle transversing it. Where superelevation runoff is effected without a transition curve, usually partly on curve and partly on tangent, the driver approaching a curve may have to steer opposite to the direction of the curve ahead when on the superelevated tangent portion in order to keep his vehicle on tangent. This is an unnatural maneuver and explains in part why many vehicles drift to the inside on curves.

3. Where the pavement section is to be widened around a circular curve, the spiral facilitates the transition in width. Use of spirals not only permits simplification of design procedure but provide flexibility in that widening on sharp curves can be applied, in part, on the outside of pavement with a reverse-edge alignment.

4. The appearance of the highway is enhanced by the application of spirals. Their use avoids the noticeable breaks at the beginning and ending of circular curves, which may be distorted further by superelevation runoff. Spirals are an essential part of the natural flowing alignment that appears pleasing and fitting to the conditions.

In recent years, the spiral transition has not had widespread use. The type of spiral curve in general use is the "Euler" spiral, which in mathematical terminology is a clothoid. The degree of curve varies from zero at the tangent end to the degree of circular curve at the curve end. By definition, the degree of curve at any point on the spiral curve varies directly with the distance measured along the spiral curve.

Formulas are available for computing the length of spiral curve depending on degree of circular curve and design speed. A more practical control for the length of spiral is that where the length of spiral equals the length required for superelevation runoff.

J. SUPERELEVATION RUNOFF

According to the AASHO Policy, "superelevation runoff is the general term denoting the length of highway needed to accomplish the change in cross-slope from a normal crown section to the fully super-elevated section or vise versa," (although in selecting values for design the Policy chose to consider superelevation runoff as only that length between zero crown and full superelevation). To meet the requirements for comfort and safety, the superelevation should be effected uniformly over a length adequate for likely travel speeds. Some states employ the spiral curve and use its length to effect the change in cross slope. Others do not employ the spiral, but designate proportional lengths of tangent and curve for the same purpose.

The AASHO Policy states that current design practice indicates that the appearance aspect of superelevation runoff largely governs its length. Required spiral lengths as determined otherwise are often shorter than that determined for general appearance, so that spiral formula values give way to longer empirical runoff values.

The recommended AASHO Policy values for superelevation runoff are presented in Table 7. The minimum lengths shown in the lower part of the table are assumed for design use. These minimum values approximate the distance traveled in two seconds at the design speed and should be used in place of the shorter lengths above the horizontal bars in the tabulation. The AASHO Policy also concludes that 60 to 80 percent of the runoff should be placed on the tangent.

TABLE 7

LENGTH REQUIRED FOR SUPERELEVATION RUNOFF FOR TWO LANE PAVEMENTS - AASHO POLICY (2)

Superelevation rate, foot per foot	L--Length of runoff in feet for design speed, mph of: 30	40	50	60	65	70	75	80
	12-foot lanes							
.02	35	40	50	55	60	60	65	65
.04	70	85	95	110	115	120	125	130
.06	110	125	145	160	170	180	190	200
.08	145	170	190	215	230	240	255	265
.10	180	210	240	270	290	300	330	330
.12	215	250	290	325	345	360	390	395
	10-foot lanes							
.02	30	35	40	45	50	50	55	55
.04	60	70	80	90	95	100	105	110
.06	90	105	120	135	145	150	160	165
.08	120	140	160	180	190	200	210	220
.10	150	175	200	225	240	250	265	275
.12	180	210	240	270	290	300	320	330
Design minimum length regardless of superelevation	100	125	150	175	190	200	220	240

K. DESIGN FACTORS OF SAFETY

Factors of safety are incorporated into the highway curve design procedure because of several variables that have not been explicitly evaluated. They are those variables which would tend to cause vehicle instability if factors of safety were not used. This section presents a description of these variables for the purpose of evaluating the validity of the AASHO Policy.

Vehicle Paths

The AASHO Policy employs the explicit assumption that a vehicle will follow the designed path of a highway curve with geometric exactness. This is exemplified by the substitution of the highway curve radius in place of the vehicle path radius into the standard centripetal force equation. Actually, if a vehicle is to remain within its lane while traversing a circular highway curve, the degree of the highway curve is about the lowest maximum instantaneous vehicle path curvature required. This relationship is discussed by Stonex and Noble (5):

> Very few drivers maintain a truly accurate course and are inclined to drive around a curve in "chords," thus running up centrifugal ratios considerably above the average required by a true path.

If, however, the driver is distracted or he simply misjudges the degree of curve (particularly likely at night), he may find his vehicle crowding the adjacent lane or the shoulder; necessitating a substantial corrective maneuver to avoid a collision with an oncoming vehicle or to avoid running off the road. The maximum degree of cornering, therefore, may vary from the degree of highway curve to some degree much greater than this value.

As was discussed earlier, side friction demand increases directly with the degree of vehicle cornering. Figure 9 illustrates the sensitivity (at design speed) of side friction demand for increases in vehicle path degree above the degree of the highway curve. It may be seen from this representation that side friction demand is very sensitive to slight increases in cornering degree for highway curves with the higher design speeds. In fact, for highway curves with a 60 mph design speed or greater, a two degree increase above the highway curve degree requires more side friction than the indicated capability of the 15th percentile pavement of Figure 4 (page 12).

To evaluate the adequacy of the apparent factors of safety employed in highway curve design, it becomes necessary to know the distribution of vehicle cornering demands encountered on our highways. The project staff was unable to locate any published research concerned with the direct measurement of vehicle cornering demands experienced on highway curves. Therefore, to validate the possibility of greater friction demands due to cornering paths greater than those designed for, a hypothetical analysis was conducted of passing maneuvers on highway curves. Passing is an entirely feasible maneuver on four-lane highways and on two-lane highways which do not have sight distance restrictions.

The analysis of the friction demands encountered in passing maneuvers on highway curves is presented in Appendix C. It is emphasized that this analysis is employed strictly for illustrative purposes and is <u>not</u> intended to suggest recommendations for design

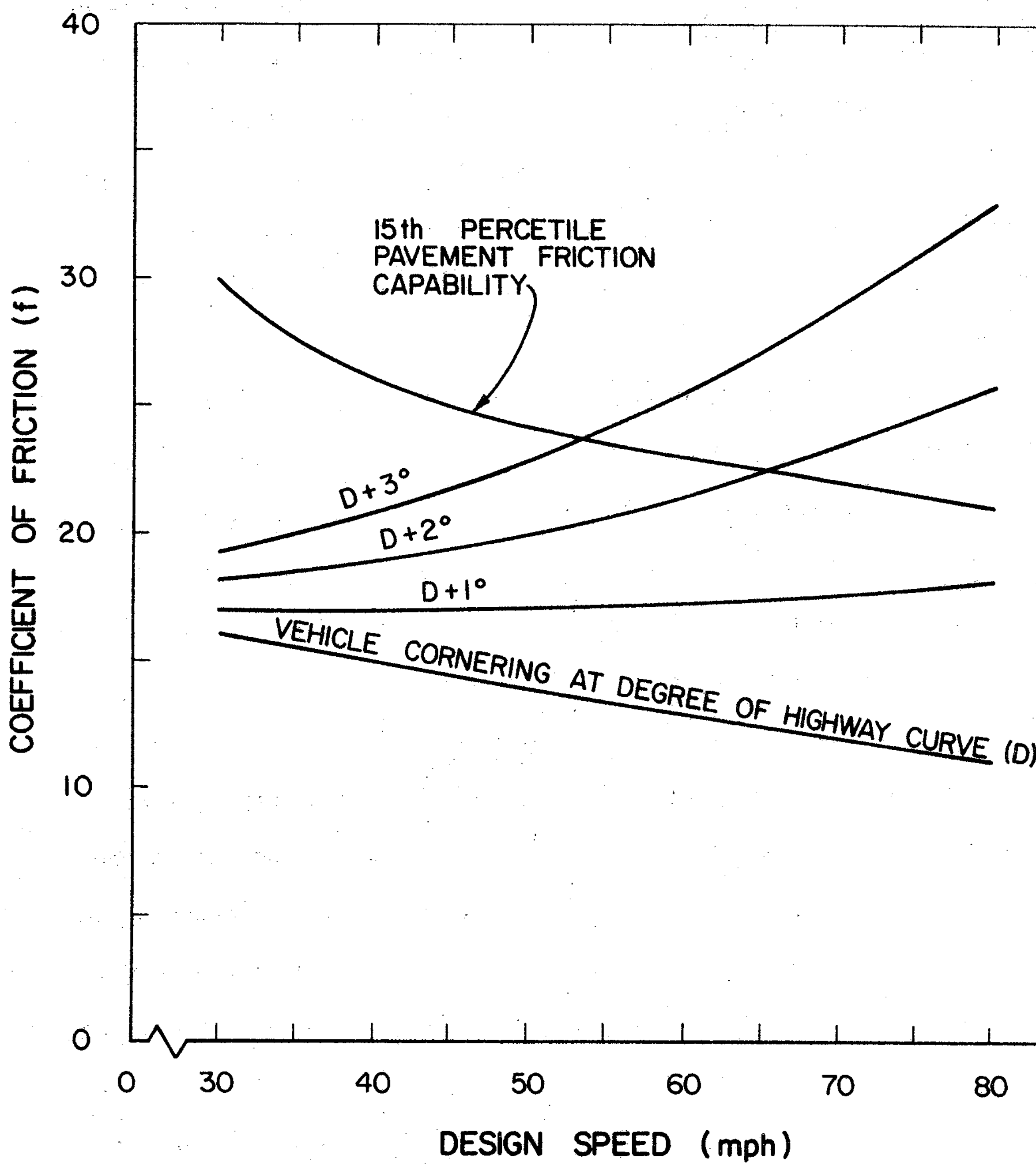


Figure 9. Friction Demand Related to Degree of Cornering for Various Design Speed Highway Curve (With e = 0.08).

standard revisions. The analysis presented in Appendix C employs the AASHO Policy's speed-distance considerations for passing sight distance design standards. In addition, the analysis employs certain rational assumptions regarding the path geometry of the passing vehicle. Based on the analysis, it is shown in Table C-1 that there is a significant increase in friction demand, ranging from 0.08 to 0.12, for AASHO design curves.

The above illustration indicates that it is feasible to expect friction demands in excess of those explicitly designed for in the AASHO Policy. In all probability, friction demands for emergency maneuvers greatly exceeds those of the normal passing maneuver. Therefore, the validity of the apparent factors of safety employed by the AASHO Policy is questionable without documentation of the actual vehicle maneuvers encountered on highway curves.

The above discussion of vehicle paths other than those of the designed geometric path brings to light another possibility. It appears possible that a vehicle could perform a circular path opposite to the geometry of the highway curve. In this case, the superelevation would be a reverse or negative superelevation and the friction demand would be increased rather than decreased by the amount of the superelevation. For example, if a 70-mph vehicle performed a 3-degree maneuver opposite the curve geometry on a highway curve designed for 70-mph (4-degrees with a superelevation rate of 0.10), the friction demand would be 0.27 rather than the 0.12 design level.

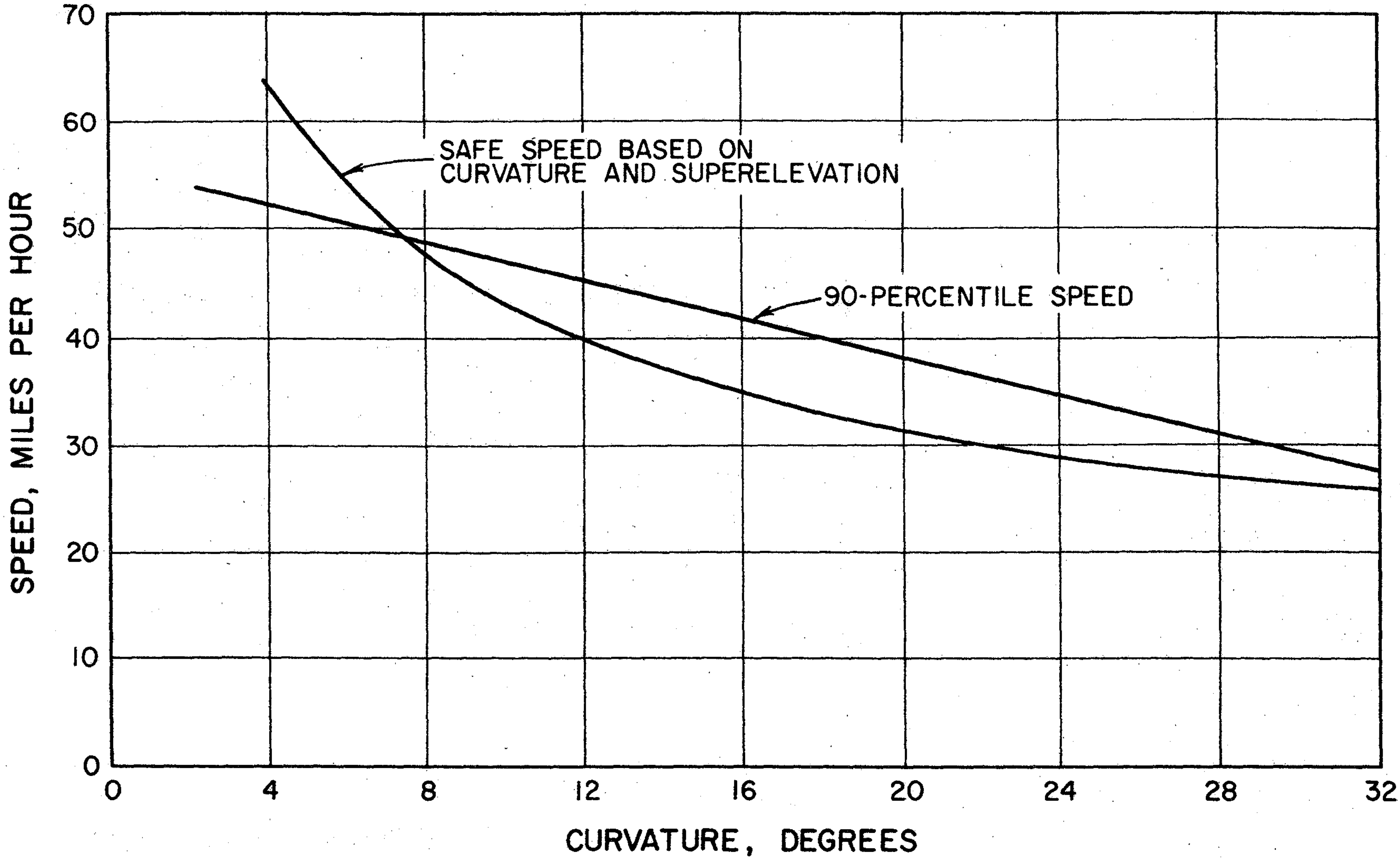


Figure 10. Relation Between AASHO Design Speed and Measured Operating Speeds on Horizontal Curves (30).

Vehicle Speeds

The design speed is assumed to be the critical operating speed of the highway. Only one reference (30) was found that related a critical operating speed to AASHO design speeds. This relationship is shown graphically in Figure 10. It is seen from this plot that a higher proportion of drivers exceed the design speed on curves with lower design speeds.

As was discussed earlier, side friction demand increases with the square of vehicle speed. Figure 11 illustrates the sensitivity (at design cornering degree) of side friction demand for increases in vehicle speed. It may be observed from this representation that side friction demand is very sensitive to slight increases in vehicle speed for the lower design speed curves.

Considering both Figures 10 and 11, the safety margin for highway curves with the lower design speeds appears to be very small.

Vehicle Acceleration and Braking

The AASHO Policy employs the explicit assumption that a vehicle traverses a highway curve at constant speed. This assumption disregards the increased resultant friction demand due the combined effects of cornering and acceleration or braking. This resultant friction demand is a vector sum of the two separate friction requirements (1). To evaluate the effect of these combined forces on the validity of the apparent factors of safety employed in highway curve design, it is necessary to study actual vehicle operations on curves.

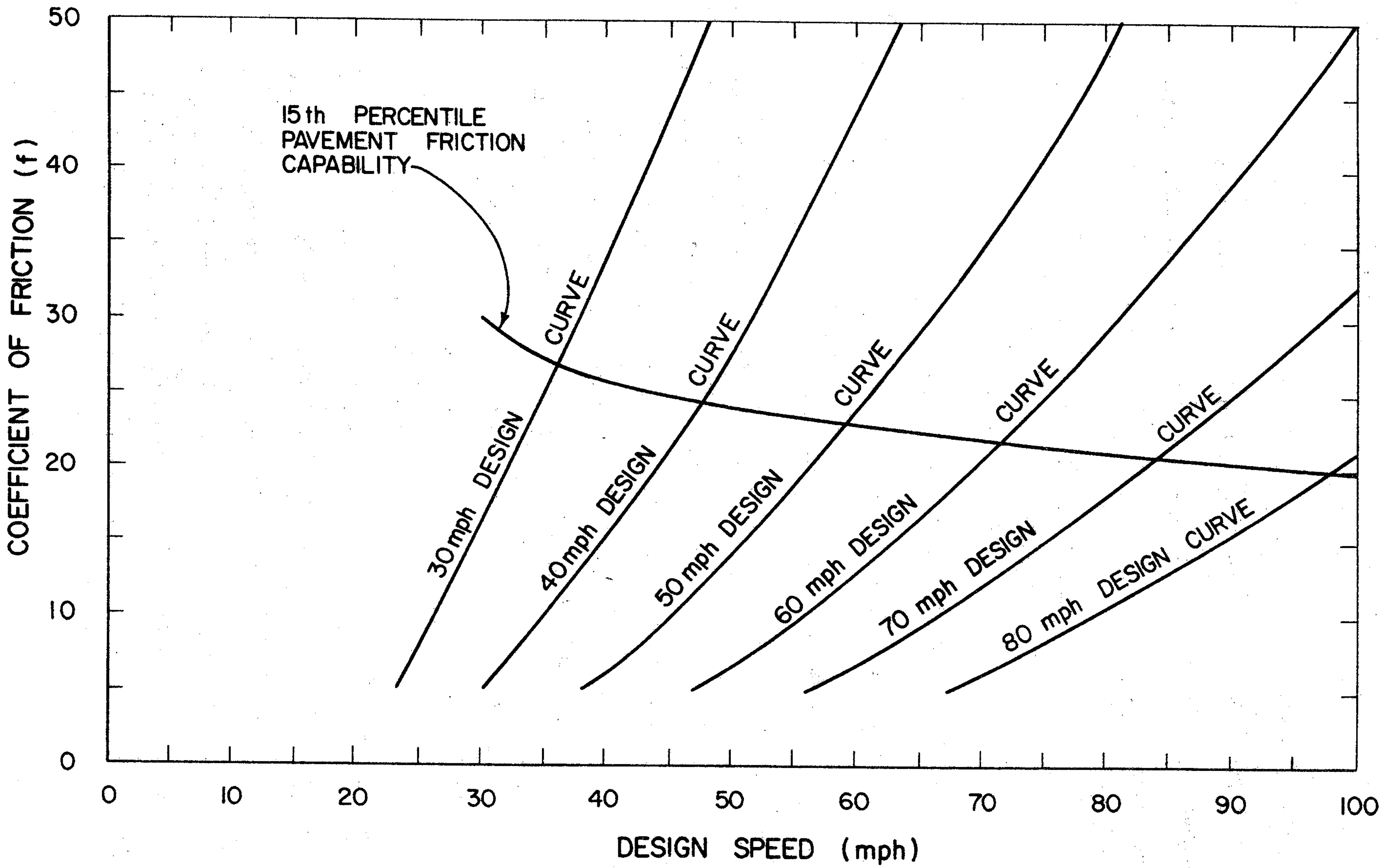


Figure 11. Friction Demand Related to Speed for a Vehicle Cornering at the Design Degree of Curve (for e = 0.08).

Vertical Curvature

In Appendix D, the standard centripetal force equation is rederived superimposing a centripetal force in the vertical direction. The new equation is:

$$f = \frac{V^2}{R_h \left(15 \pm \frac{V^2}{R_v}\right)} \qquad (3)$$

where all factors are as previously defined except:

R_h = Radius of horizontal curve, in feet

R_v = Radius of vertical curve, in feet

The sign in the denominator of Equation 3 is positive for sag vertical curves and negative for crest vertical curves. Therefore, sag vertical curves tend to decrease the side force requirement, whereas, crest vertical curves tend to increase the side force requirement. The equation for crest vertical curves is only correct for radii above the take-off radius, $R_v = V^2/15$, which is derived when the normal force, N, equals zero.

The discussion above indicates that vertical curvature has an effect on side friction demand. For conventional vertical curves used in highway design, this effect is probably small. For pavement irregularities, however, the effect could be significant. More definite statements regarding the effect of vertical curvature cannot be offered because,of course, the equation again has questionable validity for all situations due to the point-mass assumption.

Superelevation Runoff on Unspiraled Curves

For superelevated curves, it is rational to provide a cross-slope transition section from the normal crown on the tangent to full superelevation on the curve. Without the spiral transition, however, this cross-section transition appears to create a compound dilemma. This is most easily illustrated by an example as shown in Figure 12. As a vehicle approaches a curve it is presented first with Problem Area 1 in which the cross-slope is less than 0.01 ft/ft. Because of this slight cross-slope the pavement does not drain well, thus creating a high potential hydroplaning section. The vehicle no sooner gets through Problem Area 1 (where it may have experienced partial loss of control) when it is presented with Problem Area 2. In Problem Area 2, the driver may experience some steering difficulty because the cross-slope requires him to steer opposite the direction of the upcoming curve. When the vehicle passes from Problem Area 2 to Problem Area 3, the driver must reverse his steering to follow the curve. At this point if he attempts to assume the degree of highway curve the side friction demand will be greater than that designed for, since Problem Area 3 does not have full superelevation.

At design speed, for the example, the driver proceeds through the "compound dilemma area" in 2.6 seconds. It is questionable that a driver can react adequately to these demands on his perception in the time required.

There are two methods to alleviate the high potential hydroplaning secton. One is to carry the crown through the curve (this, of course, requires a flatter curve for the specific design speed). The second

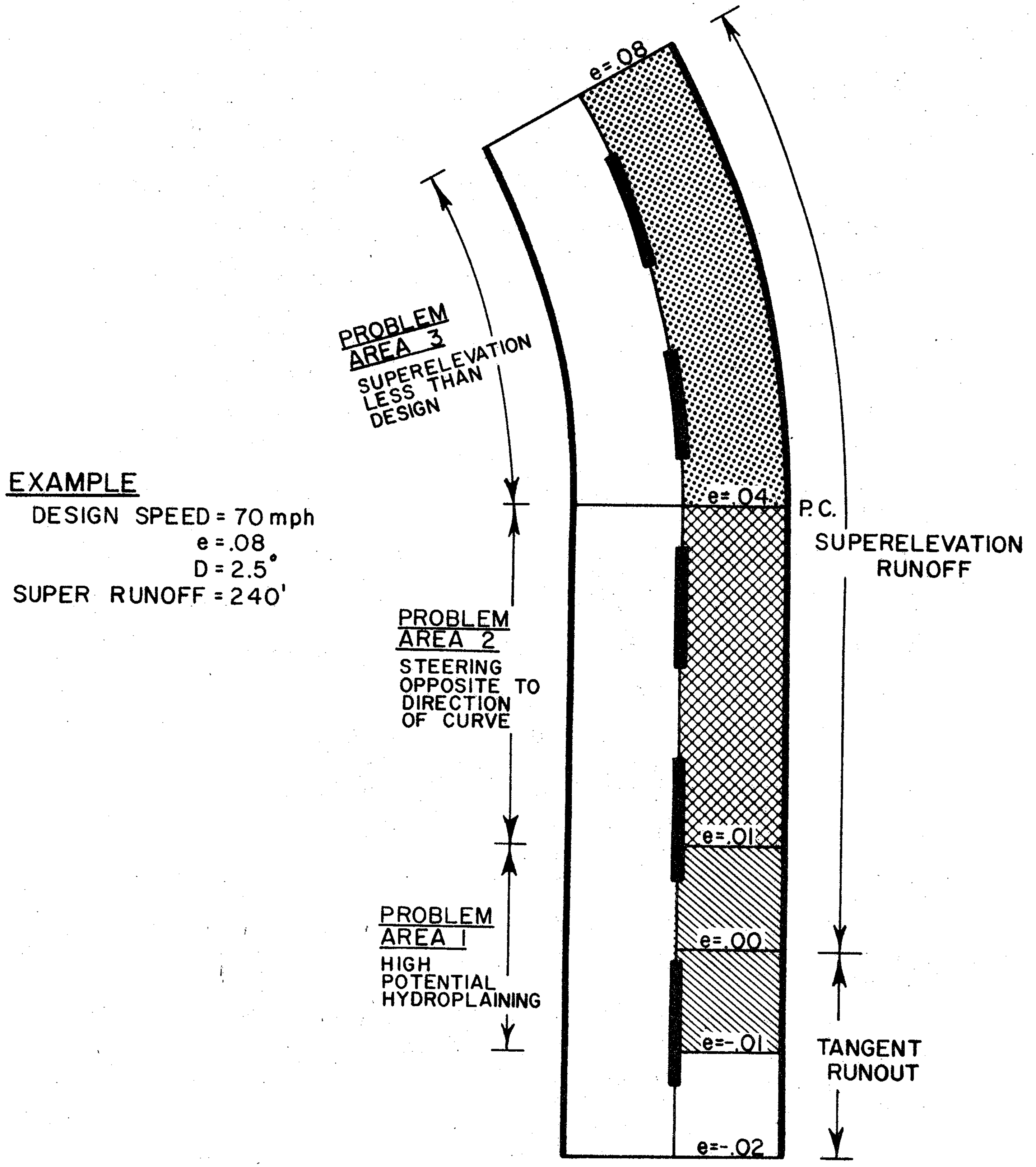


Figure 12. The Cross-slope Transition Area and its Related Maneuver Problems.

method would be to make provisions for a self-draining pavement within the problem area.

There does not appear to be an ideal division of the superelevation runoff between the tangent and the curve. By shifting the location of the runoff section, one simply shifts more of the problem from one problem area to another. One method to alleviate the dilemma in Problem Areas 2 and 3 is to carry the crown through the curve (requires a flatter curve). Another partial solution is to reduce the maximum allowable superelevation. This will reduce the severity and the length of Problem Areas 2 and 3.

Pavement Friction Degradation Due to Aggregate Polishing

Polishing of pavement surface aggregate is an important consideration in the continued provision of adequate factors of safety on highway curves. Heavy traffic and high friction demands promote a rapid degradation of the frictional capability of the pavement (1). This would suggest that lower levels of friction demand be used in design to prevent a rapid reduction of the frictional capability of a pavement. This consideration is perhaps related to maintenance economy; however, proper frictional levels cannot be maintained on a jurisdictional basis if the maintenance load exceeds the available budget.

Other Variables That May Reduce the Factor of Safety

There are several other variables which may either increase the side friction demand or decrease the side friction capability of the tire-pavement combination on highway curves depending on temporal conditions. These variables which would reduce the factor of safety against a cornering skid are:

a. Excessive water depth on the pavement -- Frictional capability decreases with increasing water depth (1). If the water depth exceeds that used to measure a typical frictional capability level, the factor of safety will be reduced accordingly.

b. Excessive tire temperature -- Frictional capability decreases with excessive temperatures (1). If the tire temperature exceeds that used to measure a typical frictional capability level, the factor of safety will be reduced accordingly.

c. Gusty winds -- High wind gusts in the direction of the outside of a highway curve may significantly increase the frictional demand of the vehicle above the design level. This is especially true with vehicles such as pick-up trucks with camper cabins on their bed.

d. Faulty vehicle characteristics -- There are many characteristics of the vehicle which may significantly reduce the frictional capability of the tire-pavement combination. These include: (1) bald tires; (2) low tire pressure; (3) uneven tire loads; (5) faulty suspension system; and (6) poor wheel alignment.

e. Foreign material on the pavement -- Snow, ice, oil, loose aggregate, or a heavy dust on the pavement will significantly reduce the side friction capability of the tire-pavement combination.

f. Power steering -- It is believed that, in emergency situations, drivers over-react in terms of maneuvering their vehicle. If the vehicle is equipped with power steering this behavior could create crucial side friction demands. With the increased availability of power steering in recent years, accidents related to over-reactive maneuvering may be increasing.

BIBLIOGRAPHY

1. Kummer, H. W., and Meyer, W. E., "Tentative Skid-Resistance Requirements for Main Rural Highways," NCHRP Report 37, 1967.

2. American Association of State Highway Officials, A Policy on Geometric Design of Rural Highways - 1965.

3. American Association of State Highway Officials, "A Policy on Intersections at Grade," 1940.

4. Stocker, A. J., Gallaway, B. M., Ivey, D. L., Swift, G., Darroch, J. G., "Tractional Characteristics of Automobile Tires," Texas Transportation Institute, November, 1968.

5. Stonex, K. A., and Noble, C. M., "Curve Design and Tests on the Pennsylvania Turnpike," HRB Proceedings, 1940.

6. "Report of Committee on Antiskid Properites of Road Surfaces," HRB Bulletin No. 27, 1950.

7. Moyer, R. A., "Skidding Characteristics of Road Surfaces," HRB Proceedings, 1933.

8. Information conveyed in communique with Kenneth D. Hankins, Supervising Design Research Engineer, Texas Highway Department, June, 1969.

9. Unpublished analysis of data measured in 1964 by a state highway department.

10. Schulze, K. H., and Beckman, L., "Friction Properties of Pavements at Different Speeds," ASTM Special Technical Publication No. 326, 1962.

11. Barnett, J., "Safe Side Friction Factors and Superelevation Design," HRB Proceedings, 1936.

12. Moyer, R. A., and Berry, D. S., "Marking Highway Curves With Safe Speed Indications," HRB Proceedings, 1940.

13. Meyer, C. F., "Route Surveying," 1949.

14. Fox, M. L., "Relations Between Curvature and Speed," HRB Proceedings, 1937.

15. "Planning Manual, Part 7, Design," California Division of Highways, updated to 1969.

16. "Roadway Design Manual," Colorado Department of Highways, updated to 1968.

17. "Procedures Manual," Colorado Department of Highways, updated to 1968.

18. "Instructional Manual," Design Division, New Mexico Highway Department, updated to 1968.

19. "Manual of Instructions -- Location, Design, Construction," Construction Division, Oregon Highway Department, revised November, 1966.

20. "Design Manual, Part 2, Highway Design," Pennsylvania Department of Highways, updated to May, 1969.

21. "Manual of Instructions, Part 6, Location, Roadway Design, Utilities," Utah Department of Highways.

22. "Procedures Manual, Road Design, Volume 1," Wyoming Highway Department, updated to 1969.

23. "Guidance Manual," Division of Design, Kentucky Department of Highways, updated to 1969.

24. "Design Handbook," Roadway Design Department, North Carolina Highway Commission, May, 1967.

25. "Design Manual for Controlled Access Highways," Texas Highway Department, 1967.

26. "Determination of Safe Turning Speeds on Various Radii at Intersections," Connecticut State Highway Department, 1950.

27. "Study of Speed Curvature Relations on Pentagon Road Network Ramps," Unpublished Data, Bureau of Public Roads, 1948.

28. Cysewski, G. R., "Urban Intersectional Right Turning Movements," Traffic Engineering, October, 1949.

29. George, L. E., "Characteristics of Left Turning Passenger Vehicles," Yale Bureau of Traffic (thesis), May, 1951

30. Taragin, A., "Driver Performance on Horizontal Curves," HRB Proceedings, 1954.

31. "Operations and Procedures Manual," Highway Design Division, Texas Highway Department, 1969.

APPENDICES

Appendix A

Derivation of the Cornering Model

A vehicle traveling a curved path on a flat surface is held in that path by side friction between tires and pavement. The forces acting on an automobile rounding an inwardly banked curve of mean radius, r, at a constant speed, v, are shown in Figure A-1. The velocity of the vehicle is normal to the plane of the figure, but the acceleration, $\bar{a}_n = v^2/r$, is toward the center of the curve and is in the plane of the figure. If the radius, r, is large compared to the dimensions of the vehicle, each point in the vehicle may be assumed to have the same acceleration. Thus, the dynamics of the vehicle may be analyzed by the principles of translation applied in the plane of the figure. The forces acting on the car may be represented by the weight, W, the normal tire forces, N_1 and N_2, and the lateral friction forces on the tires, F_1 and F_2. Each of these tire forces is, of course, the sum of the front and rear tire forces. The resultant of the tire forces, P, must pass through the center of gravity, G, since the resultant of P and W is $R = m\bar{a}_n$ which passes through G. The equations of motion are:

$$[\Sigma F_n = m\bar{a}_n] \qquad P \sin(\Theta + \alpha) = \frac{W}{g}\frac{v^2}{r}$$

$$[\Sigma F_y = 0] \qquad P \cos(\Theta + \alpha) = W$$

Dividing gives

$$\tan(\Theta + \alpha) = \frac{v^2}{gr} \qquad \text{or} \qquad v^2 = gr\,\frac{\tan\Theta + \tan\alpha}{1 - \tan\Theta\tan\alpha}$$

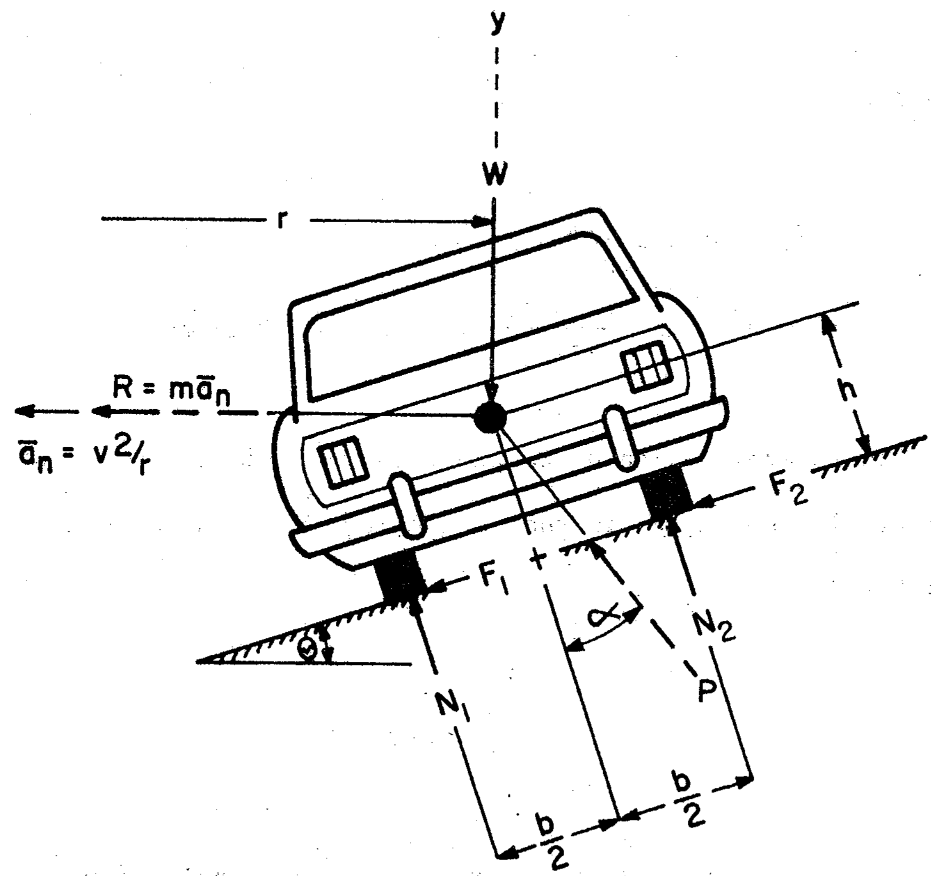


Figure A-1 Vehicle Cornering Relationship

In highway engineering terms

$\tan \Theta = e$ (superelevation in ft/ft)

$\tan \alpha = f$ (coefficient of friction)

Therefore:

$$V^2 = gr\frac{(e + f)}{1 - ef}$$

or $V^2 = 15r \frac{(e + f)}{1 - ef}$ with V = speed in mph.

The superelevation which produces no tendency to tip or slide for a particular speed, V, is that angle for which there is no side friction, thus $f = 0$, $N_1 = N_2$, and

$$e = \frac{V^2}{15r}$$

This shows that a highway curve of given radius can be properly superelevated for one speed only. The speed at which the car overturns occurs when the reaction, P, acts entirely at the outer wheels. In this event, $f = (b/2)/h$, and thus:

$$V^2 = 15r \frac{e + (b/2h)}{1 - e(b/2h)}$$

This relation assumes sufficient friction to allow P to act at the outer wheels, and is valid only if the coefficient of friction, f, is greater than b/2h.

The car will slide before it will tip, on the other hand, if the coefficient of friction, f, is less than b/2h. Thus, the speed at which sliding begins is given by;

$$V^2 = 15r \frac{e + f}{1 - ef}$$

For most practical considerations, the factor 1-ef is very close to unity; therefore the following equation is generally employed for highway curve design:

$$V^2 = 15r\ (e+f)$$

Appendix B

Tire Test Results

The National Traffic and Motor Safety Vehicle Act of 1966 provided for the development of a uniform quality grading system for pneumatic passenger vehicle tires. In order to develop this system, the National Bureau of Standards deemed it necessary to conduct tests on tires currently in production, to provide the necessary data base. Under contract to the National Bureau of Standards, the Texas A&M Research Foundation and the Texas Transportation Institute undertook the testing of 95 sets of tires during the period of March 5 to November 30, 1968 (2). The various sets of tires included in this program are presented in Table B-1. Each set of tires was tested to provide data on tractional characteristics when stopping with locked wheels and to determine loss of traction while driving through curves.

The pavements used in this test program were specially constructed to achieve predetermined coefficients of friction. They included four different asphalt pavements and one portland cement concrete pavement. Each curved test pad had a centerline radius of 286.48 feet (20 degree curve), a superelevation of zero, a length of 400 feet, and a width of 12 feet. A straight approach section approximately 100 feet in length was constructed at the beginning of each curve.

Test Vehicle Description

The automobile used in this test program was a 1968 4-door Bel Air Chevrolet (see Figure B-1). Modification was made to the suspension system, including a change to heavy duty coil springs and heavy duty shock absorbers. Prior to each day of testing, the vehicle height was determined by measuring the height of marks placed on the bumper at each corner of the car. This procedure was established to determine if deterioration was occurring in the suspension system. Air pressure for the automobile tires tested was 24 psi cold.

The tire-test vehicle was equipped to indicate and record the following information:

a. Distance traveled as a function of time;
b. Velocity of the vehicle as a function of time;
c. Rear-wheel lock-up point; and
d. Lateral forces (transverse accelerations)

Distance and velocity data were obtained from a Track-test fifth wheel assembly attached to the rear bumper. Lateral forces were sensed by two Kistler force-balance accelerometers, aligned with the front and rear axles. One accelerometer was mounted in the trunk, the other on the left front fender. Data were recorded by a Honeywell Visicorder. The AC power required for the Visicorder was supplied by a gasoline engine generator mounted in the trunk.

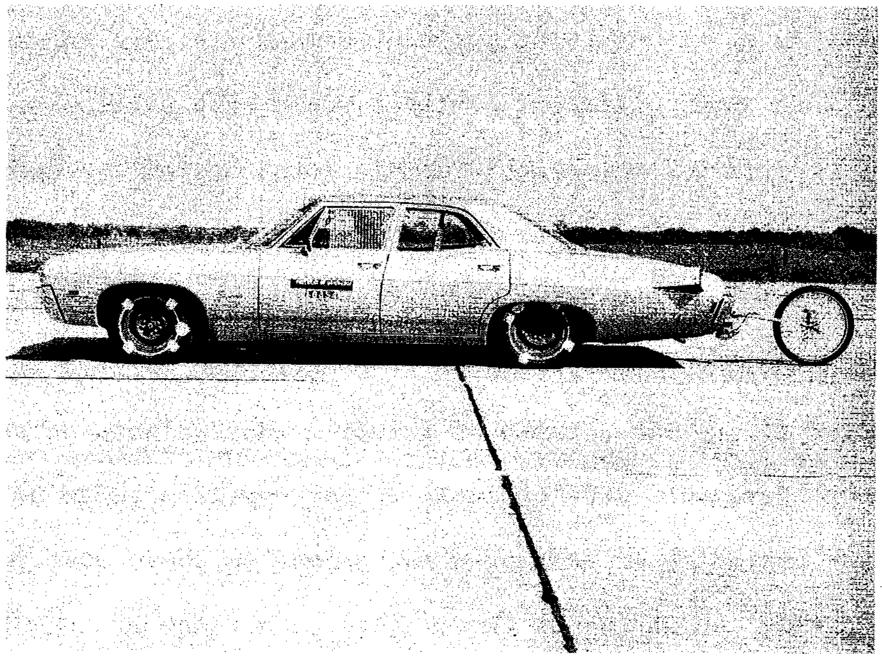
Description of the Test Surfaces

The location of the Texas A&M Research Annex on property that had previously been a jet trainer airfield permitted a wide choice in the specific location of the various test pavements. The study called for the design and construction of four different surface textures produced from selected aggregates and a single grade and type of asphalt cement. A fifth surface was required in the program which consisted of

TABLE B-1

CATEGORIZATION OF TIRE SETS TESTED

	Bias Ply	Radial	Wide Oval	Snow	Police	SAE	Wide Slicks	Total
New	20	8	12	11	2	4	1	58
Mileage Worn	13	8	0	0	0	0	0	21
Random Rerun (new and mileage worn)	5	4	2	2	1	2	0	16
						Total		95



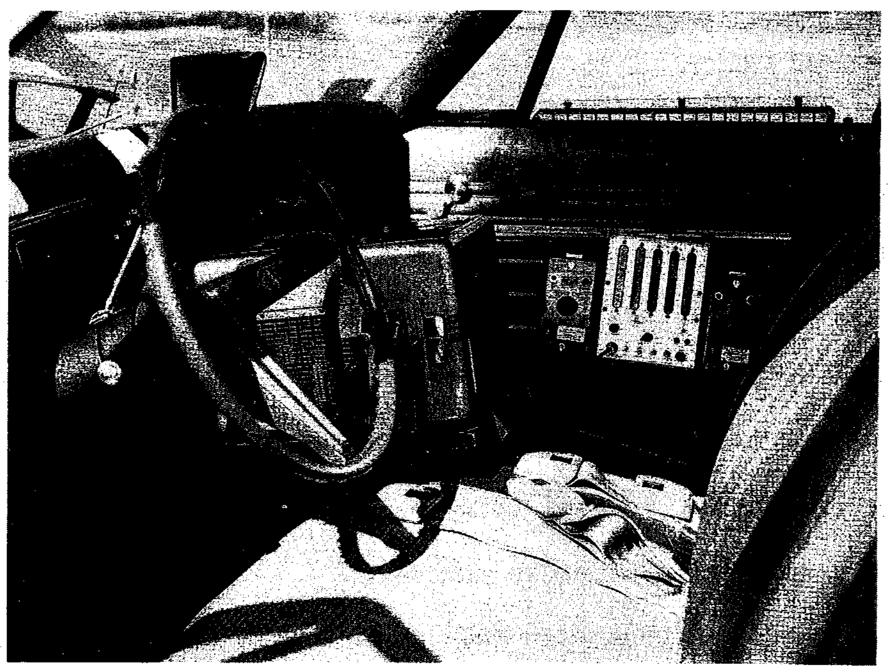


Figure B-1 Tire Test Vehicle

selected portions of the existing portland cement concrete runways. It was expected that these different surfaces would have particular coefficients of friction which would remain constant for the duration of the study. Figure B-2 illustrates the cornering test pavements.

The preparation of the existing portland cement concrete pad consisted of a thorough cleaning. The other surfaces were to be designed and constructed to provide a range of friction coefficients between 0.20 and 0.60. Pavements were produced which covered a range of 0.22 to 0.64 at the beginning of testing. The spread in these coefficients was reduced during the course of the project to a range of 0.24 to 0.55. The history of friction coefficients over the period of testing is presented in Figure B-3 for each test pavement.

Skid Trailer Measurements

The friction values shown in Figure B-3 were obtained with the Texas Highway Department Skid Trailer, (see Figure B-4), run with standard ASTM test tires. The source of water for wetting the pavements was a 4,000 gallon water truck complete with spray bars and a controlled pumping system capable of producing a uniform flow and distribution of water (see Figure B-5). Two passes (one in each direction) were made by the water truck prior to each skid measurement run. The skid trailer's self-watering system was not used.

Friction determinations were made at 20 and 30 mph. For each speed, a straight pass to the north and a straight pass to the south were made on two separate portions of each test pad. The values shown in Figure B-3 represent averages of these four measurements.

Figure B-2 Tire Test Surfaces

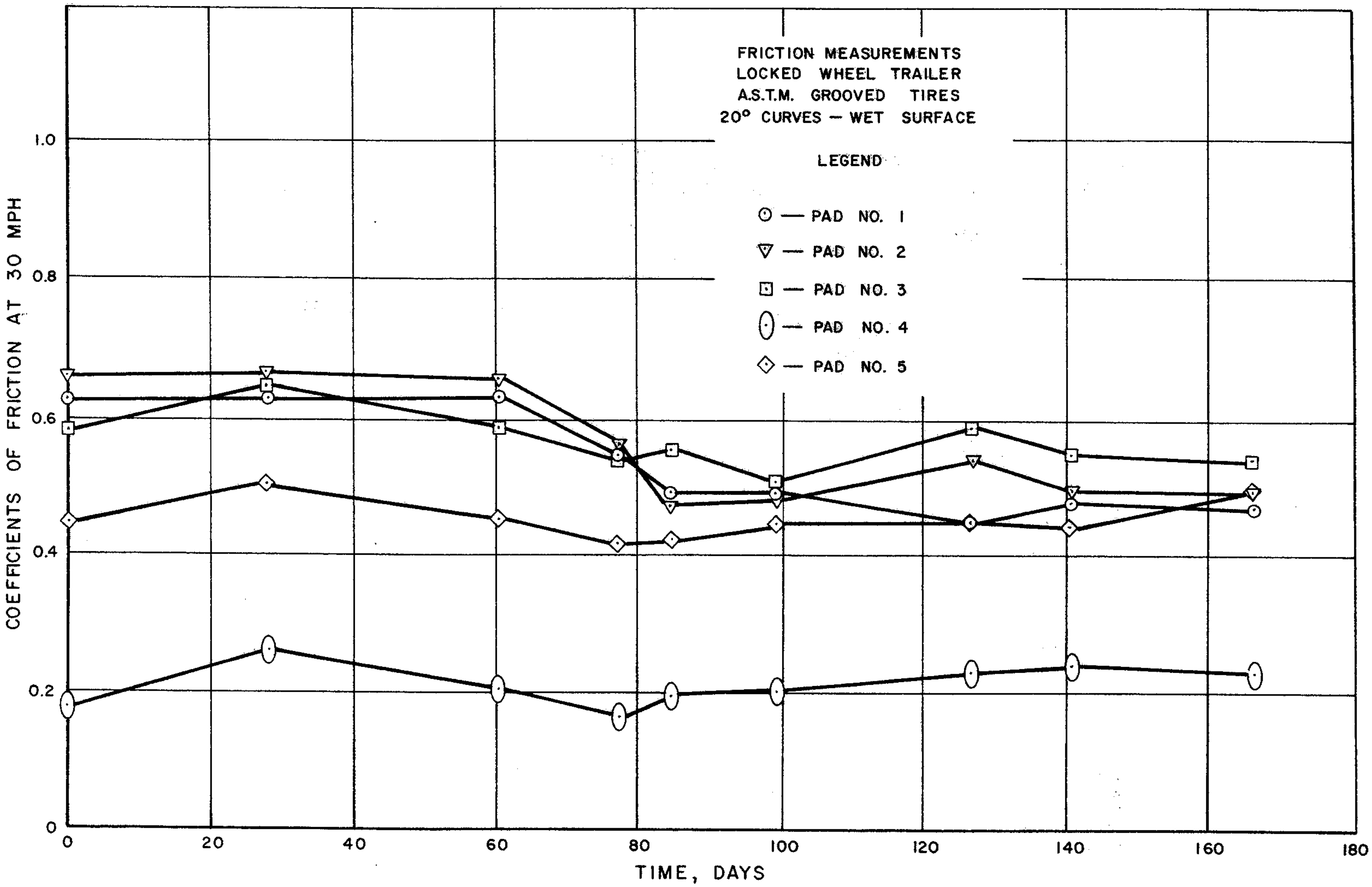


Figure B-3. History of Friction Coefficients Over the Period of Testing (4).

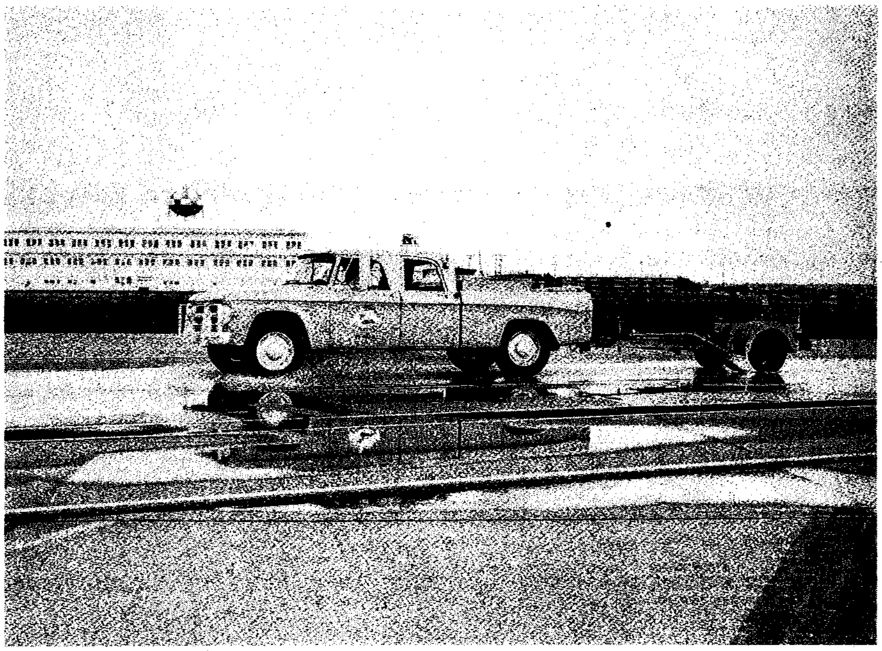


Figure B-4 Texas Highway Dept. Skid Trailer

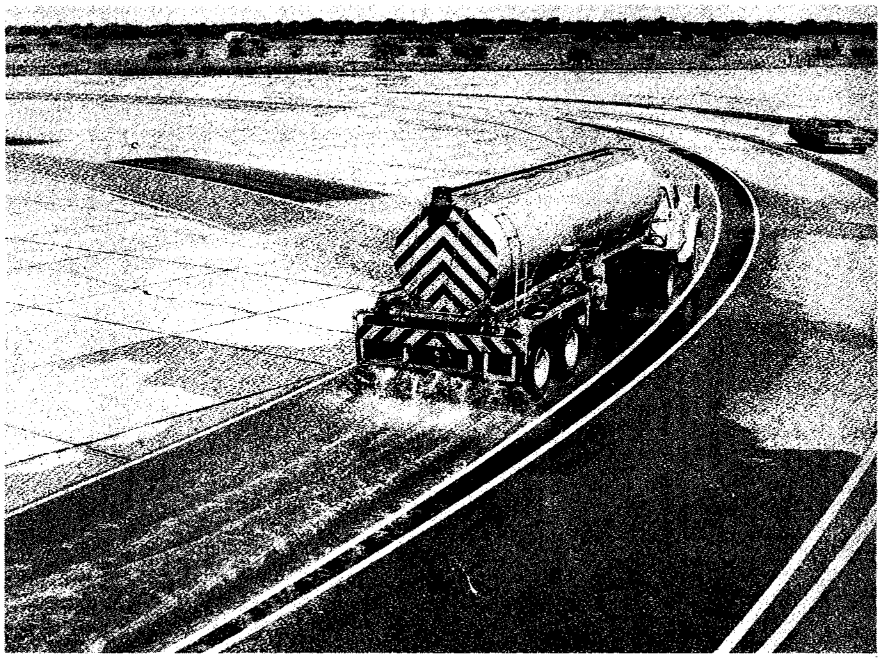


Figure B-5 Watering Truck

Spin-out Tests

The purpose of conducting the spin-out tests was to determine the lowest speed at which the vehicle would consistently spin-out on a wet pavement for each set of tires. This required the driver to make from three to ten runs for a single determination of spin-out speed so that at least two spin-outs occurred at one particular speed and no spin-out occurred at a speed of one mph. lower. The watering procedure employed was exactly the same as that used for the skid trailer measurements. Figure B-6 shows the test vehicle traversing a cornering maneuver on a test pavement.

The results of the spin-out tests are presented in Figures B-7 through B-11. These figures are frequency histograms of spin-out speed for each pavement-tire type combination. Listed on each histogram is the average spin-out speed, the appropriate skid number for the pavement, and the computed spin-out speed using the skid number in the standard cornering model. Based on the speed of spin-out for these geometric conditions, Figure B-12 show the relation between spin-out speed and the apparent (computed) friction demand.

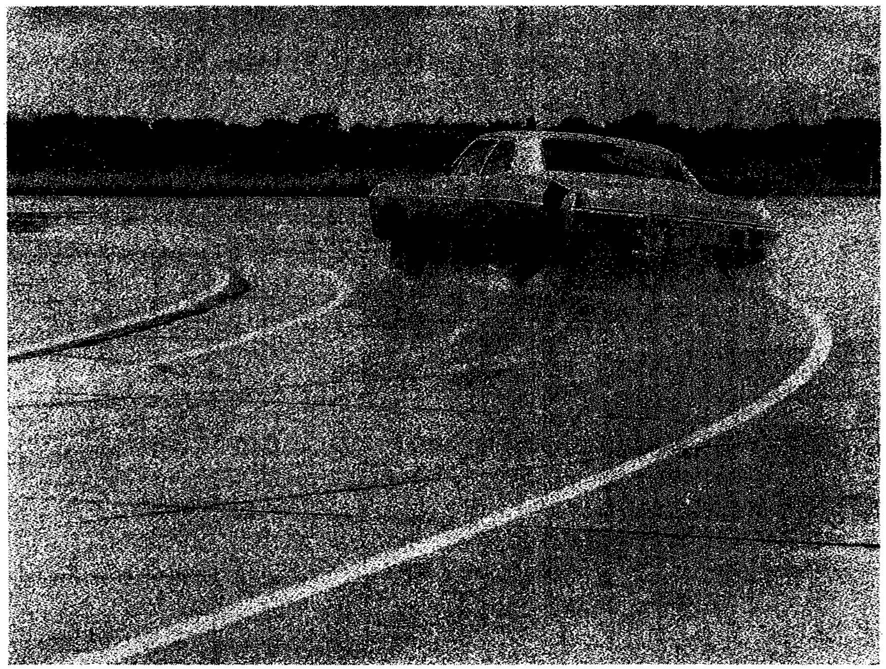


Figure B-6 Vehicle Cornering Test

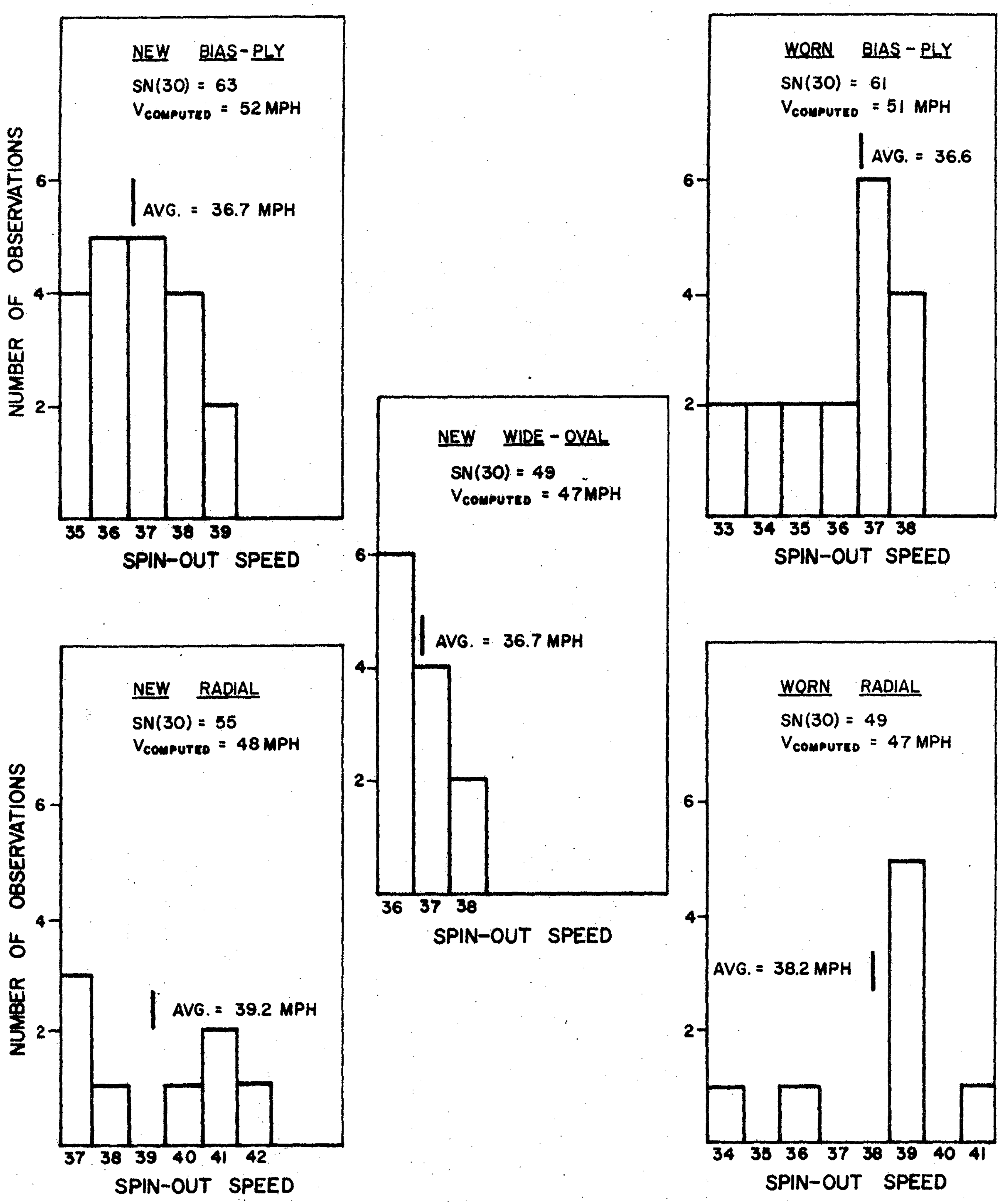


Figure B-7. Spin-out Speed Distribution on Pad 1.

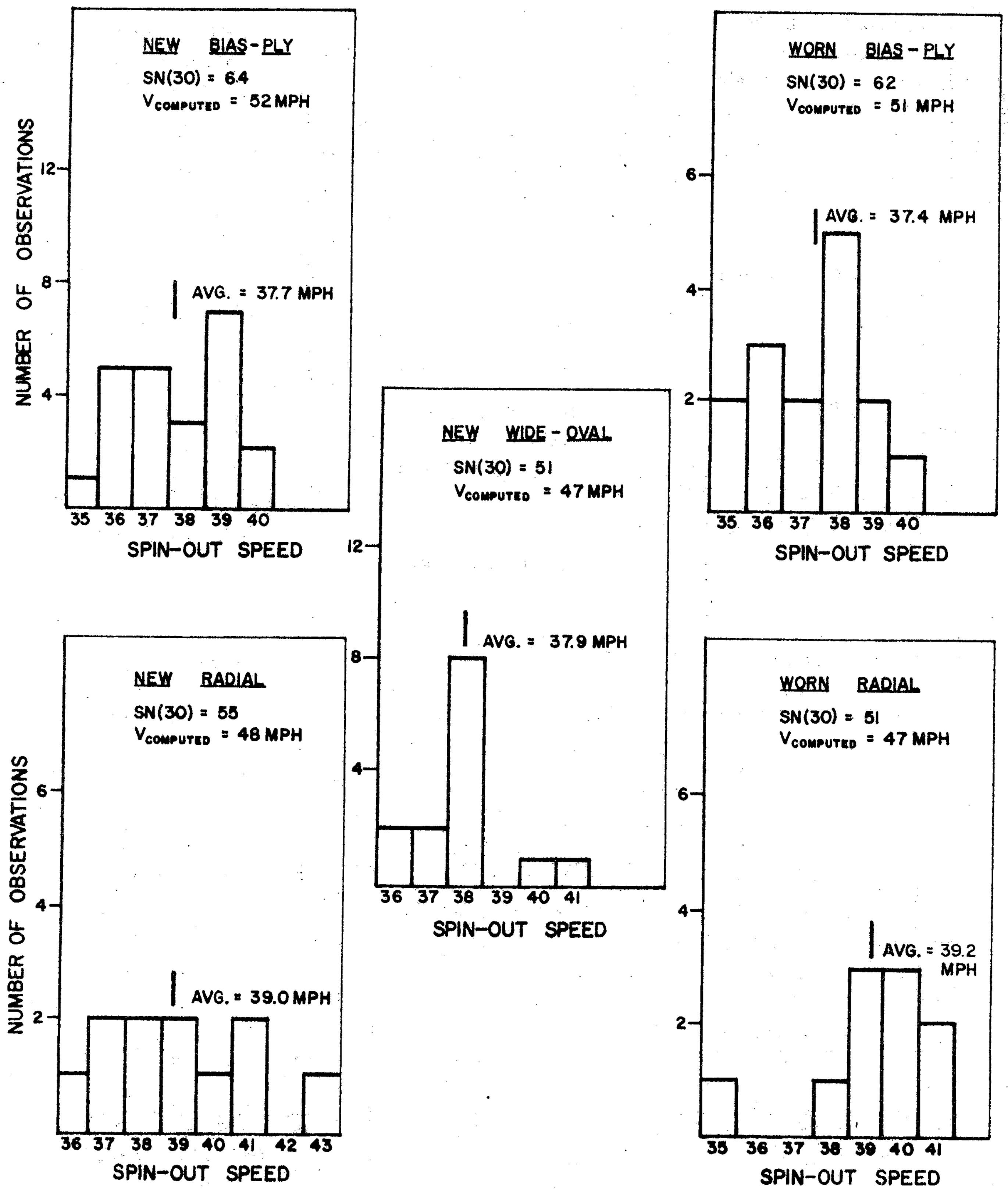


Figure B-8. Spin-out Distribution on Pad 2.

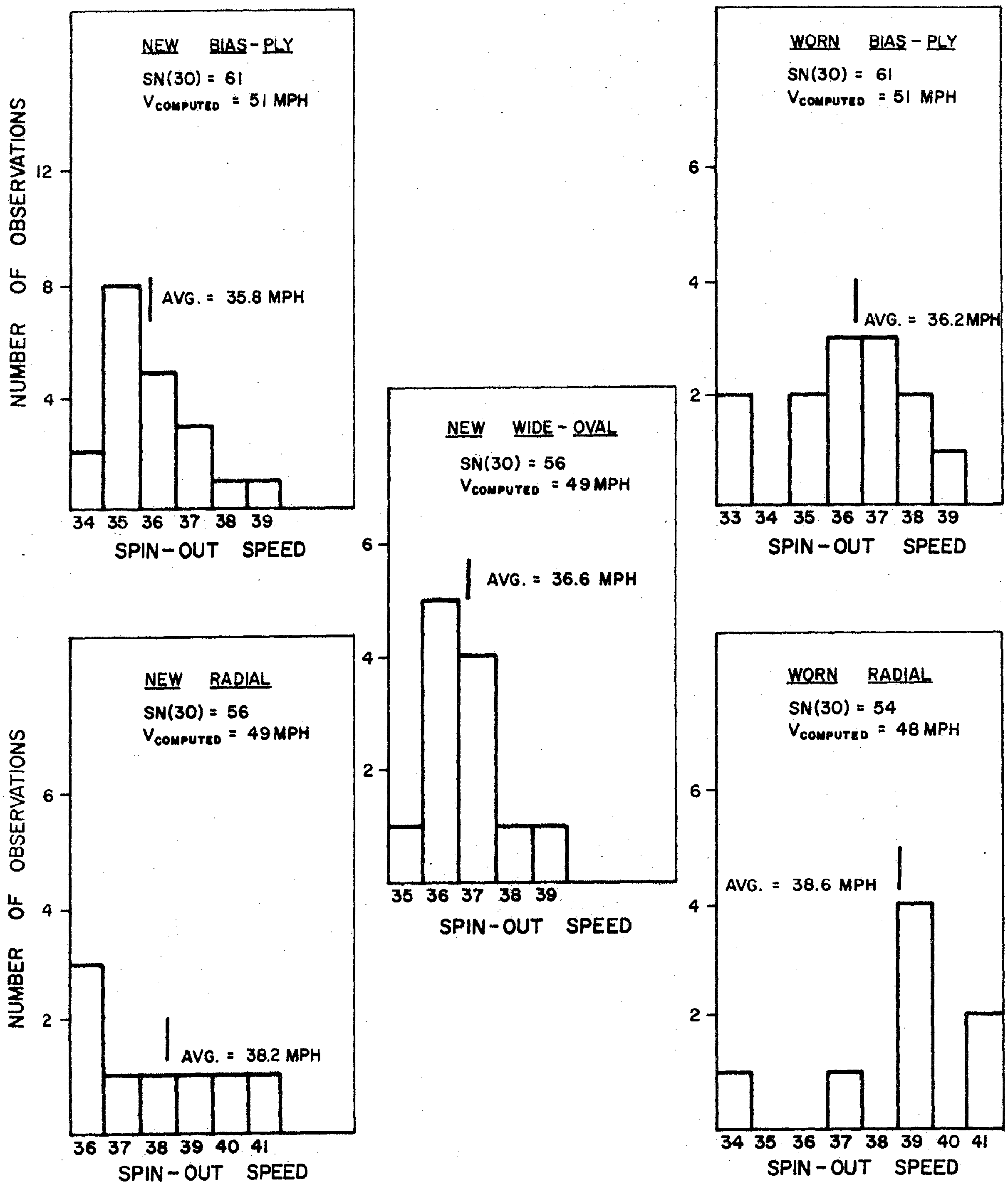


Figure B-9. Spin-out Distribution on Pad 3.

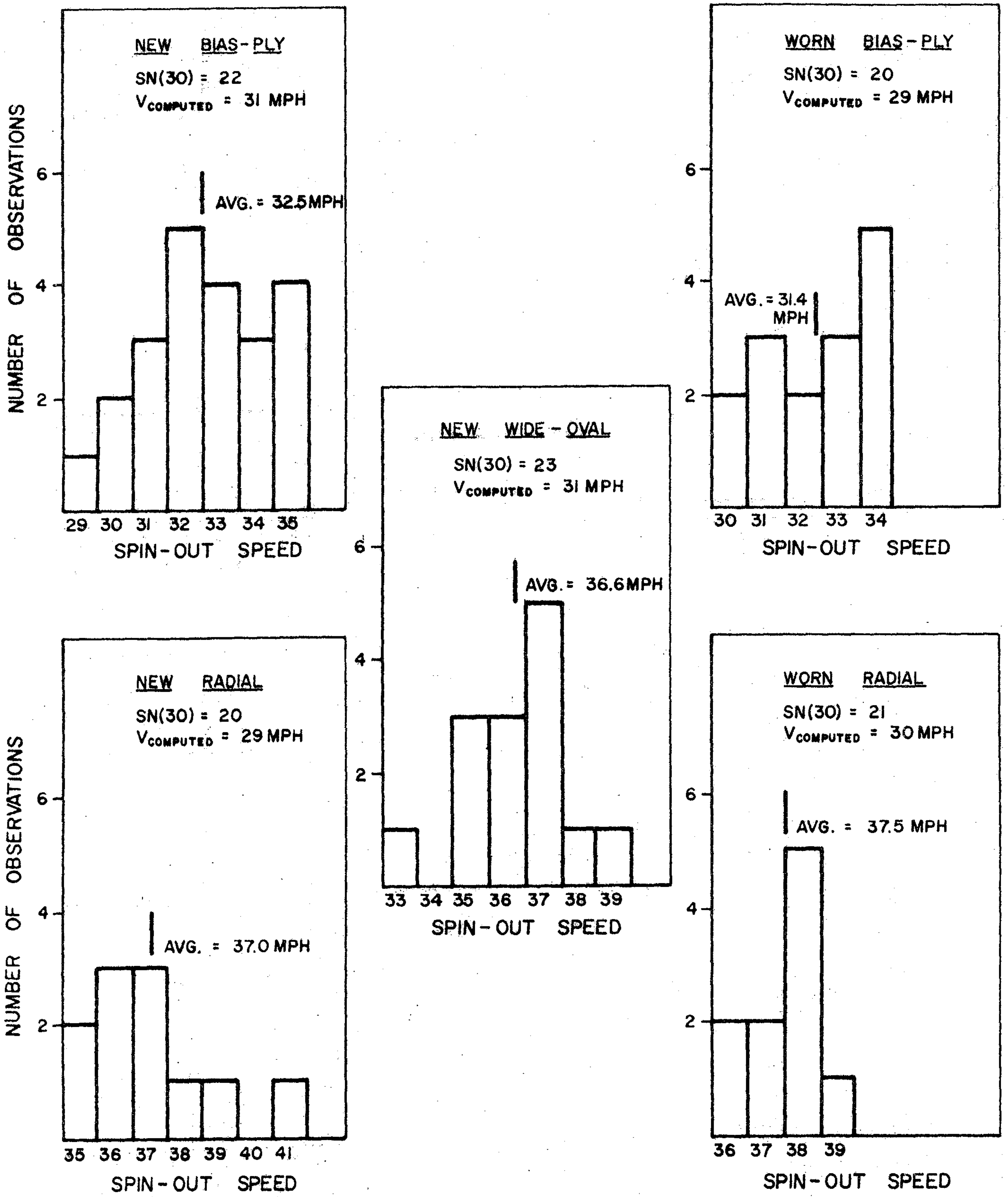


Figure B-10. Spin-out Speed Distribution on Pad 4.

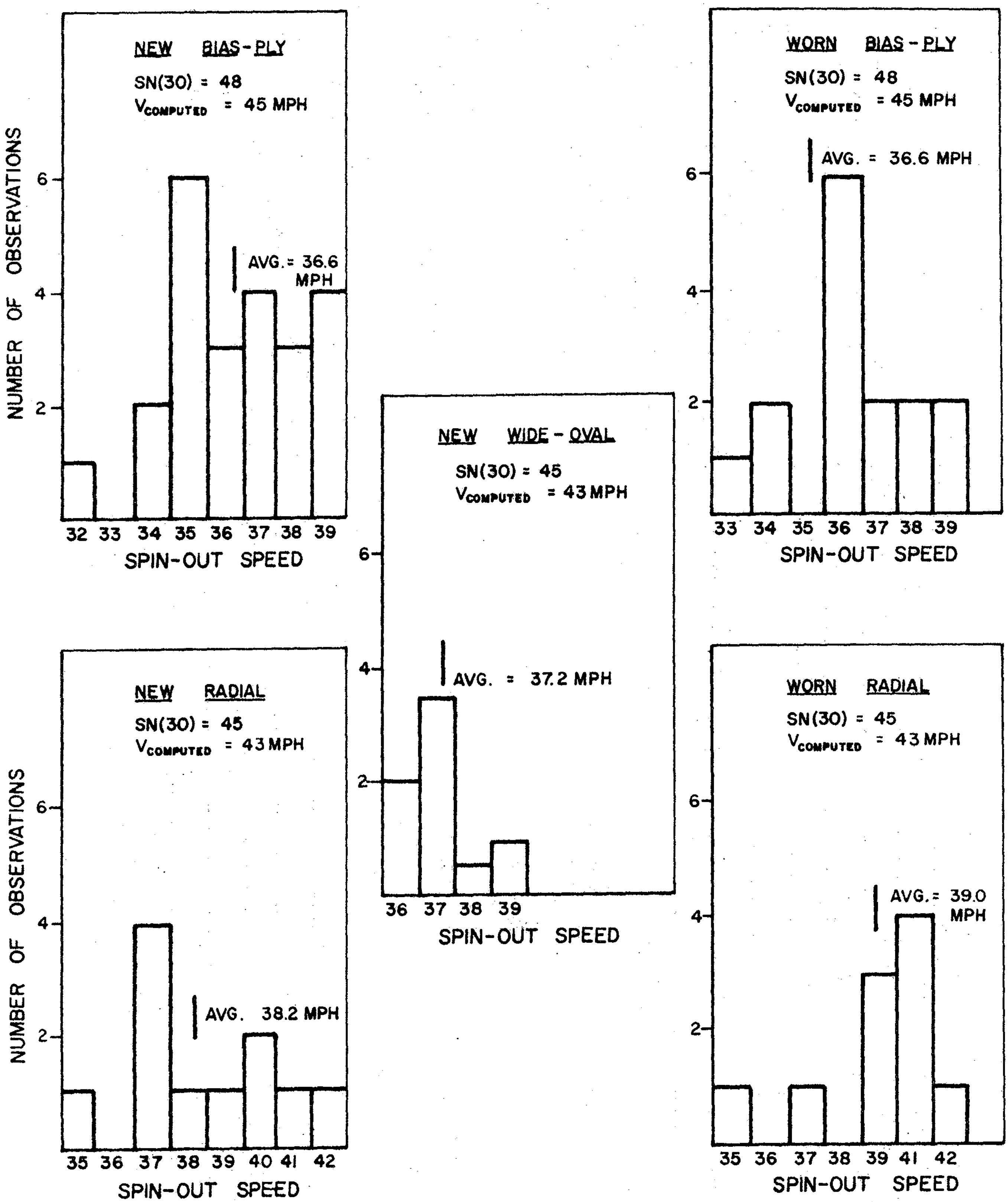


Figure B-11. Spin-out Speed Distribution on Pad 5.

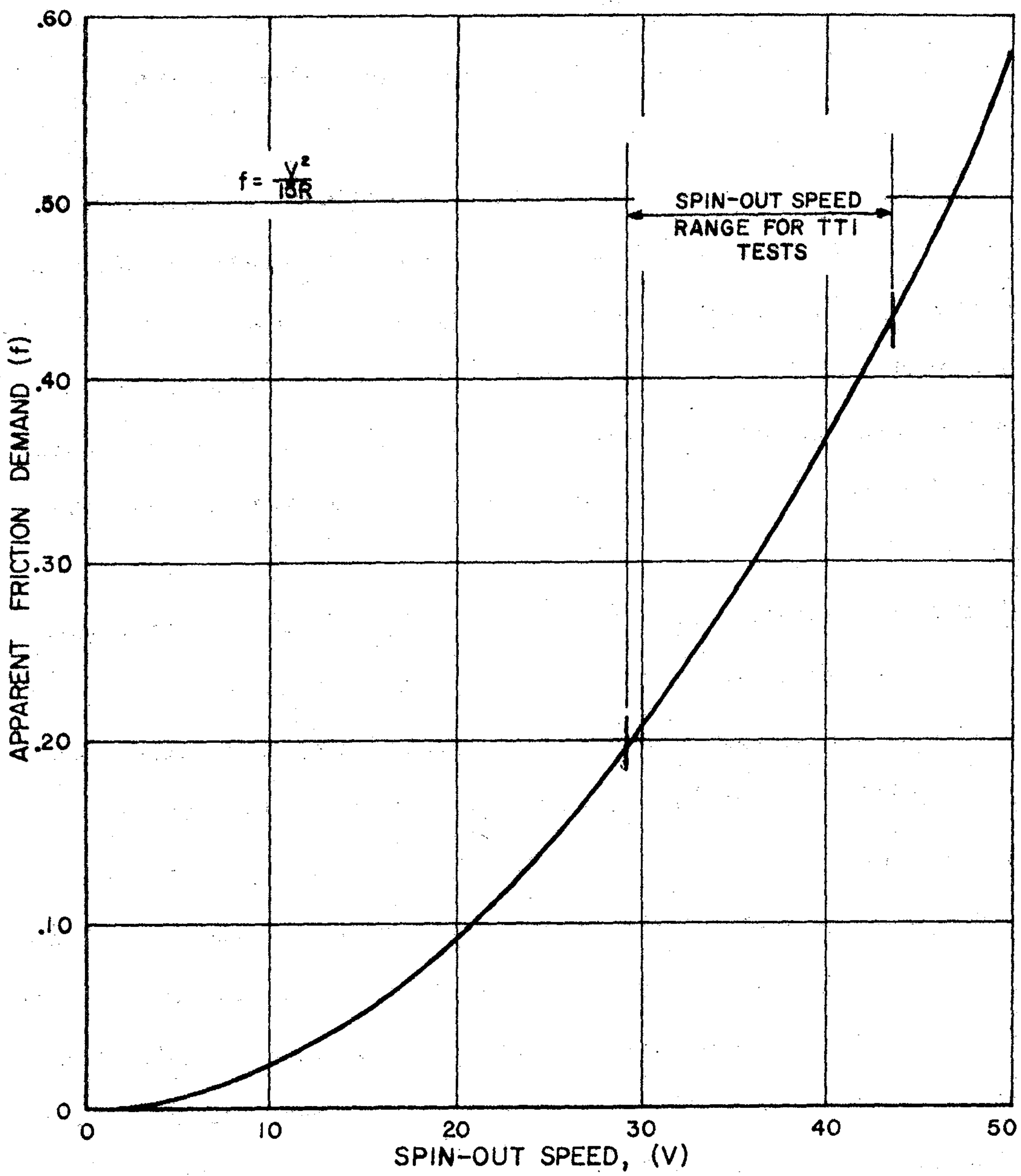


Figure B-12. Spin-out Speed Versus Apparent Side Friction Demand for 20-Degree Test Curves.

Appendix C

Analysis of Friction Demand in Passing on Highway Curves

This analysis is conducted in order to validate the possibility of a friction demand greater than that encountered when a vehicle is cornering at a rate equivalent to the degree of highway curve. For the purpose of discussion, three passing conditions will be considered: (1) 80-mph passing vehicle, 70-mph passed vehicle; (2) 70-mph passing vehicle, 60-mph passed vehicle; and (3) 60-mph passing vehicle, 50-mph passed vehicle.

The time-distance relationship assumed by the AASHO Policy (see Figure C-1) are employed in this analysis, with one modification. It is assumed here that all of the initial lateral movement takes place during the $d_2/3$ phase of the passing maneuver. The distance, d_2, is obtained by entering the graph of Figure C-1 with the passing speed.

The assumptions employed in the passing path are illustrated in Figure C-2. The passing vehicle is assumed to travel from the centerline of lane 1 to the centerline of lane 2 (a total lateral distance of 12 feet). At point A, the passing vehicle assumes a circular path which has a greater degree than the highway curve. At point B, the passing vehicle crosses the centerline, having traveled half of the $d_2/3$ distance. At point B, the passing vehicle assumes a circular path which is tangent to the arc AB and tangent to lane 2 centerline at point C. The outside passing return is assumed to have this same path. The outside initial passing maneuver and the inside passing return maneuver would be the reverse path of the above.

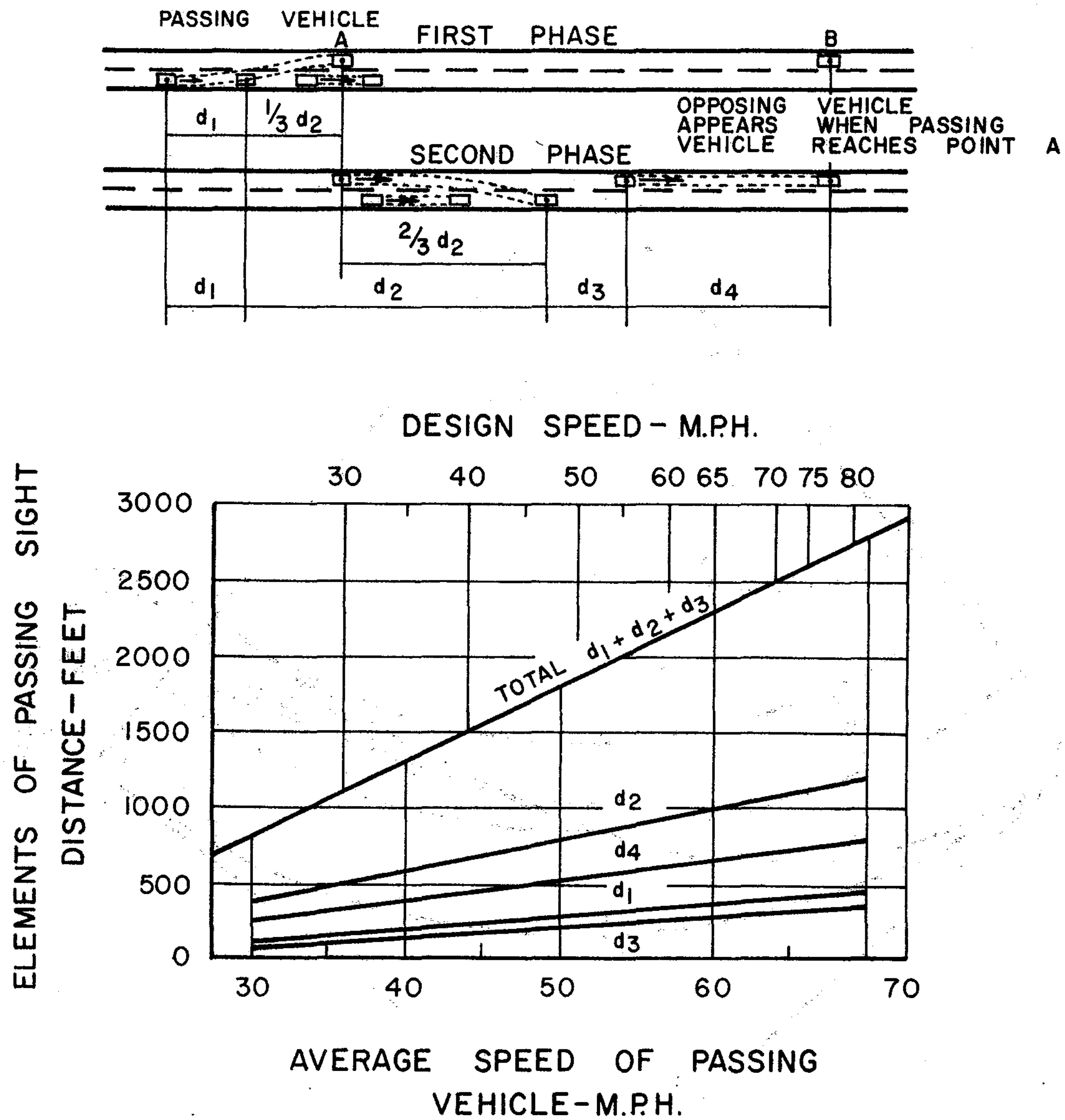


Figure C-1. Speed Distance Relationships for Passing on Two-Lane Highways - AASHO Policy (2).

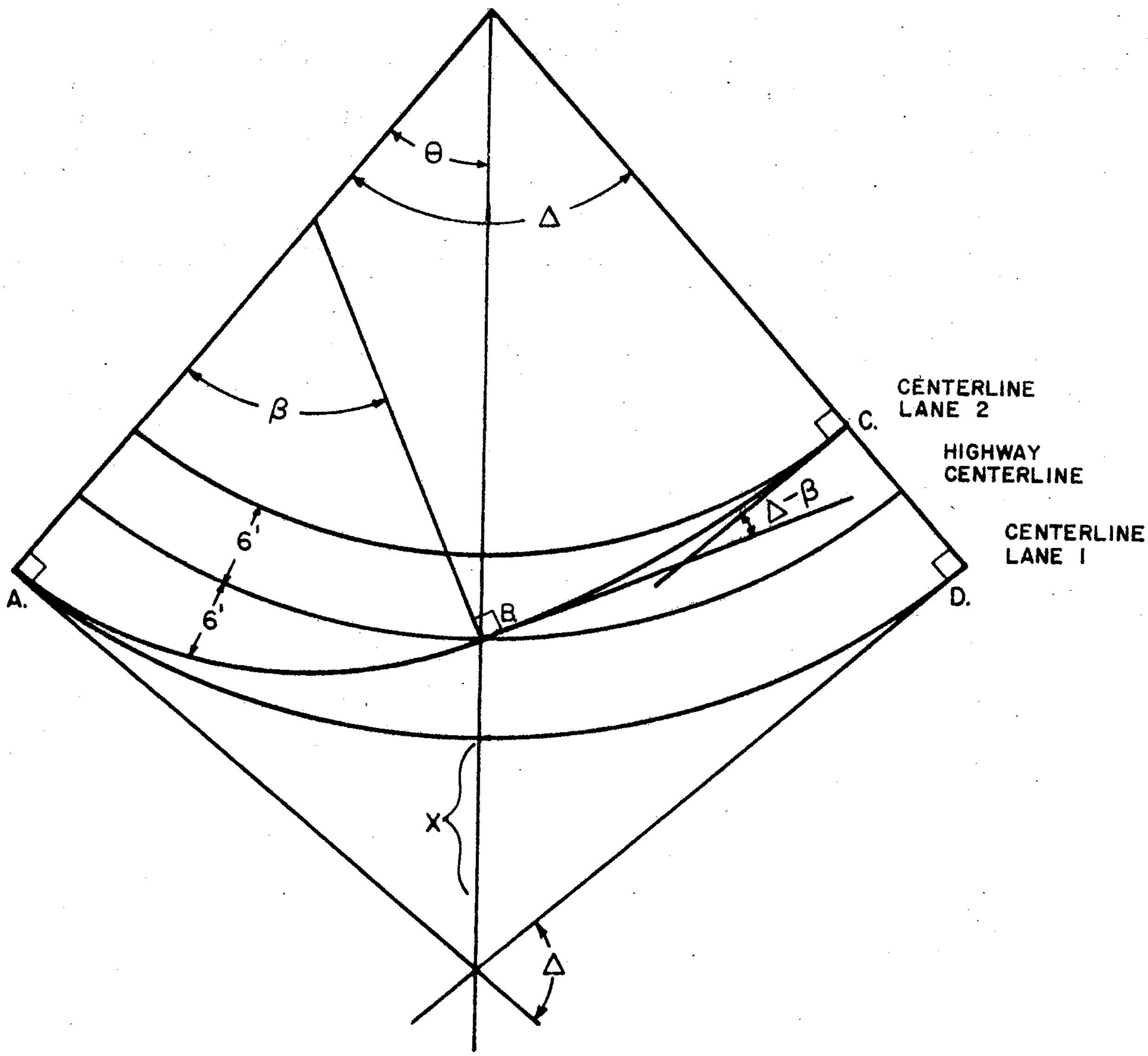


Figure C-2. Path Assumptions for Passing on Highway Curves.

By employing the approximations that the arc AD is equivalent to the passing path, AC, and that the degree of arc AD, is equivalent to the degree of highway curve, the following mathematical approximations can be employed to calculate the degree of curve in the passing maneuver.

$$\Delta = \frac{d_2/3\ D_1}{100}$$

$$\Theta = \Delta/2$$

$$X = \frac{d_2}{6} \tan \frac{\Theta}{2}$$

$$\beta = 2 \tan^{-1} \frac{6 + x}{d_2/6}$$

$$D_2 = \frac{100\ \beta}{d_2/6}$$

$$D_3 = \frac{100\ (\Delta-\beta)}{d_2/6}$$

where

Δ = The central angle of circular curve with arc length = $d_2/3$

D_1 = Degree of highway curve

Θ = Central angle subtended by arc length of $d_2/6$

X = Deflection distance from highway curve to tangent line in feet

β = Central angle of first passing curve

D_2 = Degree of first passing curve

D_3 = Degree of second passing curve.

Table C-1 shows the degree of passing curve, the maximum passing friction demand (D_2 always promotes the higher friction demand), and the excess of friction demand over the design level for various AASHO Policy design curves assuming that the passing vehicle travels at design speed.

The AASHO Policy passing distance considerations employ a 10-mph speed difference between the passing and the passed vehicles. It is noteworthy that, if the speed difference exceeds 10-mph and if the highway curve is of low degree, the second curve in the passing maneuver may be reversed in direction. In this case, the superelevation would increase rather than decrease the friction demand.

TABLE C-1

FRICTION DEMAND IN THE PASSING MANEUVER

Design Speed	Design e	Design f	Design D_1	Passing D_2	Passing Friction Demand	Passing Friction in Excess of Design Level
60	.06	.13	4.5	7.00	.23	.10
60	.08	.13	5.0	7.50	.23	.10
60	.10	.13	5.5	8.40	.25	.12
60	.12	.13	6.0	8.50	.22	.09
70	.06	.12	3.0	4.10	.21	.09
70	.08	.12	3.5	4.90	.21	.09
70	.10	.12	4.0	5.65	.22	.10
70	.12	.12	4.0	5.65	.20	.08
80	.06	.11	2.5	3.75	.22	.11
80	.08	.11	2.5	3.75	.20	.09
80	.10	.11	3.0	4.20	.21	.10
80	.12	.11	3.0	4.20	.19	.08

Appendix D

Derivation of the Cornering Model for Combined Horizontal and Vertical Curvature

Employing the same considerations as related in Appendix A, the equations of motion are:

$$[\Sigma F_n = m\bar{a}_n] \qquad P \sin(\theta+\alpha) = \frac{W}{g} \frac{v^2}{r_h}$$

$$[\Sigma F_v = m\bar{a}_v] \qquad P \cos(\theta+\alpha) = W + \frac{W}{g} \frac{v^2}{r_v}$$

where

r_h = radius of horizontal curve, in feet

r_v = radius of vertical curve, in feet

Dividing gives

$$\tan(\theta+\alpha) = \frac{v^2}{gr_h \left(1 + \frac{v^2}{gr_v}\right)}$$

since

$$1 - \tan\theta \tan\alpha \approx 1$$

$$\tan\theta = e$$

$$\tan\alpha = f$$

therefore

$$e+f = \frac{v^2}{gr_h \left(1 \pm \frac{v^2}{gr}\right)}$$

substituting speed, V, in mph

$$e+f = \frac{V^2}{r_h \left(15 \pm \frac{V^2}{r_v}\right)}$$