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PARAMETERS AFFECTING STOPPING SIGHT DISTANCE

FINAL REPORT

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TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

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ABSTRACT

Stopping sight distance (SSD) is a safety criterion in highway design that has major impact on construction costs. Because of recent significant changes in the characteristics of the vehicle fleet it was judged desirable to reevaluate the major elements upon which SSD is based.

Literature reviews and/or original research were conducted on the questions of driver eye height, roadway obstacle height, driver perception-response time, vehicle braking distances, accident experience at limited SSD locations, dry and wet pavement speeds, and mathematical relationships between SSD and geometric design elements. It was found that driver eye height should be lowered to 40 in. from the current 42 in. Obstacle height (currently 6") is a difficult criterion to study. However, with cars becoming smaller, there is merit in lowering obstacle height to 4" to assure clearance of vehicles passing over and reduce the likelihood of damage to or loss of control of vehicles impacting the obstacle. Driver perception-response time (currently 2.5 seconds) was found to be adequate. The results of the braking studies made it clear that the current guidelines for braking distance should be increased by 20 feet at 30 mph to 550 additional feet at 80 mph if controlled stops with poor tires on wet surfaces are to be achieved. Speed data were analyzed and it was again found that there was no appreciable difference between wet and dry pavement speeds of travel. Several limited SSD locations

were found to have more accidents than similar sections with less restricted SSD. Design aids for use in horizontal and vertical curves SSD analysis were developed.

Recommendations for the resulting modifications to SSD requirements are described and discussed and the implications for the geometric design of horizontal and vertical curves and safety explored.

SUMMARY

The concept of sight distance is one of the most important criteria in highway design. It affects safety, vehicle operations, and has a major impact on construction costs. Because of this it is important that the elements that determine sight distance be as accurate as possible.

This project was concerned with the problem of stopping sight distance (SSD). In it, new research and reviews of earlier research were used to examine the following elements of the general SSD model as applied to geometric design:

1. Driver eye height.
2. Obstacle height.
3. Driver perception-response time.
4. Vehicle braking distance.
5. Initial vehicle speed.

When the first SSD criteria were set up, driver eye height was established at 4.5 ft (54 in.). Over the years, as cars were made lower, the assumed driver eye height was reduced. It now stands at 3.5 ft (42 in.). The studies carried out as part of this investigation found there had been little change in the lower percentiles of the distribution of driver eye height in recent years--nor is there likely to be much change in the future. On a basis of the available data it is recommended that the driver eye height component of SSD be reduced to 40 in., which will accommodate all but 4 percent of the driving population.

Obstacle height was first set equal to driver eye height (4.5 ft, or 54 in.). It was subsequently reduced to 4 in., then increased to 6 in. Although it is generally agreed that a zero-height obstacle is preferable, this would result in a considerable increase in construction costs on crest vertical curves compared to a 6-in. obstacle. Therefore, the 6-in. obstacle was selected as a reasonable compromise between costs and driver visibility.

Construction costs are about the only objective measure available for assessing obstacle height. Accident costs associated with various levels of SSD are not available. However, with the general downsizing of the vehicle population there is merit in reducing obstacle height as well. A smaller obstacle is less likely to damage or deflect today's smaller cars if impacted and is also less likely to snag the underside should the vehicle pass over it. Hence, it is recommended that consideration be given to reducing obstacle height to 4 in.

Driver perception-response time is a concept that covers four steps. The driver must detect an obstacle, identify it as a significant hazard, decide what action to take, and put that decision into effect. The interval currently assumed for this entire process is 2.5 sec.

There have been a great number of attempts to estimate perception-response time either by research or surveys of available data. For a variety of reasons, none of these are entirely satisfactory. In the study carried out as part of the current investigation, perception-response time was measured for subjects unexpectedly confronted with an obstacle as they crested a hill. It was found that for all subjects, young and old, male and female, the 95th percentile perception-response time was about 1.6 sec.

It is still necessary to correct the results of the study to account for road users who are more fatigued, less alert, or under the influence of drugs to some degree. A 50 percent increase in perception-response time is thought reasonable to allow for these factors, producing a value of 2.4 sec. This is close to the currently used time of 2.5 sec. Therefore, it is recommended that 2.5 sec continue to be the standard perception-response time.

The braking distance element in the SSD model deals with cars and trucks having worn, but legal, tires attempting to stop on wetted pavement of relatively poor quality. Assumptions of braking distances are often based on theoretical maxima. The problem is that the driver may not be capable of controlling braking to the degree required to achieve the theoretical maxima. As part of this program studies were carried out to measure the ability of professional truck drivers to stop trucks under various conditions of load on wet pavement. These data were combined with other information from the literature to produce the required estimates.

The results indicate that for controlled stops from higher speeds by cars with worn tires on slippery roads, the required distances are considerably greater than those allowed in the current SSD model. The increase is from 20 to 85 ft for design speeds of 30 to 50 mph. At speeds of 60, 70, and 80 mph, additional lengths are 190, 350, and 550 ft, respectively.

For locked wheel stops on a poor wet road, trucks require stopping distances that are approximately 1.2 times those attained by passenger cars. For controlled stops (in which the driver modulates the brakes to prevent the vehicle from spinning around and to maintain steering control), trucks require stopping distances that are approximately 1.4 times those required for passenger cars. However, because of the differences between the eye-heights of truck and car drivers, truck drivers have stopping sight distances on crest vertical curves that are approximately 1.35 times longer than those achieved by drivers of passenger cars. Hence, for locked wheel stops (which may be disastrous if attempted at high speed), the sight distances provided for passenger cars are more than adequate for trucks, although increases in sight distances of about 10 percent over those required to allow cars to make controlled stops would be needed to allow heavy trucks to make controlled stops.

Studies were made of speeds recorded on wet and dry pavements at a number of rural sites. It was found that there is no important difference in the speed distributions and that the current AASHTO policy of stopping from the same speed on both wet and dry pavements remains appropriate.

Numerous mathematical and sensitivity analyses involving SSD and geometric design were made. Among these were original development of a relationship making the plotting of sight distance graphs easier and development of simpler and original relationships for treating the required lateral clearance on horizontal curves.

A study of accidents was made at 10 pairs of matched nearby sites, one of which had limited SSD because of vertical curvature. It was found that the accident experience at such locations was significantly greater than at the sites where adequate SSD existed.

CHAPTER ONE

INTRODUCTION AND RESEARCH APPROACH

PROBLEM STATEMENT AND RESEARCH OBJECTIVES

It is fundamental in Highway Engineering that nothing should be done by design that would unreasonably increase the risk to the roadway user. Among the most basic of these concerns is the notion that the roadway and its associated structures should not hide significant hazards from a driver until he/she is so close as to be unable to deal with them effectively. This is the concept of sight distance.

Sight distance not only is a basic safety criterion, it has a major impact on construction costs as well. Sight distance determines safe stopping and passing distances, has a pronounced effect on the required lengths of horizontal and vertical curves, and the placement of structures such as bridge piers.

Significant changes have taken place in the driver and vehicle parameters on which sight distance is based. Because sight distance is a major factor in the cost and safety aspects of highway geometric design as well as traffic operations, there is a need to evaluate the importance of all significant parameters related to current roadway, vehicle, and driver characteristics. There is also a need to update vehicle performance data related to acceleration and deceleration rates; the current highway design criteria are based on vehicle fleet characteristics over 10 years old.

The primary objective of this research is to evaluate those parameters affecting stopping sight distance (SSD), the design element related to the driver decision to stop in response to a hazard in the roadway. These parameters include:

1. Perception and response time.
2. Driver eye height.
3. Height of an object in the roadway.
4. Braking distance as affected by tire performance, brake system performance, pavement skid resistance, initial speed, and grades.

The research results will determine whether the criteria proposed in the draft report from NCHRP Project 20-7, Task 14, "A Policy on Geometric Design of Highways and Streets," (1) are supportable under today's conditions or that the parameters have changed substantially enough to warrant modification in SSD criteria. In the latter event, the study will supply data and recommendations for the required modifications.

RESEARCH APPROACH

In order to accomplish the desired objectives, seven tasks were specified:

Task 1. Literature Review--A critical review of the literature was conducted related to SSD and acceleration/deceleration characteristics. Recent and ongoing research sponsored by FHWA and others was also reviewed. The adequacy of available data for evaluating each parameter was determined and documented. The aim was to use existing data to the maximum extent possible. The output of this task is contained in the project interim report (2), dated August 1982.

Task 2. Definition of Design Driver Eye and Roadway Object Heights--Design values for driver eye height and height of object on the roadway were determined. Criteria contained in the draft report from NCHRP Project 20-7, Task 14 (1), were evaluated in view of changes in vehicle mix and driving population. The sensitivity (i.e., effect on SSD) of incremental changes in each parameter was determined over a range of design speeds.

Task 3. Development of Recommendations for Driver Perception-Response Times--Recommendations for driver perception-response time related to a hazardous object in the roadway were also developed. The research considered: (a) contrast between the object and its background; (b) object height and width; (c) conditions affecting visibility, e.g., ambient lighting and weather; and (d) conditions affecting driver performance, e.g., drugs, fatigue, and age.

Task 4. Development of Recommendations for Braking Distances--The specific performance characteristics of tires and braking systems for the current vehicle fleet, including autos and trucks (loaded and unloaded), were determined.

Effects of changes in vehicle mix, tread wear, pavement skid resistance, and grades on SSD were also considered.

Task 5. Vehicle Performance Characteristics (Acceleration and Deceleration)--Acceleration and deceleration characteristics for the current vehicle fleet were determined. This phase of the research was limited to a literature review and an evaluation of the results of the braking distance study described previously (Task 4). The results pertaining to acceleration characteristics are summarized in the project Interim Report (3) and are included in this report as Appendix F.

Task 6. Review and Analysis or Recommendations--In this phase of the work the output of Tasks 1, 2, 3, and 4 were reviewed and analyzed to produce a coherent series of recommendations about the parameters of interest. These data were compared with the criteria proposed in the draft report from NCHRP Project 20-7, Task 14 (1), to determine whether modifications in stopping sight distance criteria were desirable. The implications of these changes for geometric design and construction were explored.

Task 7. Preparation of a Final Report--The results of all work accomplished in Tasks 1 through 6, and the recommendations that followed from these findings have been summarized in this Final Report.

CHAPTER TWO

FINDINGS

PRESENT STATUS

Stopping sight distance (SSD) as defined by AASHTO in 1940, 1965, and 1971 (4, 5, 6,) is the minimum sight distance that will allow a vehicle traveling at or near the design speed to stop just before reaching an object in its path. It is one of the most fundamental elements controlling the geometric design of roads and streets.

The existing SSD model had its origins with a 1937 AASHO committee (4). The model has two components, the required SSD and the available SSD. The first, the distance required to stop, is established by considering driver perception and response time, initial vehicle speed and braking characteristics, as well as road gradient. The available SSD is based on any geometrical constraints to the clear line of sight from the driver's eye to an unexpected obstacle on the road. The location of the motorist's eye and object that the driver is to avoid by stopping his/her vehicle is considered. The height of the headlamps above the road is required for some applications.

In the dynamics of the required SSD model the vehicle is assumed to operate at a given speed, and the SSD is the sum of two distances. One of these is the perception-response distance (called brake-reaction distance (1), p. III-1), which is the driver perception-response time multiplied by the vehicle's initial speed. The second component is braking distance, which is based on the arithmetic mean deceleration of the braking vehicle from the assumed speed to a complete stop on a level surface. The environment is assumed to include a clear atmosphere, and the tire-road coefficient of friction is that found on wet roads. Snow and ice covered surfaces are not specifically considered in parameter selection.

Although every significant input to the model is stochastic, the model itself is deterministic and is based on values of the input parameters that have been drawn from near that end of the distribution that results in longer required SSD. Thus, a conservative value of required SSD is obtained. These input parameter values are "nearly all inclusive" (1, III-4).

Stopping is the ultimate way in which a collision can be avoided or the crash energy dissipation minimized if the stop cannot be completed. It has been universally accepted in the highway engineering profession that adequate SSD calculated as described above is always to be available for below-average motorists operating at an appropriate speed (4). There is a particular concern with the presence of stationary obstacles on the road. This makes it necessary to consider the line of sight from the motorist's eye to the obstacle across the inside of horizontal curves, over the crests of vertical curves and on sag vertical curves under overhead structures. After dark, the vehicle lighting system becomes important, particularly on sag vertical curves. At intersections, SSD is used to minimize the likelihood of collisions with other vehicles or trains by creating a clear sight triangle in the corners of the intersection so that converging vehicle operators can see each other in time to stop, if necessary.

Like other AASHTO policies, SSD has been periodically reviewed since its 1940 adoption. Changes were made in the 1954 (7) and 1965 editions of the Bluebook (5), in a 1971 special publication (8) and recently in the adopted draft of the next geometric policy edition (1). In the 1954 and 1965 revisions, small changes were made in some parameters (e.g., perception-response time was made constant at 2.5 sec, rather than linearly decreasing from 3.0 sec to 2.0 sec). The net result of these adjustments was little change in design values because the modifications had the effect of cancelling each other.

In 1971 (6) AASHTO endorsed about a 40 percent increase in required SSD. This resulted from using the full design speed rather than a lower average operating speed. However, the earlier values were not replaced. Instead, the 1971 values were called "desirable," and the earlier ones redesignated as "minimum."

The current AASHTO Policy (1, p. III-1) lowered the eye height to 3.5 ft and, otherwise, essentially "rounded up" the 1971 values slightly. Ranges are presented from the 1971 minimum to the 1971 desirable values, and these are called "lower" and "upper" values respectively. The implication of these changes for several basic situations are briefly reviewed in the following discussion.

Required SSD is related to horizontal alignment by the parameters of the curve and the location of objects "inside" the curve. The obstacles to vision are created by cut slopes, foliage and structures of all types. For example, for a 60-mph design speed, 3-deg horizontal curve, the minimum clearance from the edge of the lane to a bridge pier based on the lower SSD policy is 11 ft, a value that would usually be exceeded by shoulder width and other roadside surfaces clear of obstacles. However, the upper SSD policy requires a clearance of 22 ft from the edge of the pavement and this could be a controlling constraint in bridge pier location and span length. The cost implications of such a requirement are clearly substantial.

Crest vertical curve lengths vary significantly for the current lower and upper SSD values. For example, with a design speed of 60 mph and a grade change of 12 percent the two current values are 2,100 and 3,500 ft. Again cost differences are important.

Sag vertical curves have SSD problems under two conditions; after dark and when structures pass over the road. In the first case, a lower SSD-based vertical curve for a design speed of 60 mph and 10 percent grade change would have to be 1,300 ft long. Using the upper SSD, this curve is satisfactory for speeds of only 55 mph. A 1,600-ft long curve would be indicated for the 60-mph speed.

The 1965 AASHO sag vertical curve design parameters used to test structure clearance, i.e., a truck with an eye height of 6 ft, and an object height of 1.5 ft (5, p.524), have not been changed.

DRIVER EYE HEIGHT

It was desired to determine the distribution of driver eye height for the near-term population of drivers and vehicles from which desired percentile values could be selected to serve as the standard for SSD design. Experimental measurements in the field were determined to be beyond the scope of effort desired for this phase of the study. Therefore, an analytical approach previously used and recommended by Hammond of Ford Motor Company (9) was selected. As shown in Figure 1, this approach makes use of the SAE J941 eyellipse data, which provide vertical distances from the seating reference point (SgRP) to various population percentiles of eye height. The initial eyellipse data were generated from subject measurements of actual eye position using a variety of vehicles (10). These data have been modified and improved over the years to account for differences in seat back angle (11) and head rotation (12).

Essentially, the eyellipse is a locus of tangent cut-off points, such that for any line tangent to the 95th percentile eyellipse, for example, 95 percent of the eyes will be located on the ellipse side of the line (but not necessarily within the ellipse) and 5 percent will be on the other side of the tangent line. It was desired to determine

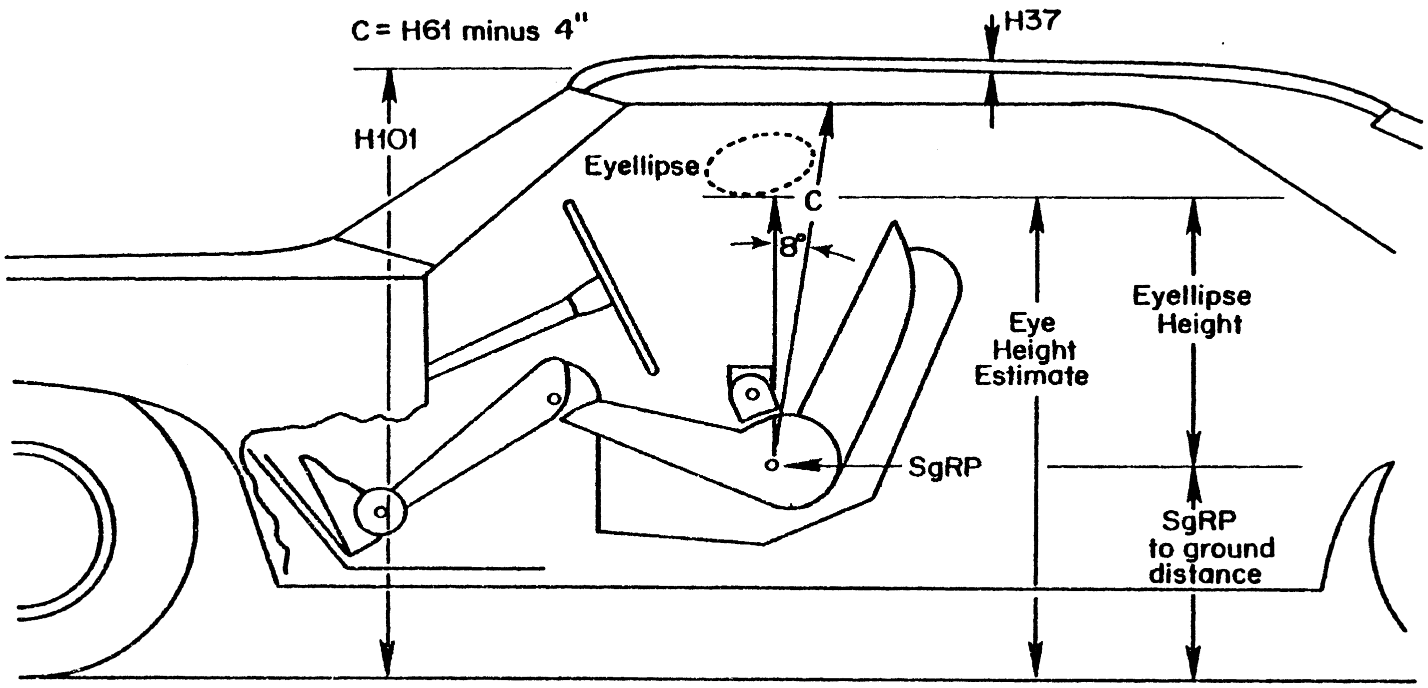


Figure 1. Illustration of specifications used to estimate SgRP height and driver eye height.

the vertical distance from the SgRP to various percentile eyellipses (see Fig. 2). This was accomplished by determining the distance from the SgRP to the horizontal tangent line to the various percentile eyellipsoids as illustrated in Figure 2. Table 1 gives the resulting percentile values for this distance.

In order to determine the eye-height-to-ground measure desired, it is necessary to add the SgRP-to-eyellipse distance, to the SgRP-to-ground distance A. This latter measure is vehicle specific and is not directly available. As recommended by Hammond and illustrated in Figure 1, this measure was calculated from other vehicle dimensions as follows:

$$A = H101 - (H61 - 4) \cos 8 - a$$

where:

H101 = vehicle height, as defined by SAE, in the "design attitude," i.e., loaded with fuel, passengers, etc.

H61 = effective head room, a measurement along a vector 8 deg rear of vertical from SgRP to the vehicle headlining, plus 4 in; and

a = adjustment factors estimated at 1.5", which include curvature of roof, headlining thickness, seat track adjustments and varying back angle.

SgRP distances to the ground were calculated for all vehicle models (both foreign and domestic) that comprise a

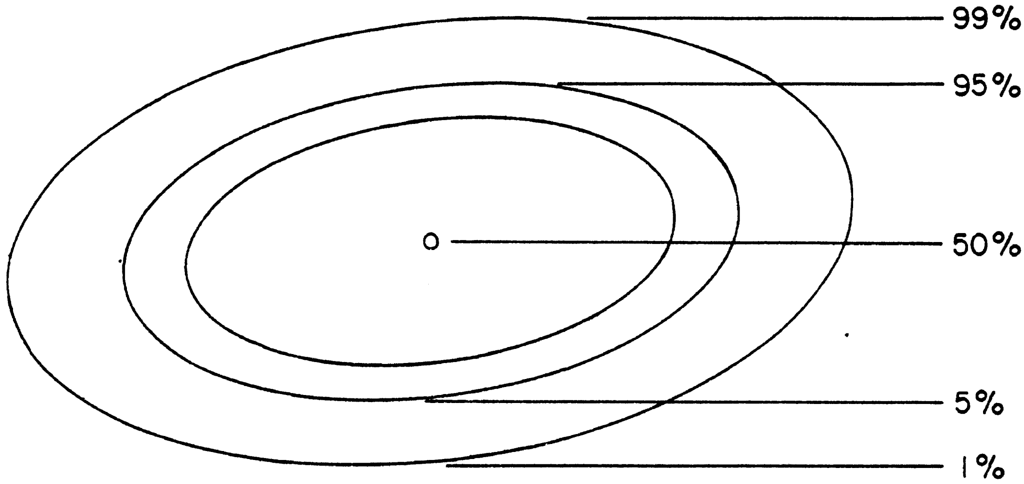


Figure 2. Percentile tangent lines used in measuring vertical distance of eyellipse to SgRP.

Table 1. Cumulative percentile eye-height values for eye heights above SgRP.

SgRP Eye Height (in.)	Percentile
22.2	1
23.0	5
23.3	10
23.6	15
23.8	20
24.1	30
24.4	40
24.7	50
25.0	60
25.3	70
25.6	80
25.8	85
26.1	90
26.4	95
27.7	99

measurable volume of the current sales market in the United States, as sold in 1981, according to the 1982 Wards Automotive Yearbook (13). These SgRP heights were ordered from smallest to largest, and a cumulative summation of all models sold was made from this list. SgRP heights for various selected percentiles (e.g., 5th, 10th, 20th, etc.) were calculated by linear interpolation between these cumulative percentages. Table 2 gives the resulting percentile values of SgRP heights above the ground.

The distribution of eye-height-to-ground distances along with various percentiles of driver eye height was calculated by assuming independence of the two distributions, noting that they were approximately normal, and appropriately combining them using the percentiles in Tables 1 and 2. The resulting percentiles, given in Table 3, provide an estimate of driver eye height based on both 1981 car fleet characteristics as given by SgRP-to-ground data, and driver population characteristics based on eyellipse-to-SgRP distance.

An estimate of SgRP height was then made for late in the decade. Projected sales for 1990 in small and large car categories were obtained (14). These data were based on weight of vehicles, and the 1981 vehicle weights were obtained (15). The 1981 distributions of SgRP were then weighted by small and large car fractions and used to develop an estimate for this factor. Details of the foregoing development are given in Appendix D.

Table 2. Percentile values for 1981 passenger-car SgRP heights above ground.

Ground SgRP Height (in.)	Percentile
15.1	1
16.2	5
16.8	10
17.1	15
17.4	20
17.6	30
18.1	40
18.3	50
19.3	60
19.7	70
19.9	80
20.1	85
20.5	90
21.0	95
21.9	99

Table 3. Eye heights for 1981 passenger cars.

Eye Height (in.)	Cumulative Percentile (%)
39.0	1.1
40.0	3.8
41.0	11.0
42.0	24.9
43.0	43.5
44.0	64.1
45.0	78.8

Table 4. SgRP heights for 1990 cars.

Ground SgRP Height (in.)	Cumulative Percentile
15.1	1
16.3	5
16.9	10
17.1	15
17.3	20
17.7	30
18.1	40
18.5	50
19.0	60
29.4	70
19.9	80
20.2	85
20.6	90
21.0	95
21.9	99

Table 4 presents the SgRP data for the 1990 estimate. Comparing Table 4 with Table 2 reveals that changes from 1981 sales will affect the eye heights given in Table 3 by less than 0.2 in. These values are higher than the 1981 values. Accordingly, the 1981 values are adopted as the basis for this study.

The current 42-in. standard is too high for almost 25 percent of the drivers while a value of 40 in. is adequate for all but 4 percent of the drivers. This value is accordingly used.

DRIVER PERCEPTION-RESPONSE TIME

Introduction

The purpose of the work carried out in this task was to develop data on driver perception-response time as a function of several variables and under realistic conditions.

The necessity of obtaining perception-response data under conditions that were as realistic as possible made it impractical to collect all the data of interest in one study. Instead, two field studies were carried out, one under conditions where the subjects were not expecting to encounter a roadway obstacle (surprise study) and the other under conditions where they were expecting such an encounter (parametric study). Details of the various studies and their results are given in Appendix A.

Surprise Study

In this study, subjects drove an instrumented test vehicle for several miles (purportedly for purposes of familiarization as a prelude to a study of driving performance) until they crested a hill and encountered an obstacle on the reverse slope. Measures were made of the time from first sighting of the obstacle until the subject removed his/her foot from the accelerator (perception time), and the time from accelerator release to brake contact (response time). The two values summed gave total time. This was referred to as the "surprise" trial.

With the surprise phase completed, data were taken on the same subjects, under identical conditions, except they knew the obstacle would be there. Each subject made five runs under instructions to lift one's foot from the accelerator and tap the brake pedal as quickly as possible when the subject saw the obstacle. These were referred to as "alerted" trials.

At the conclusion of the alerted trials a red lamp was fastened to the front of the car hood. The subjects were instructed that they were to release the accelerator and tap the brake as rapidly as possible whenever the lamp came on. Five such trials were conducted as the subjects drove the car back to the point where they had started out. These were referred to as "brake" trials.

Two groups of subjects were run in this study. One group was "young," i.e., 40 years or less. There were 49 young subjects. The second group was "older," i.e., 60 years or more. There were 15 older subjects.

The primary data from this study are shown in Figure 3. Figure 3 is a normal probability plot of the total perception-response times from the three studies conducted (i.e., surprise, alerted, and brake), for the young subjects. The perception-reaction data for the older subjects are very similar, except for the brake condition, where the older subjects averaged somewhat slower.

The most relevant data are those for the surprise condition. The 5th to 95th percentile range in Figure 3 is about 0.85 to 1.6 sec. This compares to the 2.5-sec allowance for perception-response time in the stopping sight distance model.

However, the subjects in this study were not representative of the normal driving population in some significant respects. For example, they had been driving for only about 10 to 15 min at the time they encountered the obstacle, they knew they were involved in a study (although they did not know its purpose), and they were not, as far as could be determined, under the influence of anything (e.g., alcohol) that might alter their response times. All of these factors would favor shorter perception-response times. Therefore, a correction to the values measured in this study is appropriate to allow for a driving population that includes persons who are relatively fatigued, less

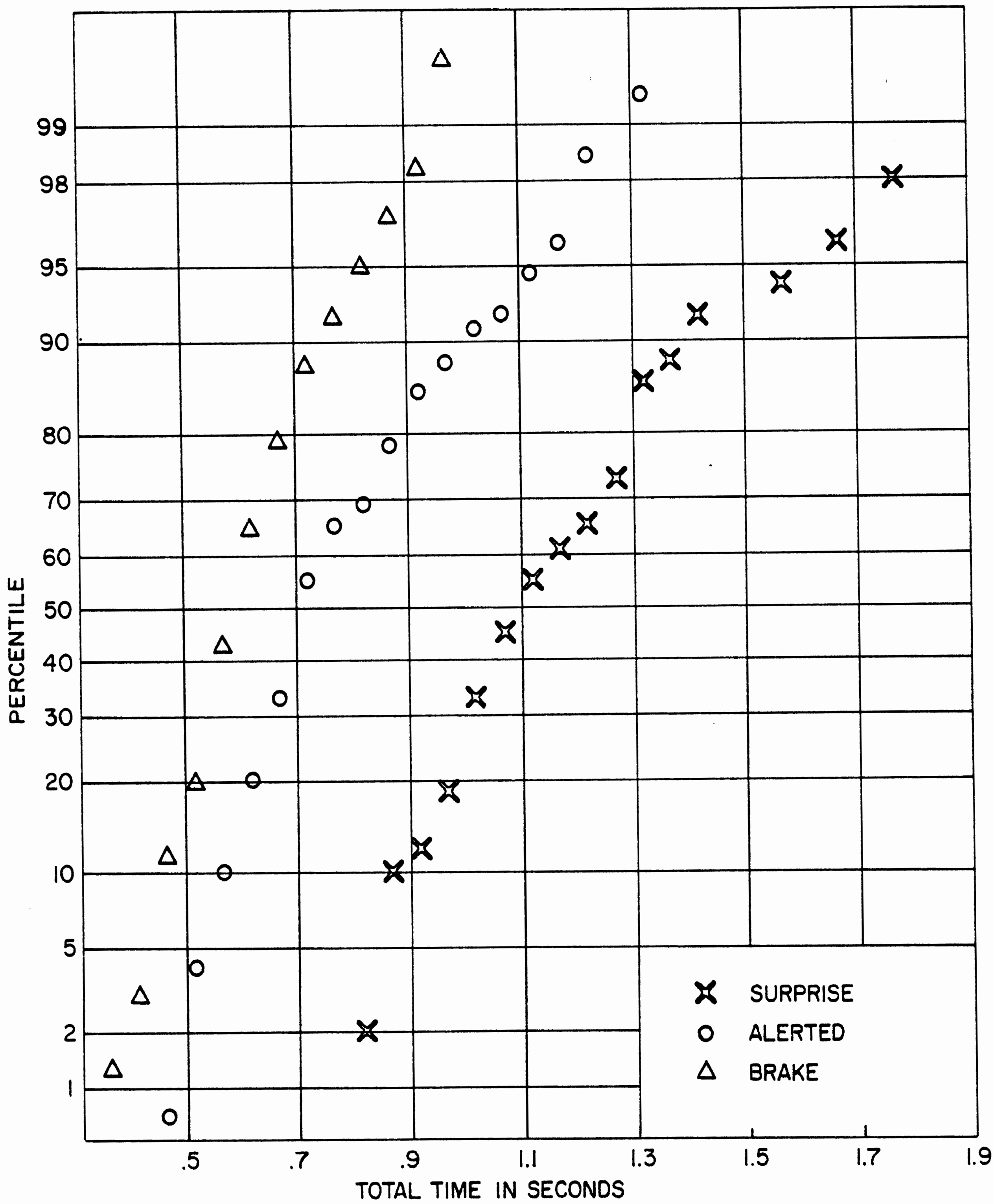


Figure 3. Normal probability distributions of perception - response time under three conditions in surprise study.

attentive, or whose senses have been dulled by drugs of some kind.

A more comprehensive treatment of this problem is contained in Appendix A. In brief, there are few data on the incidence and levels of most of the factors that would affect perception-response time. Thus, while a correction is necessary, the amount that is appropriate is uncertain. However, it can be safely assumed that the minimum values (e.g., 5th percentile) are not likely to change significantly. Reasonably alert people responding quickly are likely to produce the same perception-response times, whether in a study or in the real world. Adding in the effects of variables such as fatigue, alcohol, etc. would result in longer perception-response times, thus increasing the variance of the distribution.

The 5th to 95th percentile range from the surprise study was about 0.8 sec; this range could double, making the 95th percentile 2.4 sec, and still be within the present 2.5-sec standard. Since the available data on alcohol, and the like, suggest that perception-response-time decrements are rarely in excess of 50 percent, a correction increasing 95th percentile perception-response time by 50 percent seems reasonable. This is so close to the present 2.5-sec standard that the latter should be retained.

Parametric Study

The purpose of this study was to gather data on a number of obstacle characteristics to supplement the information from the surprise study.

The method was identical to that used for the alerted portion of the surprise study. A different site was used because the site employed in the surprise study was inconvenient to use for so many trials. A comparison was made of the response times to the same obstacle at the two sites and they were very close. Thus, it is believed the data were unaffected by the differences in test site.

The variables that were studied were obstacle height, width, and contrast. There were three levels of each variable. The intent was to consider the obstacle in the surprise study as a medium condition, and bracket it with the variables in this study. Twenty-six subjects participated. All had been involved in the surprise study.

The results of this study are summarized in Figure 4. This figure is a normal probability distribution of the total perception-response times for each of the seven obstacle conditions studied. The "standard" obstacle was the same as that used in the surprise study.

These data were subjected to statistical analysis. In general, it was found that the high and the narrow obstacles were associated with longer perception-response times than the others, and the white and dark (high contrast) obstacles

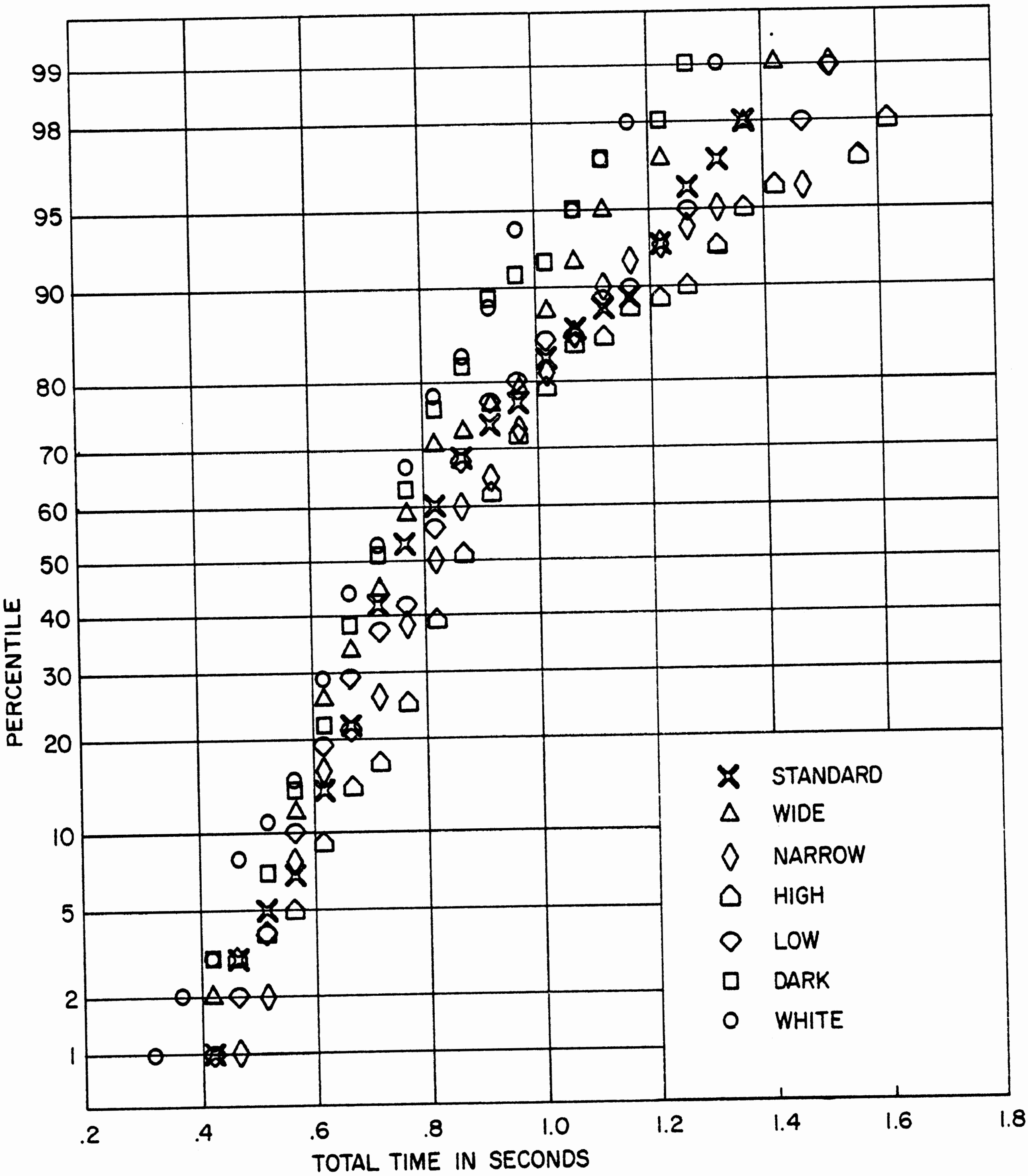


Figure 4. Normal probability distribution of perception - response time to seven different obstacles in parametric study.

were associated with shorter perception-response times than the others.

The findings concerning the high obstacle were unexpected. The authors had assumed that subject response would be determined by a certain amount of the obstacle showing above the hill crest. Here, the high obstacle at one test location first became visible against a background of curb and grass, rather than the road surface, as was the case in all the other trials. This apparently reduced its contrast and made it more difficult to detect, further supporting the importance of contrast in driver's perception and response to obstacles.

NIGHTTIME VISIBILITY

Introduction

The question of nighttime visibility is relevant on crest vertical curves because the usual source of illumination, the vehicle's headlamps, are mounted below the driver's eyes. Thus, an object that would be visible under daytime conditions may be shadowed by the vertical curve for a time, effectively reducing SSD below the design value.

An argument that headlamp mounting height rather than driver eye height is the relevant variable in stopping sight distance must be considered in the context of the visibility distance that vehicle headlamps can provide. To aid in understanding this problem, an analysis was conducted using data on the visibility of small, low-contrast objects under headlamp illumination with high beams. The details of this work are described in Appendix C.

Method

In this study subjects had to detect the target, discriminate it from others being used, and press one of an array of buttons to indicate the type and location (right or left of the car) of the target. Distance was measured from the button-press point until the subject passed the target.

The data from this study were summarized in the form of normal probability distributions, so that it was possible to determine the percent of responses for any desired visibility distance. These were compared with minimum SSD for various speeds at an eye height of 3.75 ft and at an eye height of 2 ft (representing minimum headlamp mounting height).

Results

The results of this analysis are summarized in Table 5. Shown in the second column of Table 5 are the minimum SSD values from the 1971 Blue Book (5), which assumes an eye height of 3.75 ft. In the third column are the modifications that result if eye height is made equivalent to minimum headlamp mounting height (2.0 ft). The last column shows the percent of trials in which the young subjects in the study detected the target at a greater distance than that shown in column 3.

These data suggest that headlamp mounting height presents a potential restriction to nighttime visibility on

Table 5. Comparison of design minimum stopping sight distance, loss due to headlamp mounting height and percent of drivers disadvantages at various speeds.

Speed (mph)	Minimum Design SSD (ft)	Effective SSD With 24 m. Eye Height (ft)	Percent of Young Drivers Disadvantaged
30	200	160	50
40	275	220	20
50	350	280	5

crest vertical curves at speeds up to about 40 mph. However, assumptions made in the analysis concerning the lighting equipment, obstacle placement, and driver characteristics would result in visibility being significantly overestimated. Because of this, there is a problem only at the 30-mph speed, and then only in unlighted areas.

Conclusions

The issue of headlamp mounting height being a controlling factor in nighttime visibility on crest vertical curves appears to be of little consequence. There is justification for increasing the minimum SSD for 30-mph crest vertical curves from 200 to 250 ft in unlighted areas. Otherwise, headlamps are simply not capable of reliably revealing objects at distances beyond the effective sight distance created by their relatively low mounting height.

OBSTACLE HEIGHT

Introduction

Of all the variables in the SSD equation, obstacle height is least amenable to empirical analysis. For this reason, there is considerable variability in obstacle height in the SSD models of various countries. A few examples follow (from Ref. 42):

Australia	0.23 m	(0.75 ft)
Germany	0	0
Finland	0	0

France	0.15 m (0.50 ft)
Great Britain	1.05 m (3.44 ft)
United States	0.15 m (0.50 ft)

The simplest solution is the most conservative one; i.e., that the driver should always be able to see the entire surface of the road within the SSD. This means that all hazards are accounted for, including problems with the road surface itself.

In the United States, a zero-height obstacle in the SSD model has been rejected on the basis of cost considerations. This thinking goes back to 1939, when the AASHO Committee on Planning and Design Policies met to consider the problem (4). At that time both driver eye height and obstacle heights were 4.5 ft. It was agreed that the existing policy was not sound, and consideration was given to reducing obstacle height to zero. However, calculations of cost associated with such a policy convinced the Committee that it was not economically desirable. A 4-in. obstacle was selected as a reasonable compromise, while the 4.5-ft eye height was retained. In 1965 both of these values were changed to 6 in. and 3.75 ft for the obstacle and eye height respectively. The rationale for the 6-in. obstacle height is well summarized in the following quotes from the "1965 Blue Book"(5):

> ...by increasing the height of object from 0 to 6 in., the required length of vertical curve is

> reduced by about 47 percent; by increasing the height from 0 to 12 in., the length is reduced by 57 percent; and by increasing the height from 0 to 18 in., the length is reduced by 63 percent.... Substantial economy in construction... is effected by using a 6-in. obstacle instead of the desirable zero value, yet the ability to see or appraise a hazardous situation on the road is not materially altered. On the other hand, use of a greater object height... results in little additional economy beyond that of a 6-in. controlling height, but materially decreases the ability of the drivers to see conditions on the road ahead.

Analysis

The statement that "...the ability to see or appraise a hazardous situation on the road is not materially altered" is a value judgment that appears inconsistent with describing a zero-height obstacle as "desirable." Be that as it may, there is no question that accepting obstacle heights less than 6 in. will lead to increases in construction costs for crest vertical curves.

With cost accepted as a factor in setting obstacle height, it would be desirable to know the accident costs associated with various visibility distances. A review of studies that have attempted to relate sight distance and accident rates is contained in Appendix E of this report. The results suggest a correlation between accidents and sight distance. However, road curvature and structures near the roadway create problems in addition to sight restrictions. As a result it is not possible at present to obtain a clean relationship between obstacle height and accident costs. That being the case, a variety of other criteria have been considered to develop an argument for obstacle height in addition to the economic one.

One of the best summaries of various considerations in obstacle height has been provided by Ketvirtis and Cooper (16), who set out to answer the question: "what is the minimum size object the driver has to detect at a safe stopping sight distance?" Consideration was given to perceived risk, consequences of impact, and vehicle clearances.

The results of the study are interesting. It was found that given the options:

FULL STOP

GO AROUND

PASS OVER

most drivers elected "go around" when confronted with obstacles 10 cm (about 4 in.) or higher. They rarely elected "full stop" for any obstacle. Thus, objects 10 to 12 cm or higher are described as having a "psychological impact." The authors believe that objects of about 25-cm (10 in.) height constitute a physical hazard.

Some of the assumptions and interpretations made by Ketvirtis and Cooper are questionable. For example, referencing NCHRP Report 150 (17) they state:

> ...a regular North American car will not be deflected by a 15 cm high object at a speed above 70 km/h. A 15 cm object may cause a disturbance in the steering, but rarely will it be the cause of damage or injury.

NCHRP Report 150 deals with the problem of impacts at shallow angles with curbs of various heights and contours. In that context, the quote is a reasonable interpretation of the findings. Leaving out the qualification concerning impact angles has the unfortunate effect of implying that the statement applies to all conditions, which is not true. Impacting an object equivalent to a 6-in. curb turned at approximately 90 deg to the path of travel at a speed in excess of 70 km/h could severely damage a car. It is quite possible that the tires would be blown and the wheels, suspension, and steering system damaged. Control by the driver after such an impact is questionable. The car could be undriveable. The vehicle's occupants may be thrown about by such an impact so as to risk injury, especially if they are unbelted.

Clearly, impact with a 6-in. high, solid obstacle at anything but very low speed is something to be avoided. As noted earlier, Ketvirtis and Cooper found that their subjects almost always elected to "go around" if the obstacle were 4 in. or more high. However, what happens when the driver cannot drive around the obstacle? The choice then is to pass over or stop. If the object is narrow enough, the driver may be tempted to drive over it. Ketvirtis and Cooper state that the clearance of a typical North American car is about 18 cm (about 7 in.), so that a 6-in. obstacle would pass underneath. This is probably true for the classic large American car. However, vehicle manufacturers base clearances on criteria other than a fixed obstacle; and as wheelbases have shortened, clearances have been reduced.

A good, recent summary of ground clearance data on minicompact and subcompact cars has been provided by Woods (18). These are vehicles most likely to represent the "cars of the future." Woods' data indicate that about 30 percent of such vehicles would not clear a 6-in. obstacle. A 4-in. obstacle is required to provide adequate clearance to all of the small cars surveyed.

Recommendations

What conclusions can be drawn from the foregoing discussion? Clearly, nothing has changed concerning the economic arguments. Lowering obstacle height below 6 in. will still lead to increases in construction costs for crest vertical curves. This point has been considered in the sensitivity analyses. An example is shown in Table 21 later in this chapter. For the specific case considered in Table 21, reducing obstacle height from 6 to 4 in. increased the length of crest vertical curves by about 10 percent.

Clearly, reducing obstacle height from the current 6 in. to 4 in. (to provide for under-vehicle clearance) will result in significant construction cost increases. Unfortunately, it is not possible to quantify the gains that would result from such a change, or even certify that there would be any gains.

The major change that has occurred since the 6-in. obstacle was adopted has been in the makeup of the vehicle fleet. It is still changing, and will in the future consist largely of cars much smaller than those typical of 10 to 20 years ago. This fact should have an effect on the accident costs associated with a 6-in. standard. Smaller cars are likely to suffer more damage from an impact with a 6-in. obstacle, even at low speeds, and are perhaps more likely to be deflected from their path. They are also more likely to be snagged in passing over such an obstacle.

In the opinion of the authors it is time to consider a reduction in obstacle height in deference to the change in the vehicle population. A 4-in. obstacle may be a reasonable choice.

VEHICLE BRAKING DISTANCES

Introduction

In addition to the process by which the driver sees, decides, and reacts to the presence of an obstacle in his path, the human operator is also a critical part of the process by which the vehicle-tire-pavement system creates forces to decelerate to a stop. Although the braking process has traditionally been viewed within the highway community as a process in which the driver applies the brake sufficiently to lock all wheels with the resulting deceleration being fully dependent on the frictional coupling existing between tires and pavement, this view is rather removed from reality. Whereas there are instances in which drivers will sacrifice the directional stability and control quality of their vehicles, in their efforts to stop as quickly as possible, they will, in general, modulate their braking effort so as to minimize both stopping distance and the possibility of losing directional control and stability. In particular, it is highly unlikely that drivers, when traveling at high speed on a wet pavement, will be inclined to brake sufficiently so as to lock the wheels on their vehicles and hold them at lockup. Rather, they will very likely modulate their braking input so as to retain an ability to steer the vehicle and to prevent a spinout (or a jackknife, if the vehicle should be an articulated tractor-semitrailer).

As discussed above, the braking process is much more complicated than is implied by the "standard design formula." Nevertheless, if the friction coefficient between tire and roadway, f, is interpreted as the average deceleration (in units of gravitational acceleration) achieved during a stop, the standard formula is meaningful and useful. The question is one of determining how deceleration depends on:

1. The ability of the driver to modulate his braking control so as to exploit fully the deceleration capability of a given vehicle-tire-pavement system without losing directional stability and control.

2. The deceleration capability of the vehicle-tire-pavement system itself.

In order to facilitate this discussion, it is useful to adopt the concept of " braking efficiency." In essence, "braking efficiency" is a convenient numeric that expresses (in percentage terms) the extent to which a vehicle when controlled by a perfect controller, is able to utilize the prevailing tire-pavement friction couple for stopping without locking up wheels so as to degrade steering or directional stability. If the same measure is applied to a driver-vehicle combination in which the driver is a less than perfect controller, one can view overall braking performance as consisting of the influence of the control efficiency, CE, of the driver acting on the braking efficiency, BE, of a vehicle-brake-tire system under perfect control.

This examination of the influences of pavement, tire, vehicle, and driver properties on vehicle braking distances has led to the development of sets of equations that can be used sequentially to predict the braking distance capabilities of cars and trucks operating on poor, wet roads. These equations are structured to allow highway engineers to examine the influences of skid number and skid-number-speed-gradient on the braking distances achievable by drivers operating current models of cars and heavy trucks. (Empirical evidence supporting these equations are presented and discussed in Appendix B.) The purpose of this section is to summarize the research findings so as to show how highway engineers can put them into practice.

Principal Mathematical Formulas

The sequence of considerations involved in predicting braking distance consists of establishing the following items: (1) roadway characteristics, (2) tire properties, (3) vehicle properties, and (4) driver control factors. For convenience in summarizing the findings with regard to the factors influencing braking distances, the principal mathematical formulas that have been developed are listed in Table 7, using the symbols defined in Table 6. Note in Table 7 that separate equations are used for describing cars and heavy trucks. The formulas given require sequential evaluation because: (1) tire performance depends on the characteristics of the road surface, (2) vehicle braking efficiency is a function of the level of tire-road friction available, and (3) driver control efficiency represents the ability of drivers to utilize the capability of the roadway-tire-vehicle system. The flow diagram shown in Figure 5 summarizes the procedure for using Eqs. 1 through 14 to calculate braking distances.

Table 6. List of Symbols.

A	frontal area
BE	vehicle braking efficiency
C_D	aerodynamic drag coefficient
CE	braking control efficiency
faero	deceleration due to aerodynamic drag
f_i	deceleration capability of road-tire-vehicle system if the driver could modulate the brakes perfectly
f_p	peak (maximum) rolling friction coefficient (tire to road)
f_s	sliding friction coefficient (tire to road)
$\Delta f_{s,p}$	difference between friction levels for new and bald (completely worn) tires
f_x	tire-to-road friction with x/32 in. of tread groove depth
h	center of gravity height
L	wheelbase
MD	mean pavement texture depth (in.) measured by the sandpatch method
n	tread groove height in 32nds of an in. (for new tires, n = 12 is used herein) (The influence of tire wear is deemed negligible for truck tires with groove depths greater than 12/32 in.)
P	Normalized skid number gradient (see Eq. 3)
SN_{40}	ASTM skid number at 40 mph
SN_V	ASTM skid number at V mph
V	velocity in miles per hour
V_o	initial velocity in miles per hour
W	vehicle weight
W_r	weight carried by rear suspension

x tread groove height in 32nds of an in.

Y_r proportion of braking effort due to the rear wheels

Table 7. Equations developed for use in estimating braking distances.

Roadway Characteristics

Skid Number:

$$SN_V = SN_{40}e^{P(V-40)} \qquad (1)$$

where e is the base for natural logarithms, and

$$P = -0.0016(MD)^{-0.47} \qquad (2)$$

Note: Due to the properties of the exponential function:

$$P = (\partial SN_V/\partial V)/SN_V \qquad (3)$$

Tire Properties

Sliding Friction:

$$f_s = 1.2\ SN_V, \text{ for car tires} \qquad (4)$$

$$f_s = 0.84\ SN_V, \text{ for truck tires} \qquad (5)$$

Maximum Rolling Friction:

$$f_p = 0.2 + 1.12\ f_s, \text{ for car tires} \qquad (6)$$

$$f_p = 1.45\ f_s, \text{ for truck tires} \qquad (7)$$

Tire Wear:

$$\Delta f_{s,p}/f_{s,p} = -5.08\ (MD) + 0.008045V \qquad (8)$$

Note: Equation 8 is applied to either f_s or f_p as required.

$$f_{xs,p} = f_{s,p} - \Delta\ f_{s,p}\ (1-(x/n)^{1/2}) \qquad (9)$$

Note: Equation 9 is used to interpolate between new tires or tires with groove depths greater than 12/32 in. (x = n) and bald tires (x = o).

Vehicle Properties

Braking Efficiencies:

$$BE = 0.91$$

for 1982 cars with 1 or 2
persons inside (10)

$$BE = (W_r/W)/(Y_r+f_x h/L) \qquad (11)$$

for heavy straight trucks
in the unladen condition

Aerodynamic Drag:

$$faero = 0.00238AC_D V^2/W \qquad (12)$$

Driver Characteristics

For Passenger Cars:

$$CE = 0.444 f_i + 0.267 \qquad (13)$$

where $f_i = f_x$ BE and f_x is evaluated at $V = V_o$.

For Heavy Trucks:

$$CE = 0.62 \qquad (14)$$

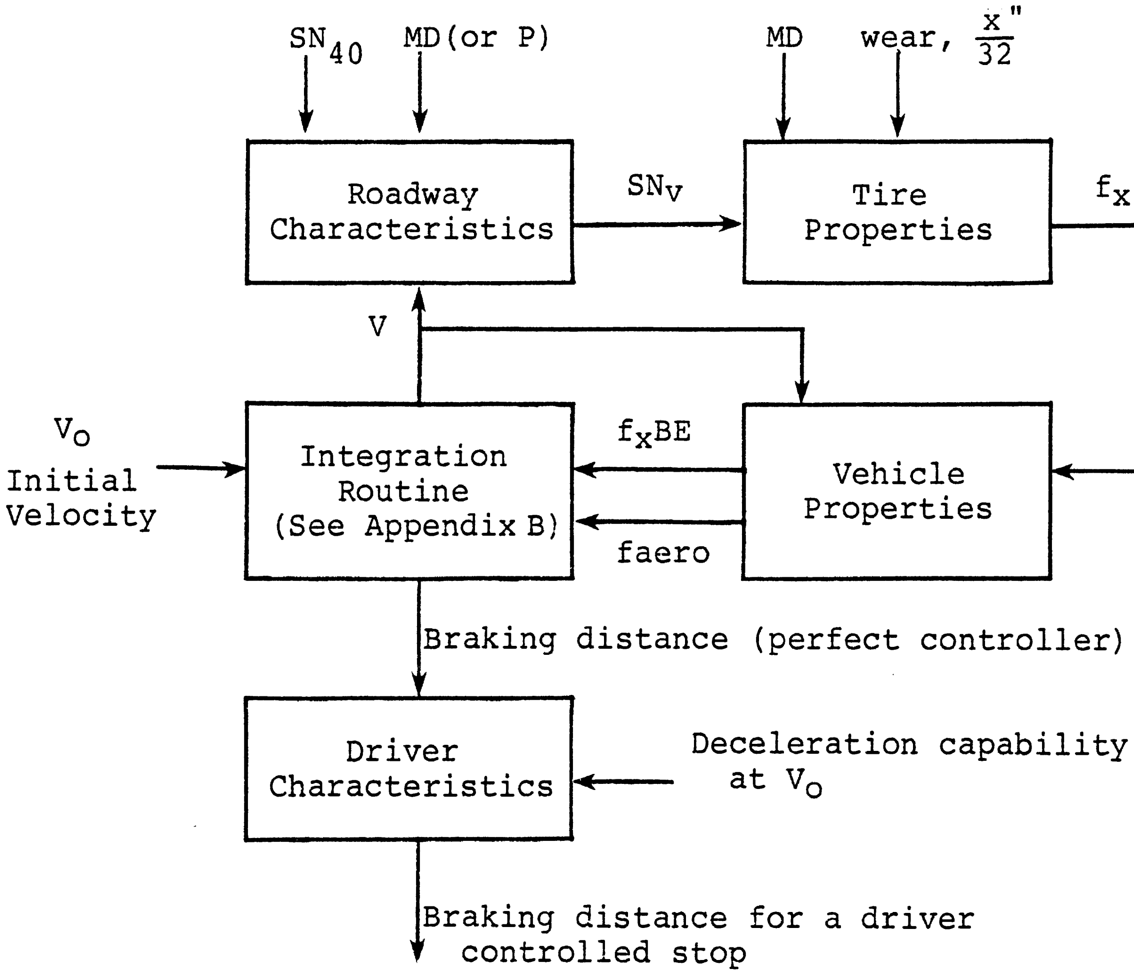


Figure 5. Diagram illustrating the order of calculation.

Comparisons of Braking Distance Results

The equations presented in Table 7 have been used to estimate braking distances. These estimates differ greatly from those given in the AASHTO Policy draft report (1), particularly at highway speeds, because the factors considered here include the abilities of drivers to keep their vehicles under control while making a rapid stop. The distances given in the AASHTO report (1) are based on locked-wheel tire characteristics. In contrast, a basic premise of the approach taken here is that at high speeds drivers will avoid locking wheels on slippery roads so that their vehicles will not: (1) spin around, (2) leave their lane of travel, and/or (3) be unresponsive to steering efforts.

Passenger Car Results. A 15th percentile road with respect to its skid number has been selected for use in illustrating the expected braking performance of passenger cars. Based on an examination of the work of Taoka (19), a 15th percentile road is defined by the following equation:

$$SN_V = 28e^{-0.0115(V-40)} \tag{15}$$

Equation 15 describes a wet road with an ASTM skid number, SN_{40}, of 28 and a mean texture depth of 0.015 in., corresponding to a skid number gradient of 0.32 skid numbers per mile per hour at 40 mph (see Eqs. 1,2, and 3).

For the 15th percentile road, representative values of tire, vehicle, and driver characteristics have been used, in conjunction with the equations defined in Table 7, to construct curves corresponding to the braking performance of driver-vehicle systems that are either (1) traveling with locked wheels, but out of control, or (2) operating with steering control while avoiding wheel lockup. The braking distances computed for cars with new tires or tires worn to 2/32 in. (the legal limit) are shown in Figure 6. In addition, Figure 6 contains a curve based on the braking distances given in the AASHTO report (1), and two curves corresponding to hypothetical situations in which a perfect controller (CE = 1.0) is available to extract maximum performance from the roadway-tire-vehicle system. To obtain the curves representing the findings of this study, the equations given in Table 7 were used to evaluate vehicle deceleration as a function of velocity. Then this functional relationship was used in connection with a numerical integration algorithm to determine braking distance from a specified initial velocity.

Examination of the results summarized in Figure 6 clearly indicates the wide spread of braking distances attainable depending on the assumptions made concerning the mode of vehicle operation ("control" versus "lockup") and the amount of tire wear ("new" versus "2/32"). According to these results the braking distances recommended in the AASHTO Policy draft report (1) roughly correspond to

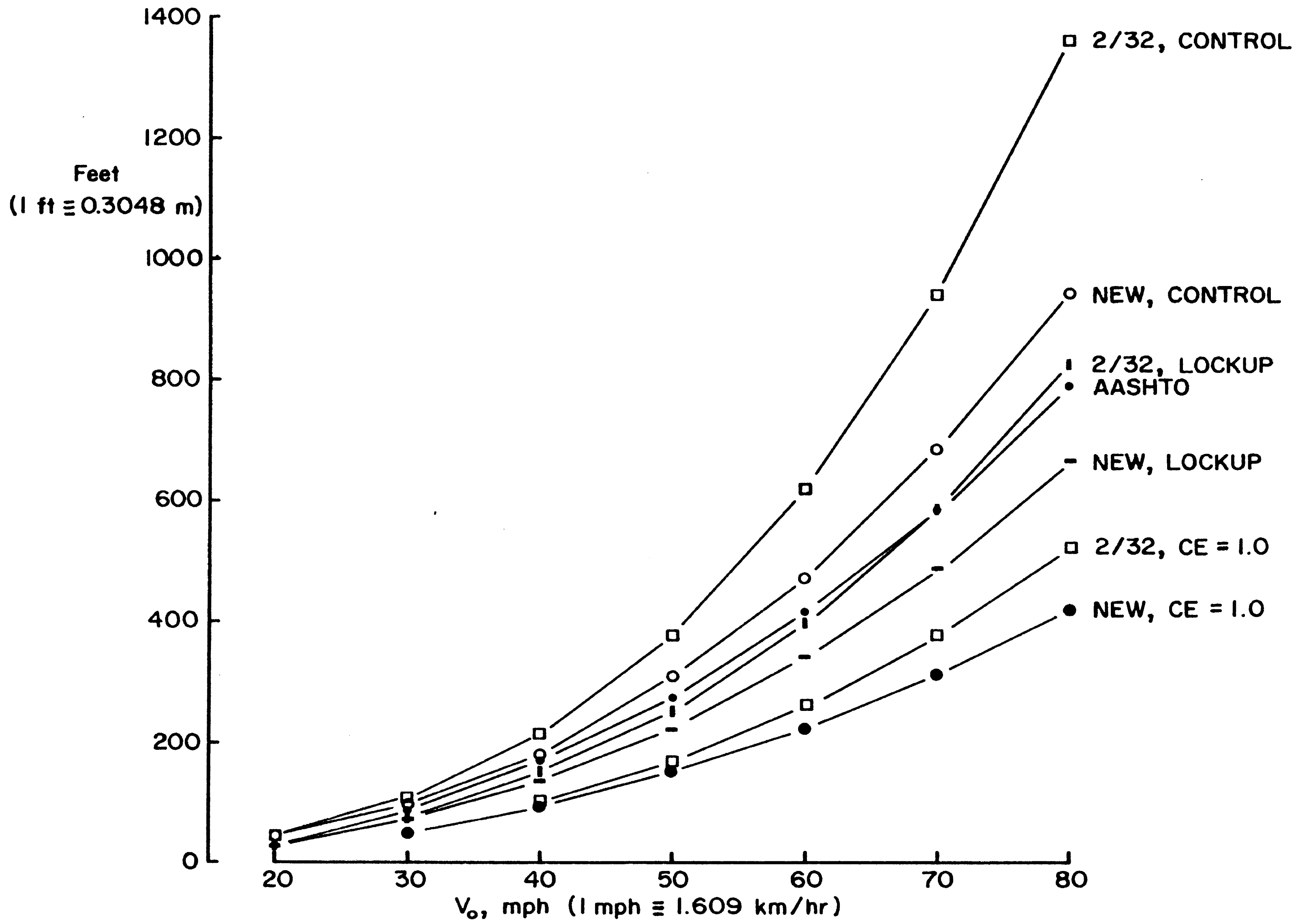


Figure 6. The braking performance of passenger cars on a poor wet road.

situations in which a driver, who is operating a vehicle with poor tires (2/32 in. groove depth) on a slippery road (15th percentile), simply locks the vehicle's wheels, thereby traveling more or less in a straight line without control of the direction of travel or the heading of the vehicle. Such a vehicle would not be able to negotiate a horizontal curve or stop within its own lane of travel.

The two hypothetical curves, labeled "CE = 1.0," reflect the improvement in friction capability of a rolling tire as compared to that of a sliding tire. However, experiments indicate that drivers are able to utilize only a fraction of this improved friction capability when braking while maintaining directional control (see the curves labeled "2/32, control" and "new, control" in Fig. 6). Even with good tires, the braking distances required for a stop with the vehicle under control are considerably longer than the AASHTO recommendations for stops from initial velocities of 50, 60, 70, or 80 mph. As indicated in Figure 6, very long distances are required if someone with poor tires attempts to make a safe, controlled stop from high speed on a very poor, wet road.

<u>Heavy Truck Results</u>. Unloaded heavy trucks have much lower braking efficiencies than those achieved by current passenger-car braking systems. In addition, experimental data show that the frictional capabilities of truck tires are approximately 0.7 times the frictional capabilities of

passenger-car tires. Consequently, heavy trucks require greater braking distances than passenger cars.

However, on a crest vertical curve, the truck driver has an SSD advantage over the car driver because of the difference in eye heights between car and truck drivers. The basic parabolic relationship employed in the design of vertical curves (5) can be used to determine the ratio of truck-driver to car-driver sight distances for vertical curves such that the length of the curve, L, is greater than the sight distance, S, viz., for $L > S_t > S_c$

$$S_t/S_c = (h_{et}^{1/2} + h_o^{1/2})/(h_{ec}^{1/2} + h_o^{1/2}) \tag{16}$$

where:

S_t = the truck driver's sight distance

S_c = the car driver's sight distance

h_{et} = the truck driver's eye height

h_{ec} = the car driver's eye height

h_o = the object height

Using typical values of (1) 93 in. for the eye height of drivers of conventional trucks, (2) 42 in. for the eye height of car drivers, and (3) 6 in. for the height of the object on the road, Eq. 16 indicates that required truck SSD, including both reaction and braking distances, should be less than 1.35 times the SSD for cars if trucks are to be able to stop within their sight distances on vertical curves designed for cars. Note that this result applies for all vertical curves for which $L > S_t$ regardless of the values of

A, the percent difference in grades, and L, the length of the vertical curve.

The parametric sensitivity of the ratio of truck-sight distance to car sight distance is illustrated in Table 8. Typically, as shown in Table 8, the driver of a cab-over-engine (COE) truck can see 6 percent further than the driver of a conventional truck.

With regard to SSD concerns on crest vertical curves, the greater sight distances of truck drivers may compensate for the reduced braking capabilities of heavy trucks as compared to cars (1). To check whether roads designed for cars are adequate for trucks, the stopping distances for cars and trucks have been computed using reaction distances based on a perception-response time of 2.5 sec and values of braking distance based on the equations given in Table 7. The results in Table 9 indicate that (1) for locked wheel stops, roads designed for cars are adequate for trucks, and (2) for controlled stops, roads designed for cars are almost, but not quite, adequate for trucks. On particular vertical curves with large amounts of truck traffic, truck performance could be compensated for by increasing sight distances by approximately 10 percent over that determined for cars.

Influence of Grades

Corrections in braking distance to account for the effects of grades are listed in Ref. (1). These corrections are based on the following equation:

Table 8. The influence of parametric variations on S_t/S_c.

Type of Variation	h_{et} (in.)	h_{ec} (in.)	h_o (in.)	S_t/S_c
1. Base line truck	93	42	6	1.35
2. Cab over engine truck or tractor	107	42	6	1.43
3. Low car	93	38	6	1.40
4. Low object	93	42	4	1.37
5. Recommended values	93	40	4	1.40

Table 9. Car versus truck stopping distances.

V_o	Reaction Distances	Braking Distances		Stopping Distance Ratio Locked	Braking Distances		Stopping Distance Ratio Control
		Car Lock 2/32	Truck lock 2/32		Car Control 2/32	Truck Control 2/32	
20	73	33	47	1.13	--	--	--
30	110	77	109	1.17	--	--	--
40	147	149	208	1.20	216	379	1.45
50	183	254	350	1.22	380	645	1.47
60	220	399	538	1.22	619	990	1.44
70	257	588	772	1.22	943	1,410	1.39
80	293	823	1,047	1.20	1,365	1,892	1.32

Locked wheels assumed for speeds less than 20 mph.

$$\Delta x = D_L\ [-g/(f + g)] \tag{17}$$

where:

Δx = the correction in braking distance;

D_L = the braking distance on a level road with an average deceleration equal to f;

g = the grade in percent/100 with $g > o$ meaning an upgrade; and

f = the average deceleration on a level road applicable to the entire stop.

Since the magnitude of the deceleration is varying throughout a stop, the use of average or equivalent friction in Eq. 17 represents an approximation. However, results obtained by using Eq. 17 are satisfactory for design.

An alternative, if more rigor is desired, would be to include the level of grade in determining the deceleration used in the numerical integration employed to calculate braking distance. Note that if this is not done, the average deceleration, f, needed in Eq. 17, may be computed from the braking distance obtained for a level road. Table 10 provides example results for downgrades of 3, 6, and 9 percent.

REQUIRED STOPPING SIGHT DISTANCE

Introduction

This section brings together the information on driver reaction and braking described in the previous sections. It reports on considerations of vehicle speed and vehicle length in determining the stopping distance. Sensitivity

Table 10. Corrections for downgrades for passenger cars making controlled stops with new tires.

Speed (mph)	Braking Distance g=0%	f	Correction in braking distance (ft)		
			-3%	-6%	-9%
30	97	.310	10	23	40
40	184	.290	21	48	83
50	308	.271	38	88	153
60	473	.254	63	146	--
70	684	.239	98	229	--
80	944	.226	144	--	--

These distances correspond to the curve labeled "new, control" in Figure 6. These distances are used to calculate the average friction factors, f, given in this table.

analyses for the various combinations of elements in the required SSD relationship are reported.

The basic relationship between required stopping sight distance, SSD, perception-response distance, PRD, speed, V, and braking distance, BD, is:

$$SSD = PRD + BD \tag{18}$$

where:

$$PRD = PRT \times V \tag{19}$$

$$BD = V^2/30(f + g) \tag{20}$$

Independence of Terms

A recent study of the decelerations used by motorists in an approach to a traffic signal revealed no indication of an interaction between initial speed and deceleration rate (20). A search of the literature had earlier revealed no evidence of such an interaction. Accordingly, no interaction is used in the model, Eq. 18 above.

Speed and Vehicle Elements

The required SSD relationship multiplies the perception-response time, PRT, by the speed, V, to obtain the PRD component of the SSD. The braking distance component takes speed into account with a square effect. In this section a study of the differences between wet and dry pavement speeds is reported so that the need to adjust speed for wet pavements can be assessed. The effect of the length of the vehicle is also explored.

Speed Study. An analysis of speed distributions from rural roads with 55-mph speed limits was carried out. The data obtained from State Departments of Transportation had been collected for the National Speed Limit Monitoring Program for rural interstate, rural principal and minor arterials, and rural major collectors. These data were used to investigate the underlying distribution of speeds on such roads and were also used to compare the distribution of speeds on wet and dry pavements. Details of this study are given in Appendix E.

Statistical tests for the underlying distribution showed that a 10 percent sample of the 900 available hourly distributions were normal at the $\alpha = 0.05$ level. Visual comparisons were made with the remaining distributions and it was concluded that they were quite similar. The results imply that the often made assumption that speeds on rural highway facilities are normally distributed is a reasonable one.

The comparison of speed distributions on wet and dry pavements was carried out by comparing the daylight speed distributions recorded at a set of 26 permanent speed monitoring stations from Illinois for which reliable weather information was also available. Speed data were analyzed for days that were known to have had a high probability of rain for the whole day and nearby days where there was no rain. Analysis of variance of the hourly speed distributions indicated that there was no time of day effect in the speeds of these sites during daylight hours. Accordingly, the speeds at a site for one day were aggregated. The daily cumulative speed distributions at a site were compared and contrasted by rainy and dry days.

Table 11 presents a summary of the speed distributions from the 26 sites in Illinois. The summary statistics in this table are for the entire sample of speeds from wet days and dry days at a site. The detailed daily speed summaries are given in Appendix E.

Figures 7 and 8 show the cumulative distributions of speeds for several wet days and several dry days collected at two of the speed monitoring sites in Illinois. These particular sites are on rural arterial roads; however, the differences in speed distributions between wet and dry days shown on these figures are typical of those seen at the other sites and show no difference by the wet and dry classification. The results indicate that the differences among the daily speed distributions on rainy days are of the same magnitude as the differences between the speed distribution on dry and rainy days. This implies that operators drive on wet pavements at the same speeds as they do on the same roads on dry pavements. Details of these studies are presented in Appendix E.

Vehicle Length. The SSD model as applied to geometric design specifies the distance from the eye of the driver to the obstacle as the SSD. The vehicle is hence assumed to be brought to a stop with its front bumper at the object. The eye of the driver is of course located some distance back from the bumper, ranging up to 10 ft. To account for this

Table 11. Summary of speed distributions at 26 Illinois sites by wet and dry conditions.

Site	Condition	Number of Days	Sample	Mean (mph)	St. Dev. (mph)	Percentile Speed (mph)		
						15	50	85
1	Dry	2	7212	61.4	5.9	56.8	60.6	65.3
	Wet	2	5947	62.2	7.2	57.1	61.2	67.4
2	Dry	2	6014	60.5	5.5	56.0	61.0	64.5
	Wet	2	7510	61.3	5.2	57.8	60.2	64.3
3	Dry	2	14419	61.5	5.2	56.9	61.0	65.8
	Wet	2	11083	62.1	5.5	57.9	61.6	66.0
4	Dry	2	6374	60.6	7.0	55.5	60.4	65.3
	Wet	2	10920	60.9	4.7	47.0	53.4	60.0
5	Dry	2	9559	62.5	4.8	58.3	62.2	66.3
	Wet	2	8202	61.6	5.7	56.8	60.2	65.1
6	Dry	2	5551	59.4	7.3	55.4	59.6	64.8
	Wet	2	9628	61.0	6.3	56.6	60.8	65.2
7	Dry	2	10880	59.5	7.9	53.0	61.0	65.8
	Wet	2	10414	59.2	5.5	54.6	59.2	63.4
8	Dry	2	511	51.0	14.0	35.0	52.2	63.5
	Wet	1	266	52.2	13.9	35.0	54.8	64.1
9	Dry	3	2457	65.5	8.4	56.7	63.2	70.7
	Wet	3	1433	64.1	7.9	55.7	63.1	68.6
10	Dry	3	1883	61.4	7.9	52.9	59.8	65.7
	Wet	3	1063	61.7	7.8	54.5	60.4	65.2
11	Dry	2	467	59.7	12.5	42.3	59.2	69.2
	Wet	1	305	61.2	14.6	35.0	62.0	72.9
12	Dry	3	1589	62.0	7.0	54.0	60.3	65.3
	Wet	3	1138	63.2	7.4	54.9	62.1	67.0
13	Dry	3	2642	61.7	7.7	54.2	60.2	65.1
	Wet	3	2259	61.2	7.0	54.6	59.9	63.9
14	Dry	3	614	60.3	9.6	50.2	58.7	67.1
	Wet	2	363	62.1	8.4	53.0	60.8	65.3

Table 11. Continued.

Site	Condition	Number of Days	Sample	Mean (mph)	St. Dev. (mph)	Percentile Speed (mph)		
						15	50	85
15	Dry	3	1364	54.1	14.8	35.0	58.4	65.2
	Wet	3	1136	54.4	14.9	35.0	58.7	65.6
16	Dry	3	8121	62.7	6.9	56.5	62.4	65.7
	Wet	3	6462	63.2	6.1	57.0	61.4	65.0
17	Dry	2	1572	60.3	9.1	51.8	59.4	65.2
	Wet	2	727	61.0	9.1	50.7	57.9	64.2
18	Dry	1	392	61.3	7.1	55.9	62.0	64.5
	Wet	3	893	61.8	7.3	53.1	59.4	65.4
19	Dry	1	615	61.7	8.2	54.7	60.2	65.8
	Wet	3	765	60.7	8.9	52.3	60.6	66.4
20	Dry	3	3323	64.1	8.2	54.8	62.9	68.7
	Wet	3	2868	64.1	7.9	54.9	63.2	68.7
21	Dry	3	4791	60.0	6.9	51.6	57.9	63.3
	Wet	3	3909	60.0	6.5	52.1	58.2	63.1
22	Dry	3	3007	66.0	8.3	57.0	63.3	71.1
	Wet	3	2595	65.6	8.0	56.8	63.4	70.8
23	Dry	1	32	45.3	11.9	35.0	40.2	53.5
	Wet	2	145	50.9	12.3	35.0	48.7	61.4
24	Dry	3	3864	57.1	7.4	48.4	55.3	61.0
	Wet	3	2935	58.4	7.9	48.1	56.7	61.7
25	Dry	3	601	57.6	11.5	44.9	57.6	64.0
	Wet	2	195	58.5	10.3	48.1	58.7	63.9
26	Dry	2	624	62.0	9.6	50.7	60.5	67.5
	Wet	3	1099	61.6	8.7	51.1	59.3	66.8

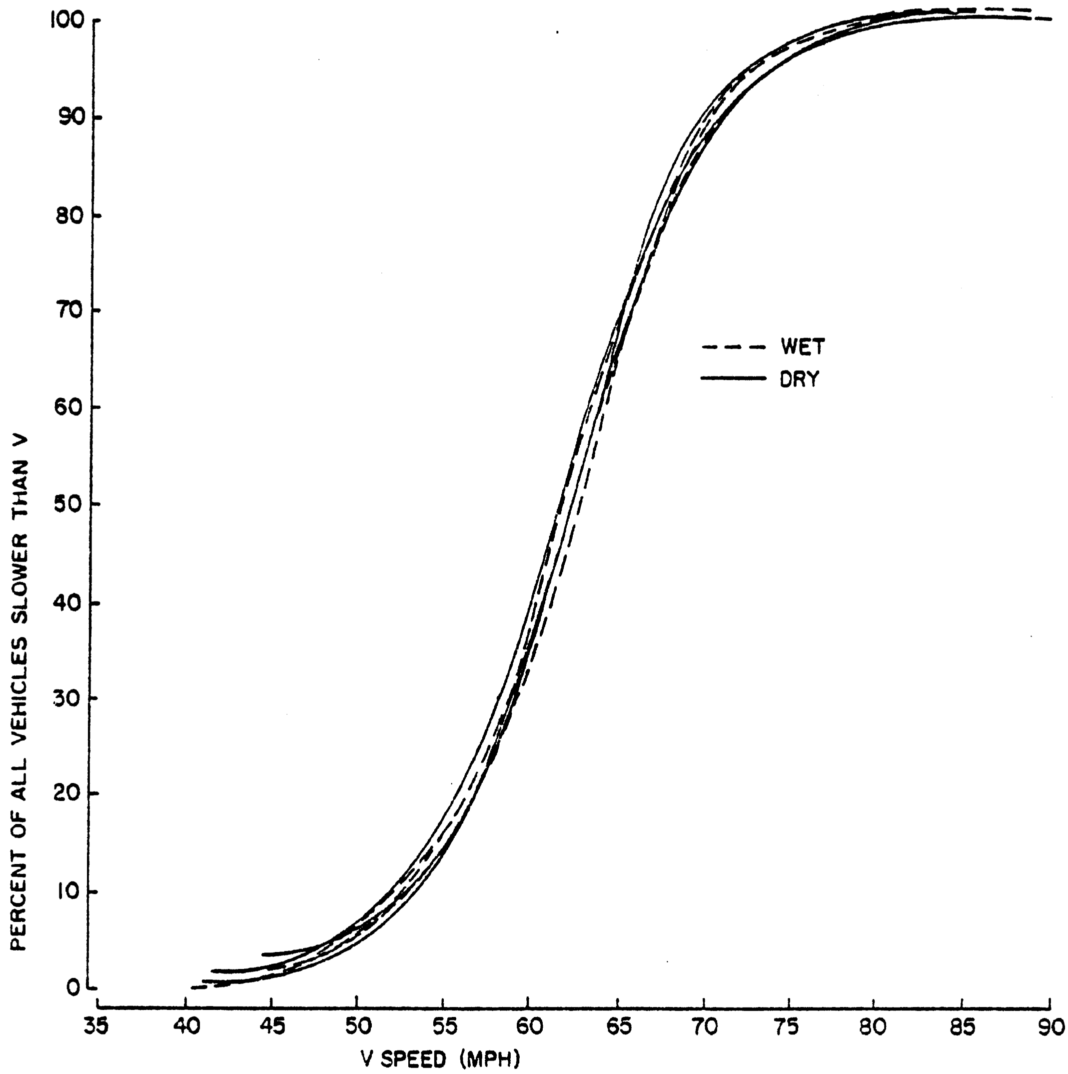


Figure 7. Cumulative distribution of speeds at three wet and three dry days at a rural arterial site in Illinois.

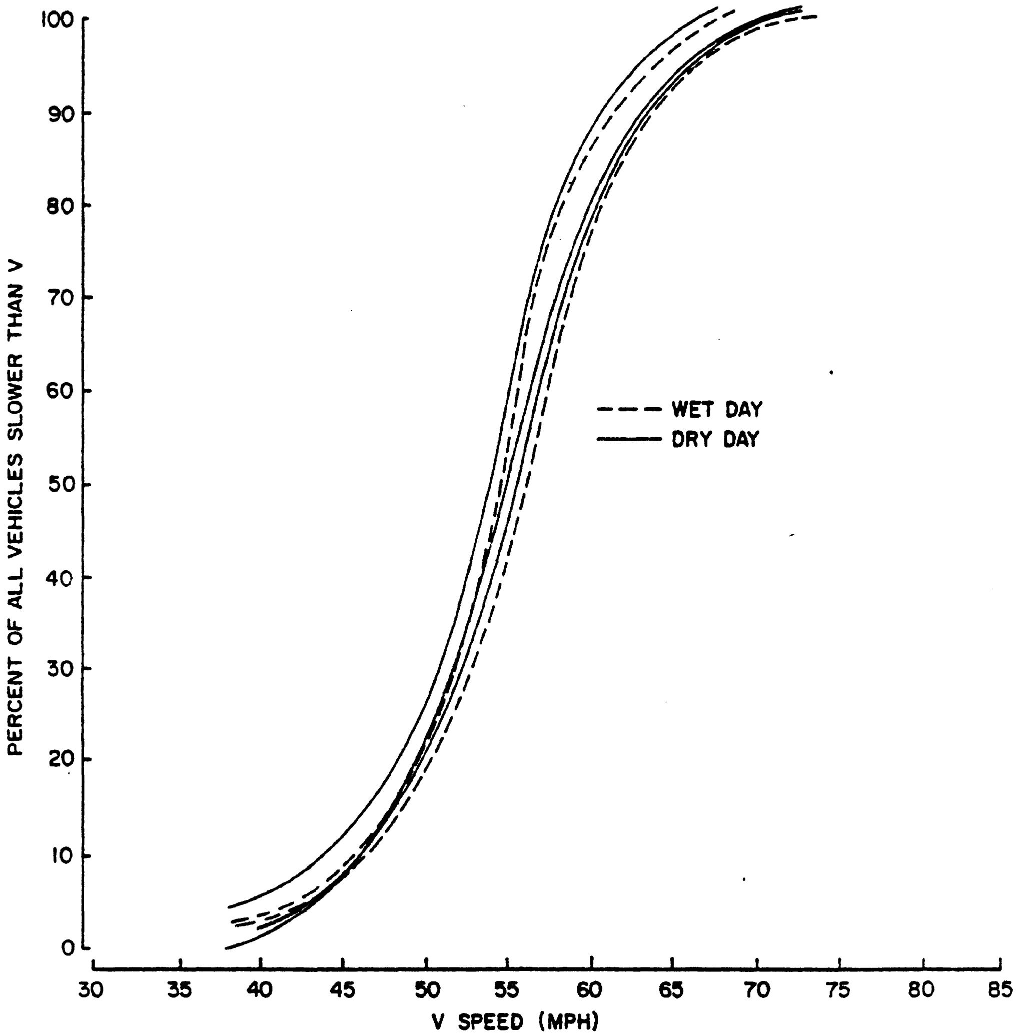


Figure 8. Cumulative distribution of speeds at two wet and three dry days at a rural arterial site in Illinois.

in the general case would involve designing for a greater SSD. However, only when the SSD is 300 ft or less would this effect be noticeable (3 percent or more), and this would be for initial speeds of 30 mph or less. This effect has not been included in the tables presented later in this report.

Sensitivity Analysis

A sensitivity analysis involving the systematic variation of each of the elements in the required SSD relationship, Eq. 18, was conducted. No gradient effects are explored in this analysis. The elements and the values selected for inputs are as follows:

Initial Vehicle Speed = 20,30,40,50,60,70, and 80 mph

Driver Perception-Response Time = 2.0, 2.5, and 3.0 sec

Average Deceleration (g's) for the following conditions:

- AASHTO Lower Policy
- Locked Brakes - New tread
- Locked Brakes - 2/32 tread
- AASHTO Upper Policy
- Controlled Stop - New tread
- Controlled Stop - 2/32 tread

For reference, the current AASHTO policy values for required SSD are presented in Table 12. The results obtained by employing the selected values in calculations of

SSD are given in Tables 13 through 19, one table for each initial speed.

The variation in the resulting SSD values for a given speed can be seen to respond more to driver response time at speeds of 40 mph or less where this component is a larger part of SSD than is the braking distance component. At speeds of 60 mph and higher, the sensitivity to such changes is only 5 to 10 percent of the total SSD. The effect of average braking deceleration rate is seen to be minimal at low speeds but of great importance at the higher design speeds.

AASHTO uses SSD from speeds below 30 mph in a number of applications (turning roadways, one-way streets, one-lane roads). Table 20 presents recommended SSD for speeds from 10 to 25 mph developed in this research. Current AASHTO recommendations are also presented in the table. The driver reaction distance can be seen to account for 60 to 80 percent of this distance. Recommendations for the full range of speeds are given in Table 27.

HIGHWAY DESIGN AND STOPPING SIGHT DISTANCE

Introduction

This section builds on the values of required SSD developed as a result of the driver, braking and speed studies. It is concerned with the impact of changed values of required SSD on the most common geometric design elements, horizontal and vertical curvature. Their impacts on excavation and embankment requirements are explored for

Table 12. AASHTO stopping sight distances (wet pavements).

Design Speed (mph)	Assumed Speed for Condition (mph)	Brake Reaction		Coefficient of Friction f	Braking Distance on Level (ft)	Stopping Sight Distance	
		Time (sec)	Distance (ft)			Computed (ft)	Rounded for Design (ft)
20	20-20	2.5	73-73	0.40	33-33	106-106	120-120
30	28-30	2.5	103-110	0.35	75-86	178-196	200-200
40	36-40	2.5	132-147	0.32	135-167	267-314	275-325
50	44-50	2.5	161-183	0.30	215-278	376-461	375-475
60	52-60	2.5	191-220	0.29	311-414	502-634	525-650
65	55-65	2.5	202-238	0.29	348-486	520-724	550-725
70	58-70	2.5	213-257	0.28	400-483	613-840	625-850
75	61-75	2.5	224-275	0.28	443-670	667-945	675-950
80	64-80	2.5	235-293	0.27	506-790	741-1,083	750-1,100

Source: Current AASHTO Policy (1, Table III-1)

Table 13. Calculated stopping sight distances for wet pavements.

Speed, V (MPH): 20

Grade, G (%) : 0.0

Reaction time, RT (sec): 2.0 2.5 3.0

Average deceleration (g):

1983 AASHTO POLICY (1971) (desirable)	NEW LOCKED WHEEL	2/32" LOCKED WHEEL	NEW CONTROL	2/32" CONTROL
0.400	0.431	0.405	0.326	0.304

G (%)	RT (sec)	V (MPH)	f (-)	Reaction (ft)	Braking (ft)	Total (ft)
0.0						
	2.0					
		20				
			0.400	58.7	33.3	92.0
			0.431	58.7	30.9	89.6
			0.405	58.7	32.9	91.6
			0.326	58.7	40.9	99.6
			0.304	58.7	43.9	102.5
	2.5					
		20				
AASHO 1965						-
AASHTO 1971						-
			0.400	73.3	33.3	106.7
			0.431	73.3	30.9	104.3
			0.405	73.3	32.9	106.3
			0.326	73.3	40.9	114.2
			0.304	73.3	43.9	117.2
	3.0					
		20				
			0.400	88.0	33.3	121.4
			0.431	88.0	30.9	119.0
			0.405	88.0	32.9	120.9
			0.326	88.0	40.9	128.9
			0.304	88.0	43.9	131.9

Table 14. Calculated stopping sight distances for wet pavement.

Speed, V (MPH): 30

Grade, G (%) : 0.0

Reaction time, RT (sec): 2.0 2.5 3.0

Average deceleration (g):

1983 AASHTO POLICY (1971) (desirable)	NEW LOCKED WHEEL	2/32" LOCKED WHEEL	NEW CONTROL	2/32" CONTROL
0.350	0.418	0.390	0.310	0.281

G (%)	RT (sec)	V (MPH)	f (-)	Reaction (ft)	Braking (ft)	Total (ft)
0.0						
	2.0					
		30				
			0.350	88.0	85.7	173.7
			0.418	88.0	71.8	159.8
			0.390	88.0	76.9	164.9
			0.310	88.0	96.8	184.8
			0.281	88.0	106.8	194.8
	2.5					
		30				
AASHO 1965						176
AASHTO 1971						196
			0.350	110.0	85.7	195.7
			0.418	110.0	71.8	181.8
			0.390	110.0	76.9	186.9
			0.310	110.0	96.8	206.8
			0.281	110.0	106.8	216.8
	3.0					
		30				
			0.350	132.0	85.7	217.7
			0.418	132.0	71.8	203.8
			0.390	132.0	76.9	209.0
			0.310	132.0	96.8	228.8
			0.281	132.0	106.8	238.8

Table 15. Calculated stopping sight distances for wet pavement.

Speed, V (MPH): 40

Grade, G (%) : 0.0

Reaction time, RT (sec): 2.0 2.5 3.0

Average deceleration (g):

1983 AASHTO POLICY (1971) (desirable)	NEW LOCKED WHEEL	2/32" LOCKED WHEEL	NEW CONTROL	2/32" CONTROL
0.320	0.396	0.359	0.290	0.247

G (%)	RT (sec)	V (MPH)	f (-)	Reaction (ft)	Braking (ft)	Total (ft)
0.0						
	2.0					
		40				
			0.320	117.4	166.7	284.0
			0.396	117.4	134.7	252.0
			0.359	117.4	148.6	265.9
			0.290	117.4	183.9	301.3
			0.247	117.4	215.9	333.3
	2.5					
		40				
AASHO	1965					263
AASHTO	1971					314
			0.320	146.7	166.7	313.4
			0.396	146.7	134.7	281.4
			0.359	146.7	148.6	295.3
			0.290	146.7	183.9	330.6
			0.247	146.7	215.9	362.6
	3.0					
		40				
			0.320	176.0	166.7	342.7
			0.396	176.0	134.7	310.7
			0.359	176.0	148.6	324.6
			0.290	176.0	183.9	359.9
			0.247	176.0	215.9	392.0

Table 16. Calculated stopping sight distances for wet pavements.

Speed, V (MPH): 50

Grade, G (%) : 0.0

Reaction time, RT (sec): 2.0 2.5 3.0

Average deceleration (g):

1983 AASHTO POLICY (1971) (desirable)	NEW LOCKED WHEEL	2/32" LOCKED WHEEL	NEW CONTROL	2/32" CONTROL
0.300	0.373	0.329	0.271	0.220

G (%)	RT (sec)	V (MPH)	f (-)	Reaction (ft)	Braking (ft)	Total (ft)
0.0						
	2.0					
		50				
			0.300	146.7	277.8	424.5
			0.373	146.7	223.4	370.1
			0.329	146.7	253.3	400.0
			0.271	146.7	307.5	454.2
			0.220	146.7	378.8	525.5
	2.5					
		50				
AASHO	1965					369
AASHTO	1971					461
			0.300	183.4	277.8	461.1
			0.373	183.4	223.4	406.8
			0.329	183.4	253.3	436.7
			0.271	183.4	307.5	490.9
			0.220	183.4	378.8	562.1
	3.0					
		50				
			0.300	220.0	277.8	497.8
			0.373	220.0	223.4	443.5
			0.329	220.0	253.3	473.3
			0.271	220.0	307.5	527.5
			0.220	220.0	378.8	598.8

Table 17. Calculated stopping sight distances for wet pavements.

Speed, V (MPH): 60

Grade, G (%) : 0.0

Reaction time, RT (sec): 2.0 2.5 3.0

Average deceleration (g):

1983 AASHTO POLICY (1971) (desirable)	NEW LOCKED WHEEL	2/32" LOCKED WHEEL	NEW CONTROL	2/32" CONTROL
0.290	0.353	0.301	0.254	0.194

G (%)	RT (sec)	V (MPH)	f (-)	Reaction (ft)	Braking (ft)	Total (ft)
0.0						
	2.0					
		60				
			0.290	176.0	413.8	589.8
			0.353	176.0	339.9	516.0
			0.301	176.0	398.7	574.7
			0.254	176.0	472.4	648.5
			0.194	176.0	618.5	794.6
	2.5					
		60				
AASHO 1965						491
AASHTO 1971						634
			0.290	220.0	413.8	633.8
			0.353	220.0	339.9	560.0
			0.301	220.0	398.7	618.7
			0.254	220.0	472.4	692.5
			0.194	220.0	618.5	838.6
	3.0					
		60				
			0.290	264.1	413.8	677.8
			0.353	264.1	339.9	604.0
			0.301	264.1	398.7	662.7
			0.254	264.1	472.4	736.5
			0.194	264.1	618.5	882.6

Table 18. Calculated stopping sight distances for wet pavements.

Speed, V (MPH): 70

Grade, G (%) : 0.0

Reaction time, RT (sec): 2.0 2.5 3.0

Average deceleration (g):

1983 AASHTO POLICY (1971) (desirable)	NEW LOCKED WHEEL	2/32" LOCKED WHEEL	NEW CONTROL	2/32" CONTROL
0.280	0.336	0.278	0.239	0.174

G (%)	RT (sec)	V (MPH)	f (-)	Reaction (ft)	Braking (ft)	Total (ft)
0.0						
	2.0					
		70				
			0.280	205.4	583.3	788.7
			0.336	205.4	486.1	691.5
			0.278	205.4	587.5	792.9
			0.239	205.4	683.4	888.8
			0.174	205.4	938.6	1144.0
	2.5					
		70				
AASHO 1965						600
AASHTO 1971						841
			0.280	256.7	583.3	840.0
			0.336	256.7	486.1	742.8
			0.278	256.7	587.5	844.2
			0.239	256.7	683.4	940.1
			0.174	256.7	938.6	1195.4
	3.0					
		70				
			0.280	308.1	583.3	891.4
			0.336	308.1	486.1	794.2
			0.278	308.1	587.5	895.6
			0.239	308.1	683.4	991.4
			0.174	308.1	938.6	1246.7

Table 19. Calculated stopping sight distances for wet pavements.

Speed, V (MPH): 80

Grade, G (%) : 0.0

Reaction time, RT (sec): 2.0 2.5 3.0

Average deceleration (g):

1983 AASHTO POLICY (1971) (desirable)	NEW LOCKED WHEEL	2/32" LOCKED WHEEL	NEW CONTROL	2/32" CONTROL
0.270	0.332	0.260	0.226	0.157

G (%)	RT (sec)	V (MPH)	f (-)	Reaction (ft)	Braking (ft)	Total (ft)
0.0						
	2.0					
		80				
			0.270	234.7	790.1	1024.8
			0.332	234.7	642.6	877.3
			0.260	234.7	820.5	1055.2
			0.226	234.7	943.9	1178.6
			0.157	234.7	1358.7	1593.4
	2.5					
		80				
AASHO 1965						741
AASHTO 1971						1083
			0.270	293.4	790.1	1083.5
			0.332	293.4	642.6	936.0
			0.260	293.4	820.5	1113.9
			0.226	293.4	943.9	1237.3
			0.157	293.4	1358.7	1652.1
	3.0					
		80				
			0.270	352.1	790.1	1142.2
			0.332	352.1	642.6	994.6
			0.260	352.1	820.5	1172.6
			0.226	352.1	943.9	1296.0
			0.157	352.1	1358.7	1710.8

Table 20. Recommended low-speed stopping sight distances (wet pavement).

V (mph)	Reaction Distance (ft)	Braking Distance f	Braking Distance (ft)	Recommended SSD Total(ft)	Current AASHTO Policy(ft)
10	37	0.47	7	45	50
15	55	0.44	16	75	80
20	73	0.41	30	105	120
25	92	0.38	49	150	150(160)

This model assumes:

$S_N(40) = 28$ (15% road)

tires 2/32" tread

locked wheel stop

reaction time 2.5 seconds

*For turning roadways

example cases. A comparative study of accident experience involving restricted SSD is also reported.

In order to respond to significant possible changes in design, it was found necessary to develop some mathematical relationships for horizontal and vertical curves not found in the literature. The results have been used where appropriate and derivations are included in Appendix E.

Grades

A calculation of the extra length of available SSD on roads on constant upgrades over the horizontal distance reveals that the following additional actual stopping distance is available. This increase is clearly of no practical importance.

Grade (%)	Extra Length (ft/station)
5	0.1
10	0.5
15	1.1

A more important grade correction is the lengthening of the required SSD when a vehicle stops on a downgrade. This effect has been explored in Table 10 for braking distance. It is noted that AASHTO (1) uses the design speed of the road in calculating downgrade SSD lengthening and the average running speed on upgrade SSD shortening.

Vertical Curvature

SSD affects vertical alignment on tangent roadways in two ways, on crest and sag vertical curves. The line of sight from the driver's eye to the obstacle is broken by the road surface for the crest curve and by an overhead structure for the sag curve. In addition, after dark the headlamp illumination will affect available SSD on both types of curves.

Some results of the analyses of vertical curves are presented in this section. Most are for crest curves. The TRB Committee on Operational Effects of Geometrics recently reported on its research needs priorities, and among the four problems with the highest priority is one dealing with design controls for crest vertical curves.

<u>Mathematical Development</u>. Appendix E contains a development of the fundamental relationships involved in available SSD for crest vertical curves. It was undertaken to provide a means of obtaining sight distance graphs analytically rather than graphically (<u>5</u>, p.151). With the mathematical relation these graphs could then be plotted by computer. Sight distance graphs make it possible to determine easily that portion of the length of a crest vertical curve for which the available SSD is at its minimum value as well as providing a basis for evaluating other values of required SSD for that curve. Figure 9 presents an example of such a graph.

<u>Eye and Object Height Sensitivity Analysis</u>. For a given required SSD, there has been much discussion of the effect of eye and object height on the length of crest vertical curves. A sensitivity analysis of the effects on L

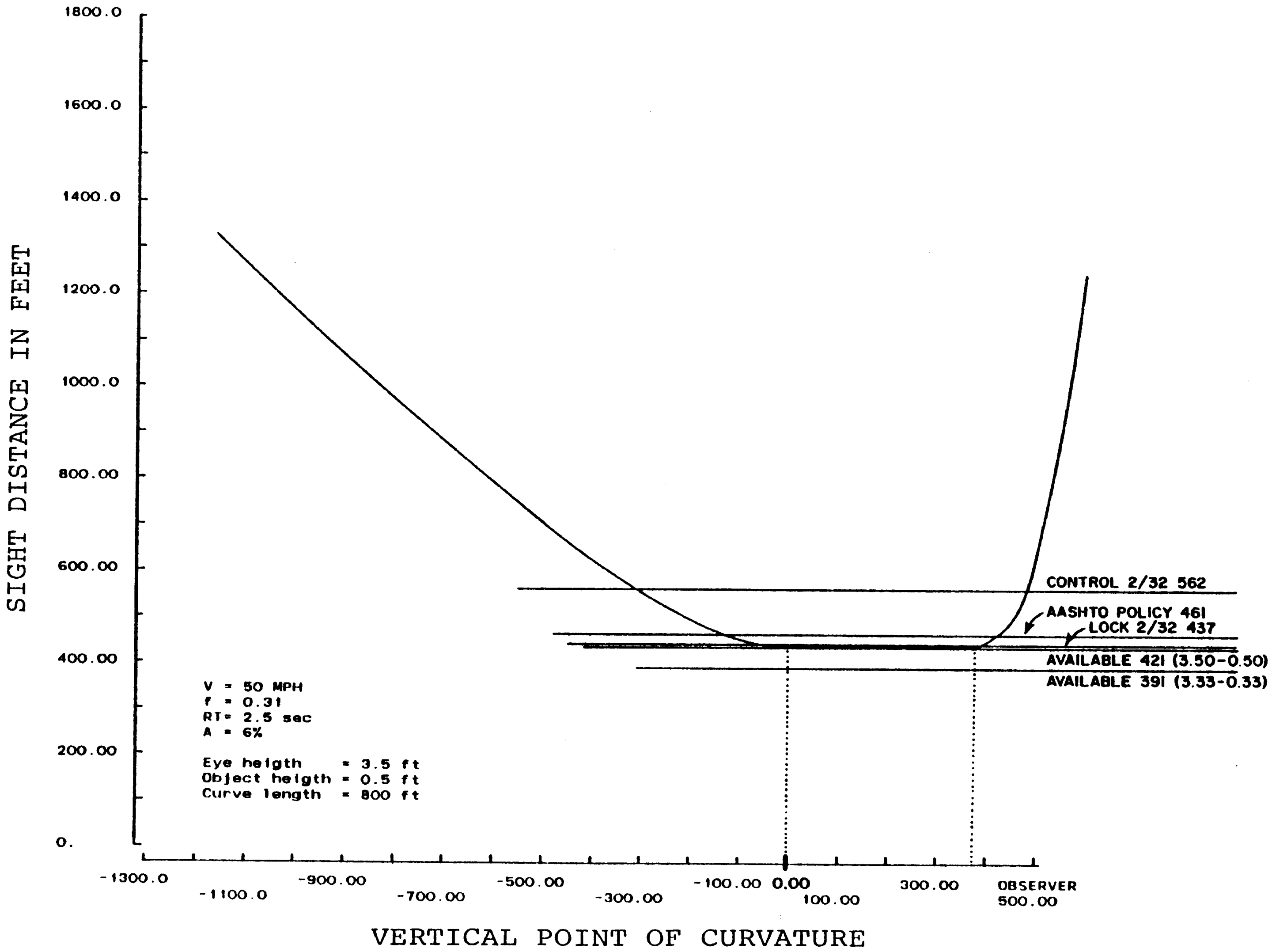


Figure 9. Sight distance graph for V=50 mph and L=800 feet.

of varying these elements was made. The values used in the study are given as follows:

Height of Eye = 3.75, 3.50 and 3.33 ft

Height of Object = 0.5, 0.33 ft

They were evaluated for a reasonable range of the algebraic change of grade, A, and for a required SSD of 650 ft. This SSD corresponds to the upper value in the current policy for a design speed of 60 mph (about 53 mph in the recommendations of this research). The analysis is presented in Table 21. The lower object height requires about 10 percent more vertical curve length, while a 40 in. eye height needs about 3 percent more length than the 3.5-ft. height. For grade changes of 6 percent or more, the difference in the length of crest vertical curve required is important, with the more restrictive values of the height parameters lengthening L about 21 percent, a significant joint effect.

Stopping Distance Sensitivity Analysis. The sight distance graph has been used as an aid in this sensitivity analysis. Five crest vertical curves have been selected for presentation. In one case a vertical curve was designed based on the current lower AASHTO required SSD. In the other four cases the curves were designed based on current upper AASHTO SSD values. The value of L selected was rounded up to the nearest 100 ft. The available SSD for that design is shown by the plot points. SSD values for other conditions have been entered on the graph.

Table 21. Eye-object height sensitivity analysis-crest vertical curve V=60 mph, required SSD=650 ft.

A	Length of Curve					
	h_e 3.75 ft		h_e=3.5		h_e=3.33	
	h_o=0.5	h_o=0.33	h_o=0.5	h_o=0.33	h_o=0.5	h_o=0.33
2%	602	670	635	708	659	733
4%	1208	1340	1272	1414	1317	1466
6%	1813	2010	1907	2120	1977	2201
8%	2418	2680	2542	2826	2636	2935
10%	3023	3350	3178	3533	3295	3669
12%	3627	4020	3814	4240	3954	4403

In evaluating existing curves or candidate designs the duration in time or distance of a deficiency from desirable values is believed to be important. Using a sight-distance graph one can see that the duration of a deficiency of a known amount for only 0.5 sec is not as critical as one lasting for 15 or 20 sec.

Figure 9 shows the sight-distance graph for a 50-mph design curve with a 6 percent change of grade designed to AASHO lower standards with an eye height of 3.75 ft and an object height of 0.5 ft. This type of design could have originated in the 1950's and 1960's. It is quite inadequate for the wet pavement controlled stop with the 2/32-in. tread tires used as the basis for recommendations in this study. The deficiency is not great for the locked wheel stop and the current eye-object height standards. The current AASHTO upper SSD value is 70 ft greater than that given by this curve for the 40"-4" eye-object height. The excellent SSD for the first 200 ft of the curve can be seen.

Figure 10 shows an urban-like design for a 30-mph design speed; a 6 percent change of grade, and a design following the current AASHTO upper policy (L = 200 ft, eye height=3.5 ft, object=0.5 ft). This design is more than adequate for all conditions.

Figure 11 presents a current AASHTO upper policy design for a 50 mph road with A = 4 percent. The results are similar to those shown in Figure 9.

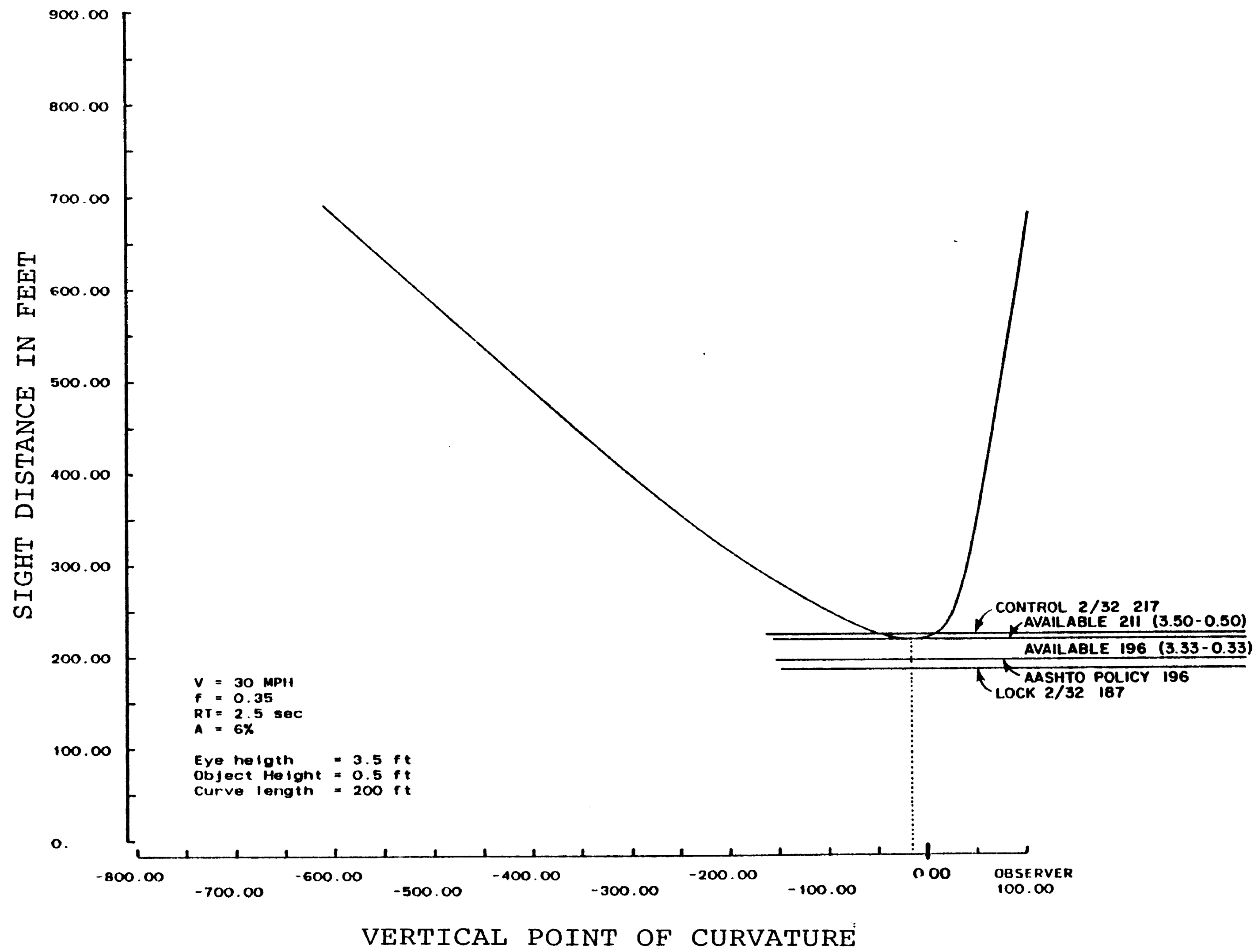


Figure 10. Sight distance graph for V=30 mph and L=200 feet.

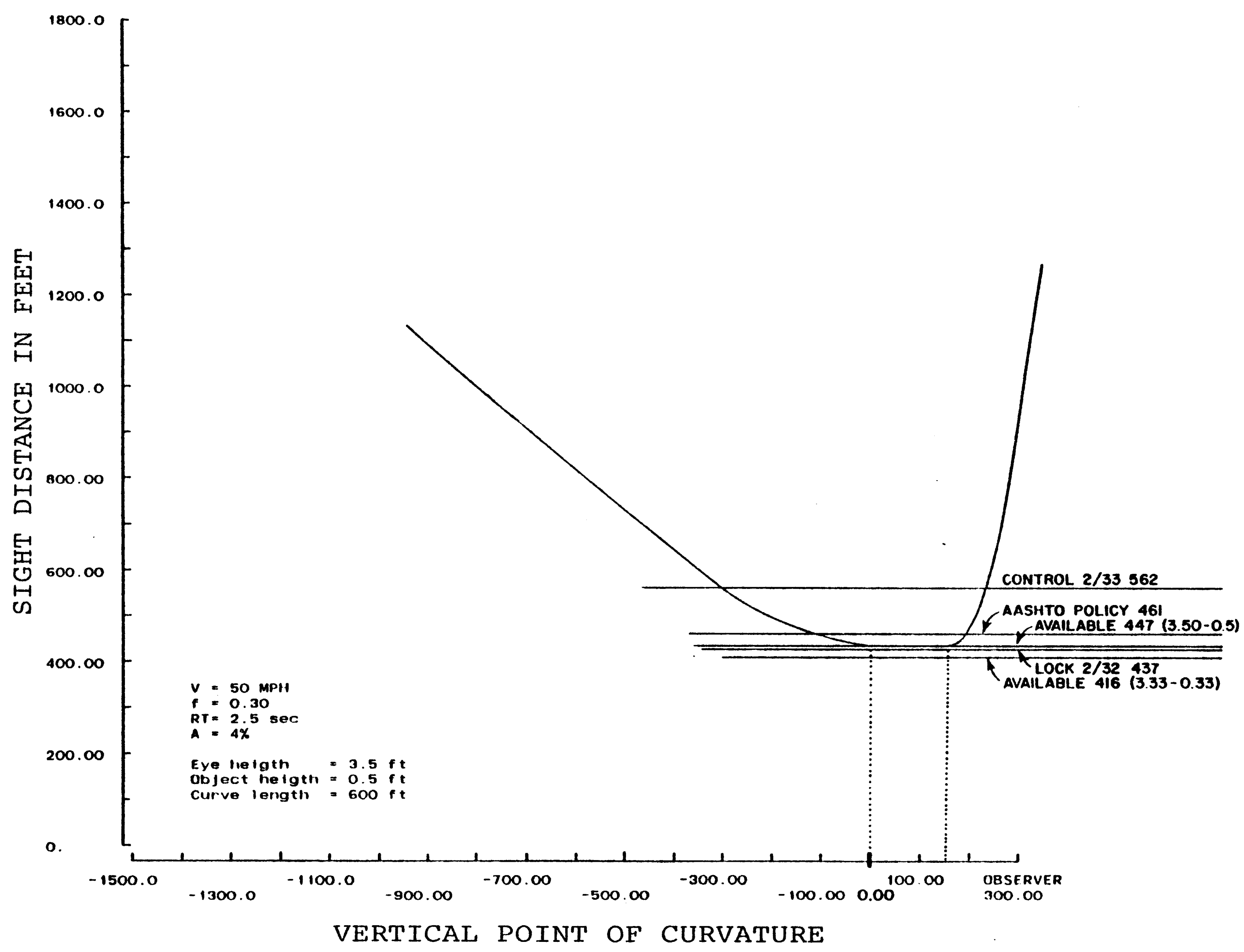


Figure 11. Sight distance graph for V=50 mph and L=600 feet.

Figure 12 is the same as Figure 11, except that the change in grade is much more severe, A = 12 percent. A 1,400 ft vertical curve is clearly inadequate.

Figure 13 presents a current upper policy design for a 70-mph A = 4% very high type situation. The deficiency for the controlled stop extends over almost the entire length of the vertical curve.

The crest vertical curve is often used to explore SSD problems. Appendix C presents an analysis of the vertical curve as it relates to seeing with headlamp illumination after dark. In this chapter the braking distance analysis comparison of trucks with passenger cars also uses a crest vertical curve.

Sag Vertical Curves. The current AASHTO models for sag vertical curves based on SSD considerations consider two cases. In one case a headlamp beam 2 ft above the road surface with an upward divergence of the light beam from the axis of the vehicle of 1 deg is used. Where this beam strikes the road surface is defined as the available SSD. In the second case a structure over the road obstructs the line of sight of the driver. Each of these cases is considered in turn.

For the headlamp case, using a 1-deg upward divergence from the headlamp as the criterion for sight distance on sag vertical curves makes assumptions about visibility with headlamps that are not supported by the facts. In Appendix C data are supplied concerning driver response distances to

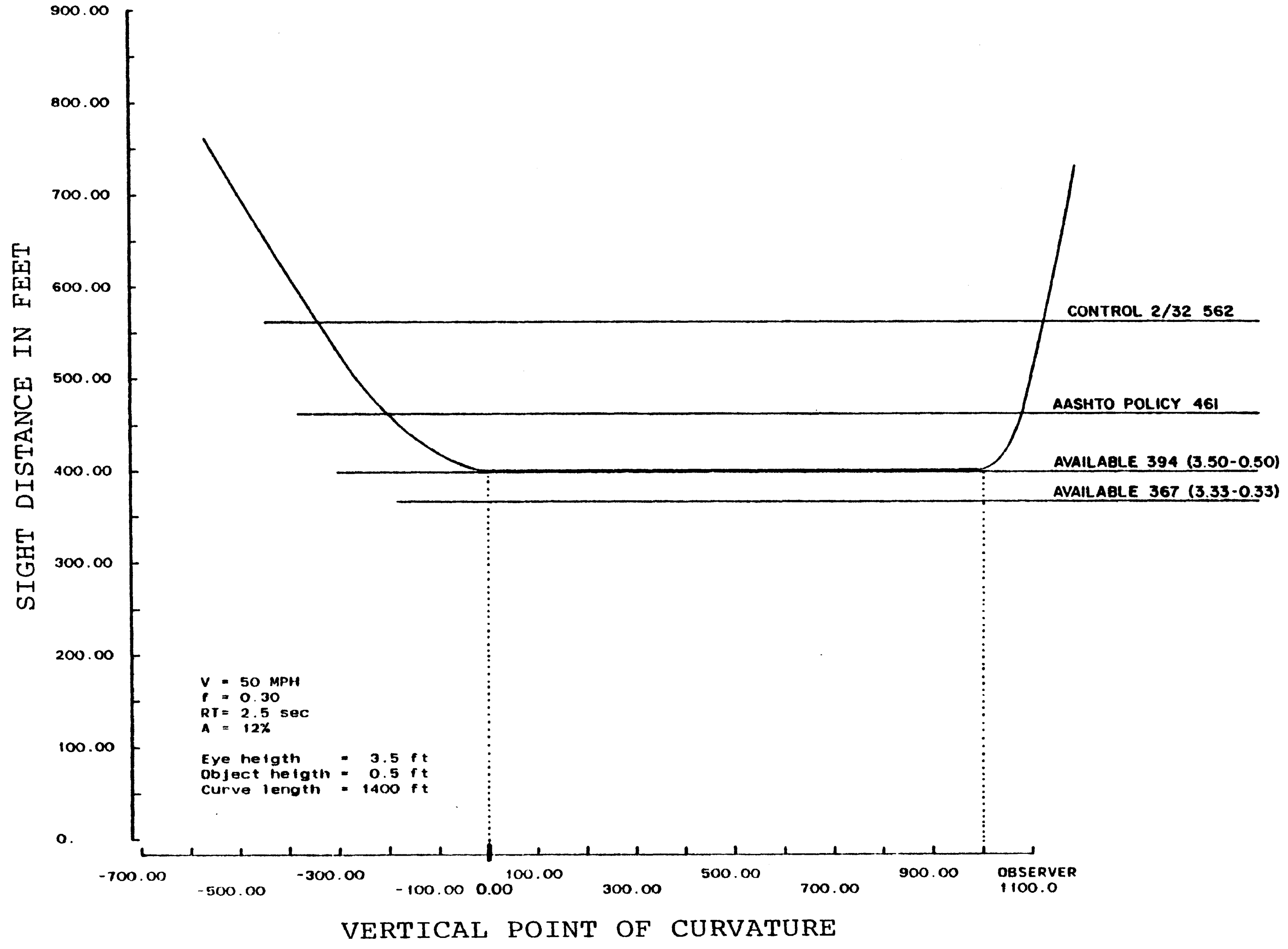


Figure 12. Sight distance graph for V=50 mph and L=1,400 feet.

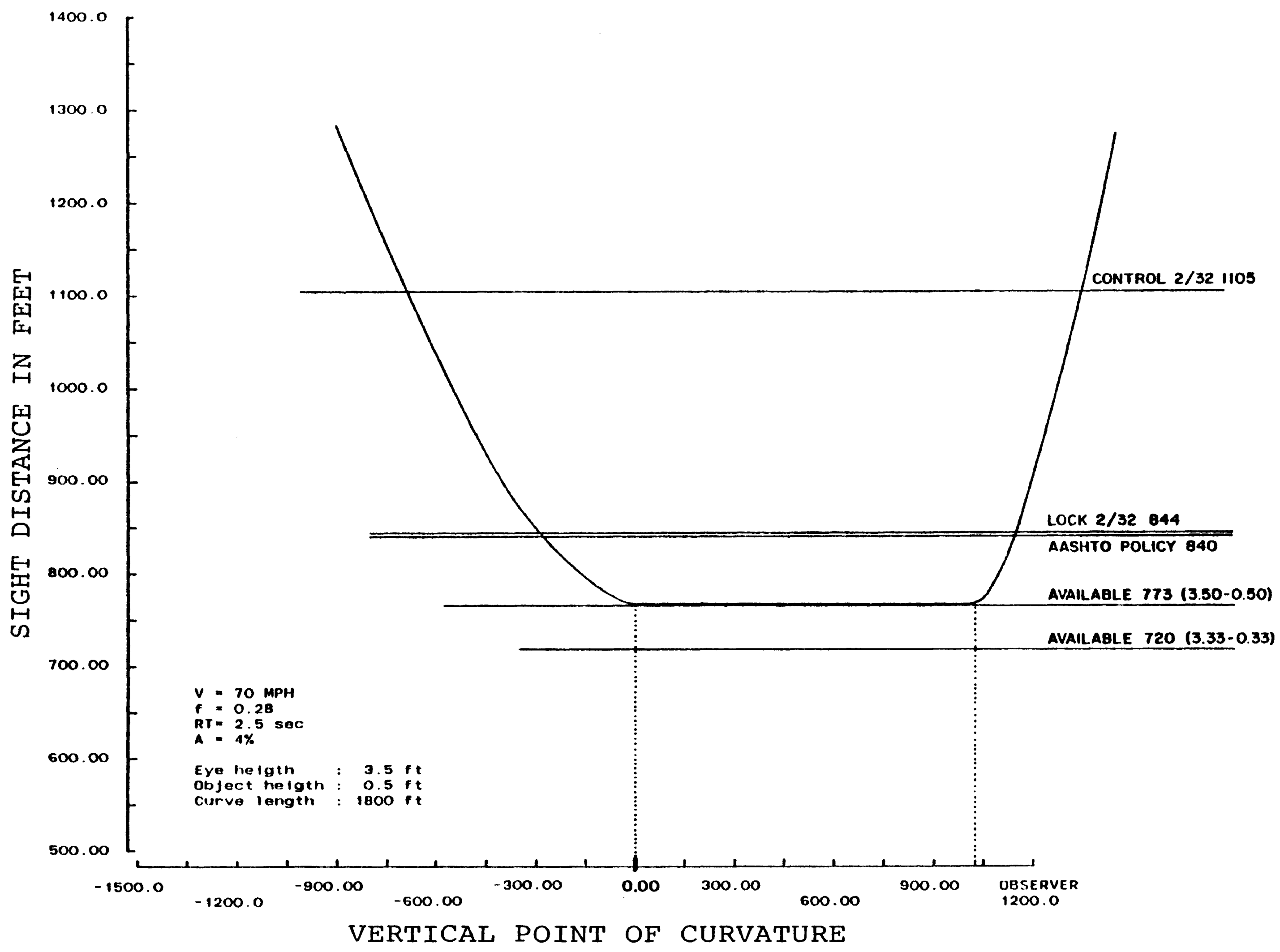


Figure 13. Sight distance for V=70 mph and L=1,800 feet.

small obstacles on the roadway under headlamp illumination. With the obstacle on the right side of the lane, the 5th to 95th percentile response distances were 25 to 200 ft. With the obstacle on the left side of the lane, the same percentile response distances were 15 to 110 ft. To obtain the position at the start of the perception interval it is appropriate to add a distance equal to 1.1 sec of travel.

The obstacles in this test were on the pavement and below the level of the headlamps. If it is assumed that low-beam lamps are being used, there is a very large drop in output above horizontal. This is exemplified in Table 22, which is a candela matrix for a fairly typical U.S. low-beam unit. Note the difference in intensity comparing 1 deg down (about where the test obstacles were detected), and 1 deg up. Clearly, the discouragingly short distances mentioned above are significantly longer than could be expected if the obstacle were located 1 deg above the headlamps.

If high beams can be assumed, the situation becomes somewhat better, since there is little difference between 1 deg down and 1 deg up. However, the 5th percentile response distance would probably still be no better than about 100 ft when dealing with low-contrast obstacles. High-contrast (but diffusely reflective) obstacles might improve on this figure by about 100 percent, far short of most of the SSD values shown in the table.

The only type of target to which the present lighting criterion can be applied with any confidence is a

Table 22. Candela matrix for typical U.S. low-beam headlamps.

		Degrees Left										V	Degrees Right									
		10	9	8	7	6	5	4	3	2	1	0	1	2	3	4	5	6	7	8	9	10
Degrees Up	2	200	200	200	200	200	200	200	200	200	300	300	400	500	500	500	400	400	400	300	300	300
	1	400	400	400	400	400	400	400	400	500	600	800	1000	1100	1100	1000	800	700	600	500	500	500
	0	900	700	700	700	700	800	800	900	1000	1300	2600	5800	6600	5800	4900	3500	2500	1900	1300	1000	1000
Degrees Down	1	2200	1900	1900	1900	1900	1900	2100	2300	2800	3800	9200	21900	26900	22400	17300	12100	8500	5600	3500	2700	2500
	2	3500	3300	3300	3300	3400	3500	3700	4400	5900	8700	15000	28700	33700	29800	23800	18200	14200	9800	6500	5200	4900
	3	2900	3200	3200	3200	3300	3400	3600	4700	6200	8600	11500	16500	18500	18600	16400	13600	10900	7900	5700	5100	5000
	4	900	1400	1500	1500	1500	1600	1700	2000	2600	3400	3600	4500	5000	5700	5800	4800	3800	2900	2600	2500	2500

retroreflective one. So, pavement markings, signs, barricades, vehicles, and anything else carrying retroreflective material have a good chance of being detected at the distances given in the table, using illumination projected 1 deg up from low-beam lamps. For this reason the standard should be retained. However, consideration should be given to making the reference something likely to be detected (e.g, vehicle rear markers).

For the case of the structure located over the road and with the intrado centered over the vertical point of intersection (VPI), the equations found in the 1965 AASHO Policy (5, p. 526) have been evaluated for a truck driver eye height of 9.0 ft and an object height of 0.5 ft. This eye height is typical of that found in cab-over-engine tractors. The resulting equations are:

$$L = 2S - 74/A \qquad L \leq S \qquad (21)$$

$$L = AS^2/74 \qquad L \geq S \qquad (22)$$

The resulting curves are about 10 percent longer than those found in the 1965 Policy.

Length of Curve. In highway practice the vertical curve length is defined by its horizontal projection. The actual length of the curve and, hence, available SSD is greater. An analysis shows that the actual curve length, L_A^1 can be expressed by

$$L_A^1 = L \ (1 + A/100)^{1/2} \qquad (23)$$

A brief tabulation of this result is shown as follows:

A(%)	L_A^1
3	1.015
6	1.030
9	1.044
12	1.058

This effect, about half A in percent, is significant for differences of grade of 6 percent or more. Ignoring its effect leads to selection of greater SSD than is needed.

<u>Stopping on Vertical Curves</u>. As a braking vehicle slows on a vertical curve the gradient is constantly changing (at the constant rate K = L/A). One must equate kinetic energy to work done and integrate to solve for the braking distance. Accordingly, the relationship to use is based on taking into account the changes in elevation because of the grade effect.

$$V^2/30 = (BD)f + h \tag{24}$$

where:

BD = braking distance, ft;

f = deceleration rate, g;

V = speed, mph;

h = change in elevation to stopping; and

g(x) = the slope of the curve at station x along the curve.

It is relatively easy to solve for BD for the four possible cases for the initial and final position of the vehicle with respect to the curve. This effect has not been quantified in this research since its magnitude is usually much less than A percent and is generally not great.

Another effect is that of centrifugal force as the vehicle follows a curved vertical path with radius R. On a crest curve this force reduces the effective weight of the vehicle, and on a sag curve its weight is increased. This "unloading" effect on a crest curve was studied.

The instantaneous radius of curvature R at station X was calculated and determined to be approximately:

$$R = L/a = 100(K) \tag{25}$$

where a = A/100. This radius was shown not to change much with position along the curve.

When a vehicle passes over a design crest vertical curve the unloading increases the braking distance, BD, by about 2 percent for speeds of 50 mph or less and 1.5 percent for higher speeds when current lower AASHTO policy values are used. When upper values are used the increase is less than 1 percent above 50 mph and reaches 2 percent only at 30 mph. The effect on curve length would be to increase it from 2 to 4 percent. A similar effect in the opposing direction is found for the sag curve.

Horizontal Curvature

This section is concerned with sight lines crossing the inside of horizontal curves and the needed clearance from the path of the vehicle to objects. It successively considers the following elements:

1. Effect of recommended required SSD on clearance need.
2. Clearances needed near the end of curves.
3. Effect of required SSD>L on clearance needs.
4. Sensitivity analysis of lateral eye and object position.

Derivations and draft design-aid curves are presented in Appendix E. In the appendix, derivations for the case where required SSD > L have been made. The effects of this situation have been explored there as well.

Effect of Required SSD on Clearance. The standard design relationship between the curvature, required SSD, and obstacle clearance (5) has been explored for this research. The effect on the needed clearance distance, M, for curves for higher design speeds is shown in Table 23. In this research M and R are measured from the position of the driver's eye and object. The large increase in the need for a cleared area on the inside of the curve is apparent.

Clearance Requirements Near Ends of Curve. Because of the large increase in the offset to an obstruction on a horizontal curve resulting from the increased required SSD shown in Table 23, it becomes more important to consider the two situations that reduce the offset from that found when both the observer and object on the roadway are in the

Table 23. Horizontal sight distance clearance.

Design Speed (mph)	Typical Degree of Curve (deg)	AASHTO Upper Policy		Recommended	
		SSD (ft)	M (ft)	SSD (ft)	M (ft)
50	8	475	38	560	54
60	5	650	46	840	78
70	4	850	63	1200	124
80	3	1100	78	1650	180

horizontal curve. The first of these cases is when the SSD is greater than the length of the curve, L. This case has not been treated by AASHTO except to mention that it can be developed graphically. In the second situation, when the observer is on the tangent within a distance of SSD from the PC (or object beyond PT), the offset needed to have a clear line of sight varies nonlinearly from 0 at a point SSD in advance of the PC to the full value of M for the standard case when the observer is at the PC (or object at the PT). All points on the curve closer than SSD/2 to the PC will need less than M. Also, points along the tangent will require a clearance that has not been discussed in earlier AASHTO publications. In this research, both of these modifying elements have been explored in Appendix E and the results related to M for convenient application.

The required horizontal line of sight is determined by the location of the driver's eye and the object. Assuming that the object is on the same path as the driver's eye the line of sight begins to move toward the inside of the curve when the eye is at a distance SSD before the PC of the curve. This is shown in Figure 14.

Referring to Figure 14 and considering m(x) for points on the tangent prior to the PC and within a distance S/2 (SSD/2) from the PC, one can show by elementary calculus, similar triangles and the relationship of the offset (O) from a tangent extended a distance, T, from the PC ($T^2 = 2$ RO), that

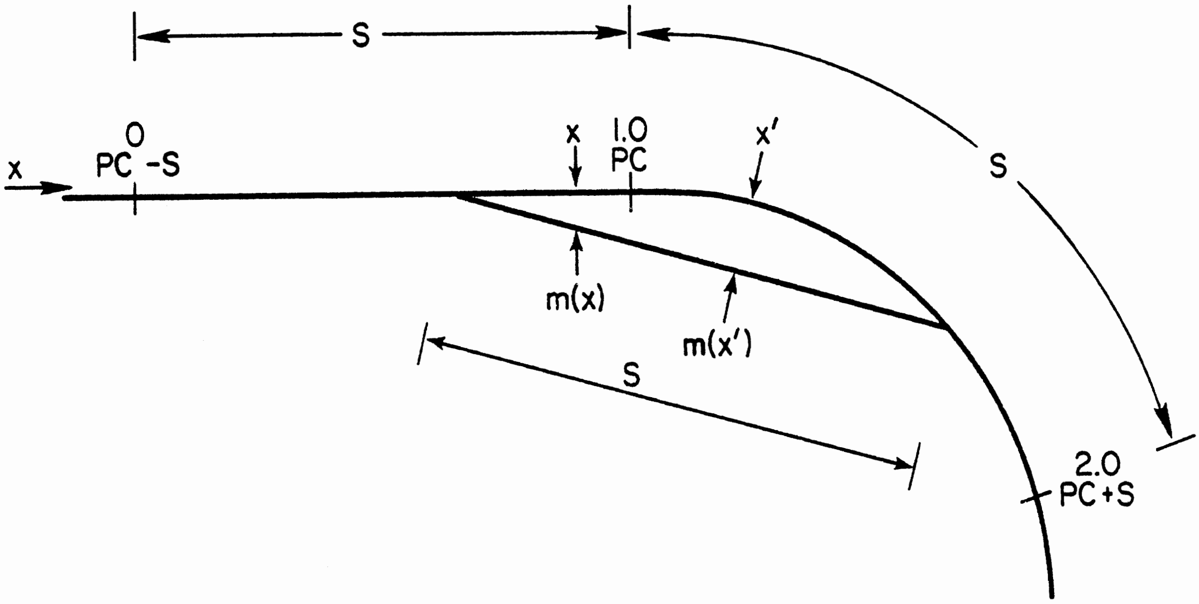


Figure 14. Clearance requirements near horizontal curve.

$$m(x) = (S^2/18R)\ (x + x^2/3) \qquad 0 \leq x \leq 1.5 \qquad (26)$$

where x is the distance from a point S in advance of the PC expressed as a fraction of S (SSD).

As an example, consider a curve designed for V = 50 mph (S = 475 ft developed from current AASHTO lower SSD requirements) with R = 704 ft (D = 8 deg). Table 24 is obtained. It can be seen that although the maximum value, M, is 40.1 ft this is not reached until the driver is 238 ft (S/2) into the curve. By symmetry, no 8 deg curve shorter than 475 ft will require the full 40-ft clearance. It should also be noted that there is a clearance requirement on the tangent that in this case is 23.7 ft from the center of the lane at the PC.

Figure 15 compares the need for clearance for a typical horizontal curve for AASHTO upper and recommended SSD values.

Clearance Needs on Curves Where SSD > L. When the required SSD is lengthened so that it exceeds the length of the horizontal curve, there is again a reduction in m below M This is shown for various SSD/L ratios in Figure 16. For example, on short curves where the SSD is twice the curve length, m is only 75 percent of that required if the curve were longer than SSD. This relationship is based on an accurate chord approximation to the path followed by the vehicle.

Raymond (72) developed a graphical method of determining the relationship between the offset to an

Table 24. Curve end SSD clearance transitions.

Location	Sta	X	m(ft)
S before PC	0+00	0.00	0
S/2 before PC	2+38	0.50	3.0
Clearance pt. is at edge of 8' shoulder	3+99	0.84	14.0
PC	4+75	1.00	23.7
PC+200	6+75	1.42	39.7
S/2 beyond PC	7+13	1.50	40.1

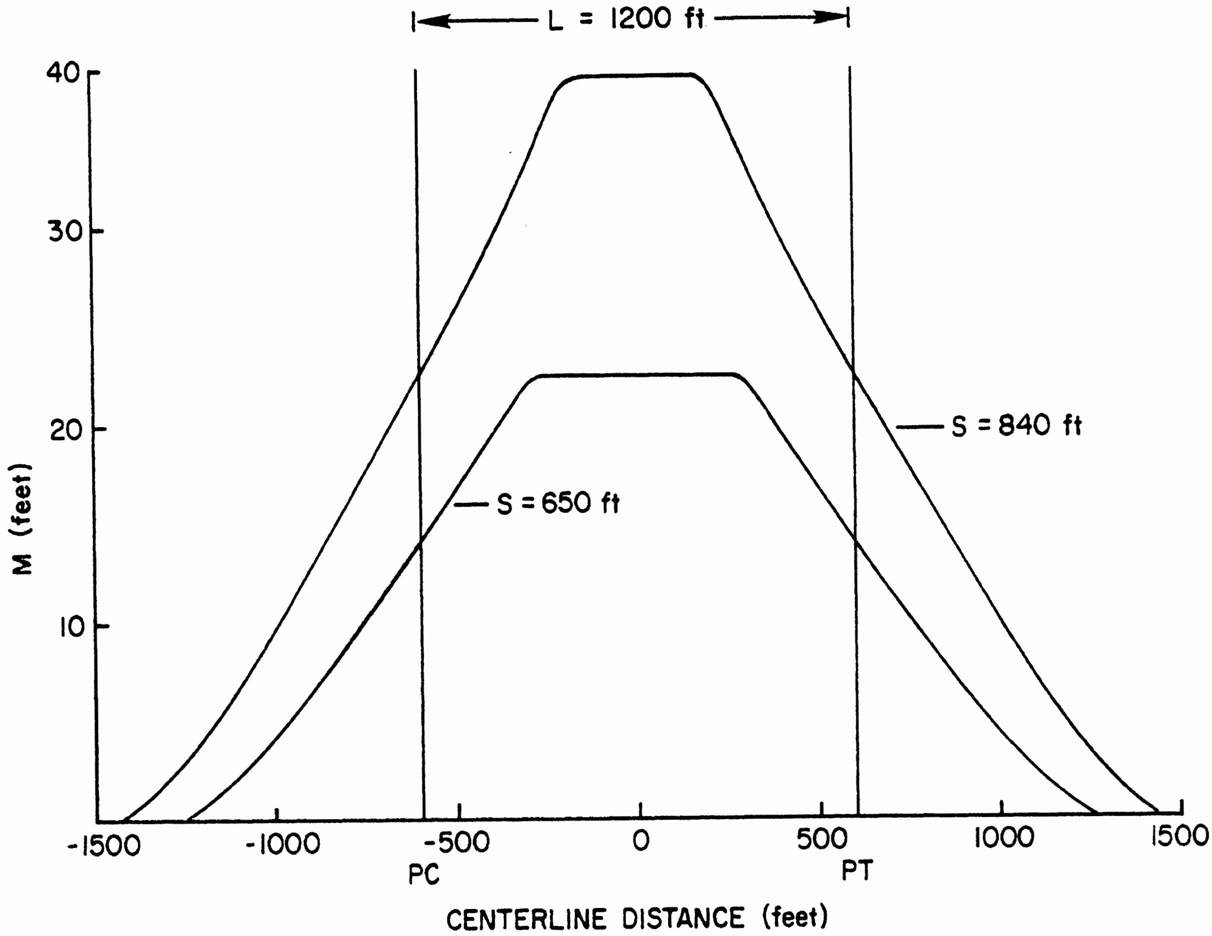


Figure 15. Horizontal curve clearance needs.

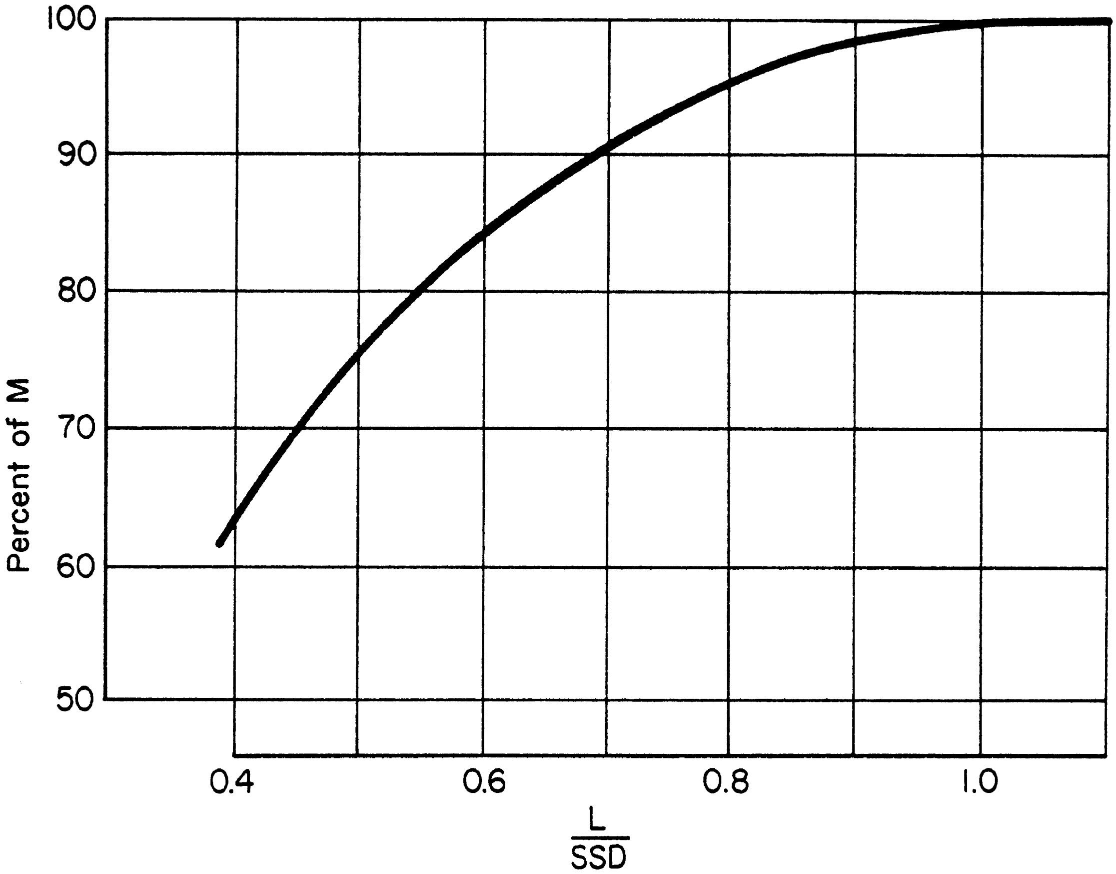


Figure 16. Reduction of middle ordinate as percent of M when SSD<L.

obstacle near the end of a horizontal curve, the full offset on the curve and the available SSD. He did this by computer sensitivity analysis and also included the effect of various spiral transition lengths. He demonstrated that the relationship is independent of the curvature except where the radius is less than S, then an error on the low side of as much as 10 percent occurs. The research team did not find any other treatments of this problem in the literature. In Appendix E mathematical relationships for these design elements have been derived using the line of sight distance as an approximation to the vehicle path length.

Effect of Spiral Transition. The spiral transition curve reduces the need for object clearance on the inside of the path of travel when the driver is on the tangent section. The magnitude of this effect was developed for a range of typical AASHTO transition designs for speeds from 50 to 80 mph. It was found that the ultimate offset value for the spiral is about 4 ft. This correction could be applied as a reduction in the required m in situations where a spiral is used.

Effect of Horizontal Curvature Forces. It is known that the side acceleration experienced on a horizontal curve, f_e, decreases the available deceleration for stopping, f. The available total friction, f_t, is related to the others by

$$f_t = (f_e^2 + f^2)^{1/2} \qquad (27)$$

Using the force equilibrium relationship for horizontal curvature and the braking distance relationship one obtains:

$$f^2 = f_t^2 - (V^2/15R - e)^2 \qquad (28)$$

At high speeds this effect could be significant on curves with minimum radius designs for those speeds and should be considered by design engineers.

Lateral Eye and Object Sensitivity Analysis. Design SSD on a horizontal curve is typically determined for the situation where both the observer and the object are positioned on the centerline of the critical lane. Typically, a driver's eye is not positioned at the centerline of a lane, but is more likely to be about 3 ft from the centerline away from the center of the lane. On the more critical inside lane of a horizontal curve this would place the observer's eye about 3 ft measured laterally toward the outside of the curve. An object that is at the centerline is obviously cause for concern, but an object that is only 1 or 2 ft from the edge of the pavement can still present a problem for the vehicle.

Sight distance on a horizontal curve changes as the observer changes to another position on the cross section of the road. Similarly, the position of the object on the cross section affects whether or not it is in the observer's line of sight.

A comparison of SSD was carried out with the observer and object on the centerline of the critical lane and with the observer 3 ft from the centerline of the critical lane toward the outside of the curve and the object 4 ft from the centerline toward the inside of the curve, which corresponds to an object about 2 ft from the pavement edge. The results of the comparison showed that the SSD in both situations were not significantly different from each other.

Examples of Impacts

A number of illustrative geometric examples for actual highway conditions have been prepared and presented elsewhere in this report. This section explores some additional problems of concern. It looks at two vertical crest curve cases, an excavation and an embankment. One of the main effects of greater SSD is the lengthening of vertical curves. The result often is a significant increase in the amount of excavation or embankment earthwork quantities.

Farber (73) has presented a graph indicating that earthwork quantities increase with SSD raised to the fourth power. While this effect is considerable, the quantity is more appropriately represented by SSD to the third power or less when SSD > L and to more than the fourth power when SSD < L, the usual case.

The example in this section first considers excavation in which the existing ground is assumed to be level and at the elevation of the VPI. The difference in excavation (in cubic yards) on a crest vertical curve between the vertical point of curvature (VPC) and vertical point of tangency (VPT) of the longer vertical curve can be shown to be

$$D = 1.543.10^{-5}AW(L_e^{\,2}-L_s^{\,2}) + 5\ .787.10^{-9}A^2R(L_e^{\,3} - L_s^{\,3}) \qquad (29)$$

where:

D = difference in excavation, cu yd;

A = absolute value of the algebraic difference in grade, %;

W = width of roadbed, no ditch assumed, ft;

R = slope of roadside (R, horizontal to 1, vertical);

L_e = length of longer vertical curve, ft; and

L_s = length of shorter vertical curve, ft.

This expression has been evaluated for several reasonable combinations of vertical curve length reflecting existing AASHTO policy and this research and the differences are shown in Table 25. The differences are large for the higher speed curves. Assuming some recent costs of such excavation of $1.03 per cubic yard, additional costs of this pay item alone would be as much as $133,000. Resurfacing of up to 0.6 mi and reworking drainage structures would increase the retrofit cost much more.

Second, an example is presented of the effects on embankment requirements for the moderate design speed overcrossing in flat terrain as discussed in the current AASHTO policy (1, pp. X-25 - X-27). A road with a rise of 25 ft was selected. The other parameters selected were:

Table 25. Earthwork differences in a cut.

Design Parameters	Vertical Curve Length		Excavation Difference (cu yd)
	Short	Long	
W = 40 ft	300	500	633
R = 2:1	800	1200	3,469
A = 6%	1100	2000	13,110
	1600	3000	33,391
W = 44 ft	300	500	1,630
R = 4:1	800	1200	10,570
A = 4%	1100	2000	44,960
	1600	3000	128,813

Design Speed = 40 mph

Roadbed Width (W) = 40 ft

Grade (G, 100g) = 5%

A = 10%

Side Slope (R) = 4:1

A vertical curve length of 600 ft was selected for the current design (ASSHTO lower=560 ft, AASHTO upper value = 800 ft). A comparable vertical curve for the recommended SSD would be 975 ft. However, a vertical curve length of 900 ft was selected for this example. Since the effect of the sag curves linking the grades with the level ground would be similar, their effect is ignored.

The difference in earthwork for this special case was determined. For the 1,000-ft length of interest the longer vertical curve requires an additional 62,800 cu yd of material ($94,000 at a recent value of $1.50/cu yd). This means a total of about 250 percent of that required for the shorter curve. The location of the VPI for the longer curve is 7.5 ft higher than that for the existing intersection. More than twice as much embankment is required under the 40-ft roadbed for this curve.

The values of lateral clearance on horizontal curves required for SSD indicate a need for special attention to this element of design, particularly where structures and railings are the potential off-road design hazard. As in interchange design (5, p 526) the need to reduce curvature to achieve needed SSD must be recognized. This results in the need for the selection of degrees of curvature of much less than half the maximum curvature at such locations.

Increasingly, longitudinal barriers are higher than the design driver eye height and there is accordingly limited SSD on horizontal curves. Considering such a barrier next to a high-speed traffic lane on a freeway, M is often equal to 20 ft. The following tabulation, based on the required perception-reaction-braking SSD developed in this report, shows that this is only suitable for relatively low speeds for the range of curve sharpness frequently encountered.

D(deg)	R(ft)	SSD(ft)	V(mph)
2	2865	667	54
3	1910	553	50
5	1046	409	42

Safety and SSD

Although it is accepted that sight distance has impacts on safety, the relationship has never been successfully quantified. The review of the literature (summarized in Appendix E) shows that there have been a number of studies of the relationship between SSD and safety, but many of the results are not supported and in some cases conflicting. The problem with most of the studies is that it is difficult to separate sight distance effects from other roadway design elements and to maintain proper controls. There is an obvious correlation between sight distance and alignment. Where roads have no horizontal or vertical curvature available SSD is unlimited and such roads are the safest.

Analyses of the effect of sight distance separate from horizontal or vertical curvature have not been found.

As part of this research, a study with the objective of examining the effects of available SSD on safety was carried out. The study was designed to compare the number and type of police-reported traffic accidents over a period of several years that occurred on each of a pair of road segments matched for traffic characteristics, road design factors, roadside features, and abutting land use. The two segments were to be within 1.0 mi of each other on the same road. The road segments in the sample were to be on tangent sections of road, one segment was to have available SSD below the 1965 AASHTO standard and the other was to have available SSD meeting the 1965 AASHTO standard. The limited SSD sites were to be on crest vertical curves. The original intention was to identify such sites where the limited SSD site and its matched site, called the control, would have the same algebraic difference of grades but different lengths of curves. However, identifying such a matched pair sample of sites was not possible and this requirement was relaxed. The final sample consisted of 10 matched pairs of two-lane rural road segments from Oakland and Washtenaw Counties in southeastern Michigan. The limited available SSD site and its control were all on the same road within one mile of, but not adjacent to, each other. There were no major intersections between the two sections. They were matched for traffic volume, type of traffic, lane widths, shoulder types and widths, roadside characteristics and abutting land uses. The vertical alignment on all the segments had not been changed over the years for which the number of accidents were counted. Any changes in pavement markings, and pavement or shoulder improvements were made at the same time on the limited SSD site and its matched control site.

Table 26 presents a summary of the sample and the accident counts for the limited SSD safety study. The detailed descriptions of the sites including the measured SSD are given in Appendix E.

There was a total of 136 accidents for 30.28 mi-years of exposure. Of these, 82 accidents occurred on the limited SSD sites and 54 occurred on their control sections. No particular pattern of accident types was found in comparing accidents at the limited SSD sites and their matched controls in this sample.

Comparison of the number of accidents on the limited SSD sites and the matched controls by standard statistical techniques shows that there is a significant difference ($\alpha = 0.05$) between the accident experience of the limited SSD sites and their matched controls. This difference is significant when all accidents from the limited SSD sites are compared to all the accidents from all the matched control sites and also when a pairwise comparison is made. The details of the statistical analysis can be found in Appendix E.

Table 26. Summary of sample in limited SSD safety study.

Site	Type	Length (mi)	No. of Years	No. of Accidents
1	Limited SSD	.50	6	11
	Control	.50	6	3
2	Limited SSD	.23	6	1
	Control	.23	6	0
3	Limited SSD	.40	6	2
	Control	.40	6	2
4	Limited SSD	.25	6	7
	Control	.25	6	1
5	Limited SSD	.22	6	13
	Control	.22	6	6
6	Limited SSD	.25	6	17
	Control	.25	6	26
7	Limited SSD	.24	4	24
	Control	.24	4	13
8	Limited SSD	.15	6	5
	Control	.15	6	2
9	Limited SSD	.17	6	2
	Control	.17	6	1
10	Limited SSD	.20	6	0
	Control	.20	6	0
Total	Limited SSD			82
	Control			54

CHAPTER THREE

INTERPRETATION, APPRAISAL, APPLICATIONS

BRAKING DISTANCE

In this chapter the implications of the findings reported in Chapter Two are explored. Particular attention is given in this section to the braking distance change on practice, the most significant effect of this research. An alternative to lengthening SSD available to the highway agency is to improve the surface skid characteristics and maintain them at a higher level so that f can be increased and braking distance reduced.

Experience (see Ref. (21)) has shown that the results of the numerical integration procedure used in computing braking distance can be approximated by evaluating deceleration at 0.707 times the initial velocity and then employing this value of deceleration as the average deceleration to be used in solving for braking distance (see Eq. 18.) Besides simplifying the calculation procedure by eliminating the need for numerical integration, this approximation makes the solution of the inverse problem of determining the skid number required to attain a selected braking distance rather straightforward using the following equations:

$$D = V_o^2/29.94(f_x BE + faero)CE \tag{30}$$

where f_x is evaluated at $0.707V_o$, faero is deceleration due to aerodynamic drag evaluated at $0.707V_o$,

and

$$CE = 0.444\ f_x BE + 0.267, \tag{31}$$

where f_x is evaluated at V_o.

For example, consider the following question: What value of SN_{40} is needed for a controlled braking distance approximately satisfying the AASHTO upper specification (1) of 414 ft from an initial velocity of 60 mph? For this example, assume a mean texture depth of 0.025 in. and a tire groove depth of 8/32 in. [Both of these depths correspond to the modes of histograms showing the distributions of sample populations (22)]. Using the appropriate equations from Table 7 and V = 42.42 mph, MD = 0.025 in., and x = 8 yields:

$$f_8 = 0.1921 + (SN_{40}/100)\ 1.2632 \tag{32}$$

$$faero = 0.0187$$

Substituting these values along with BE = 0.91, V_o = 60 mph, and D = 414 ft. into Eqs. 18 and 19, and solving for SN_{40}, yields:

$$CE = 0.4234(SN_{40}/100) + 0.1748$$

$$SN_{40} = 34.8$$

In summary, a skid number of approximately 35 as measured at 40 mph is needed to achieve the AASHTO policy upper braking distance of 414 ft while making a driver-controlled stop from 60 mph using tires worn to a typical level on a wet road with a typical texture depth.

The current AASHTO policy upper decelerations for speeds from 20 to 80 mph have been used in an analysis to explore their effect on SN_{40} values. A mean surface texture depth of 0.025 in. was assumed, and a modal tread groove depth of 8/32 in. was used. Most importantly, the stop was assumed to be one in which the vehicle was under control with an ability to turn in response to driver input and hence stay in the lane as well as negotiate a horizontal curve. The results are presented in Figure 17. It can be seen that SN_{40} values in the 32-37 range will make this performance possible.

The preceding example calculation illustrates that increasing skid number may be effective as an alternative to increasing SSD at existing sites if directional control is to be maintained while stopping. The hazards associated with hard braking on curved roadways, or where traffic demands may require traveling within a lane without spinning, may be reduced by increasing skid number to allow a margin for directional control.

The 15th percentile road has been found to have a skid number (SN_{40}) of only 28. This represents a very slippery surface, even for use on a proving grounds.

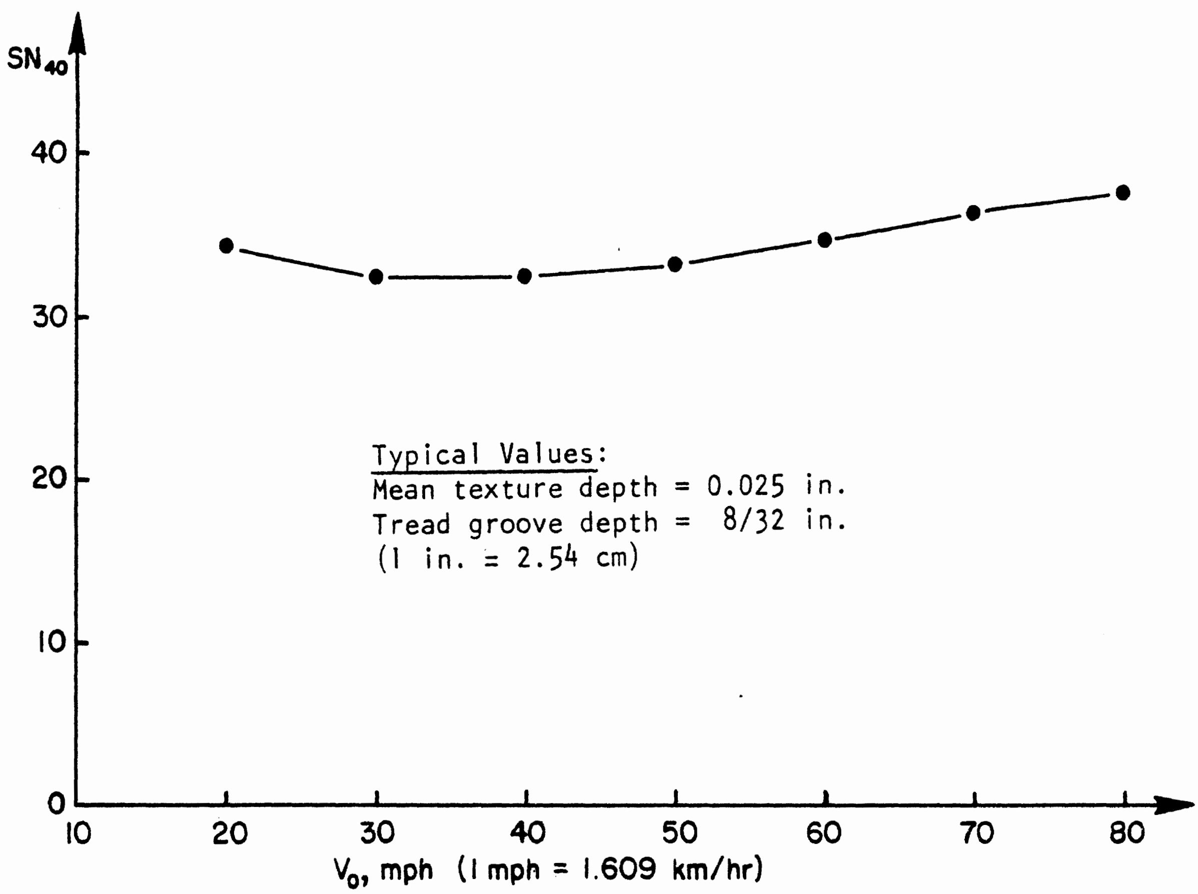


Figure 17. Skid numbers required to achieve AASHTO braking distances while stopping under control.

Experimentalists hesitate to perform tests above 40 mph on such surfaces, even in a proving grounds environment with vehicles having new tires. In the original driver control studies (23), the researchers had planned on doing tests at 60 mph but they decided that 60 mph was too dangerous and settled for 50-mph stops. Currently, expert drivers are performing tests from 60 mph on a surface comparable to the 15th percentile road using new passenger cars. Prior to this, experiments involving high speed stops have been performed almost exclusively on good, dry surfaces.

Locked wheel stops on surfaces as slippery as the 15 percent wet road are very dangerous. For example, procedures exist for measuring pavement friction using a vehicle with locked wheels. However, this approach has been found to be hazardous even though the tests were being done by experts making straight-line stops in controlled situations (24). To avoid these hazards, diagonal braking (one front and one rear wheel) is now employed in using a vehicle to measure tire-road friction during stops initiated at high speed. The other two nonbraked wheels provide the directional stability and control needed to safely operate the vehicle.

The idea of attempting emergency locked wheel stops from speeds of 50 mph or above on surfaces comparable to the 15th percentile road (or worse) with vehicles with heavily worn tires appears to be unreasonable. Drivers will not be able to control these vehicles without reducing pedal force so that their wheels will become unlocked. The findings of this study indicate that the stopping sight distance requirements recommended by AASHTO for high speed operation on poor, wet roads are not realistic if safe stops are to be accomplished. To alleviate this difficulty, the highway engineer may consider improving tire-road friction or increasing sight distance, but in either case, the prospects for unskilled drivers of vehicles with poor tires being able to make safe stops from 70 or 80 mph on wet roads appear to be very poor. Possibly, the notion of allowing for stops from 70 or 80 mph on wet roads is simply unreasonable, given the current characteristics of tires, vehicles, and drivers.

EYE HEIGHT

It is not believed that there will be significant changes in driver eye height for the practical horizon period. The results of the findings of this research are consistent with those of the last several years. Accordingly, a value of 40 in. (3.33 ft.) is recommended.

OBJECT HEIGHT

The 6-in. obstacle has been standard for a number of years. Recent changes in the composition of the passenger vehicle fleet have been in the direction of much smaller vehicles than were usual 10-20 years ago. Compared to large cars, these small cars are more likely to be damaged or deflected from their path by a collision with a 6-in. obstacle. They are also more likely to be snagged by passing over such an obstacle.

These problems arising from changes in the composition of the passenger vehicle fleet seem clear enough, but they cannot be quantified in a way that allows a direct comparison with the construction cost increases that will result from reducing obstacle height. However, it is the judgment of the investigators that the vehicle changes are significant enough to make a reduction in object height desirable. The height recommended is 4 in. This will reduce the impact problems mentioned above and allow all vehicles sufficient undercarriage clearance.

SPEEDS

The analysis of speeds indicated that there is still no significant difference in the operating speeds on the same road on dry or wet pavements. This supports earlier findings and implies that the speeds used in calculating required SSD for wet pavements should not be reduced but that the design speed value should continue to be used.

SAFETY AND SSD

The results of the safety study of limited SSD sites showed that there was a significant difference in the accident experience at locations where the sight distance was limited and the available SSD was below the 1965 AASHTO standard, and at locations that are very similar with respect to traffic characteristics, road design, roadside features, and abutting land use but have available SSD meeting the 1965 AASHTO standard. In this particular small sample of sites the difference was such that there were one-third fewer accidents on the matched control sites than there were on the limited available SSD sites.

Caution must be used in interpreting this result and in using the measure of one-third fewer accidents for safety predictions for available SSD improvements, because this figure comes from a small sample study.

The important finding is that when all other factors were controlled there was a significant difference in the accident experience between sites with inadequate SSD and those with adequate available SSD with the limited available SSD sites experiencing more accidents. Since the sites are matched, the causes of accidents, except for available SSD, are matched also and the implication is that there are significant safety impacts attributable to available SSD. Improving available SSD would therefore clearly have safety benefits, a conclusion consistent with the engineering judgment underlying concern with this phenomenon. More reliable quantification of this effect could be obtained from similar carefully controlled studies with larger samples.

RECOMMENDED POLICY

In Table 27 the recommended required SSD values from this research for speeds from 20 to 80 mph have been added to Table 12, the current AASHTO policy. The increase in SSD ranges from 10 percent at 40 mph, to 30 percent at 60 mph, and to 50 percent at 80 mph. The increase lies entirely with the braking distance component of SSD. Table 20 presents the recommended results for some very low speeds.

The effect of these SSD values on a standard design aid is shown in the dashed lines on the Design of Crest Curves upper policy of AASHTO in Figure 18 (6). This figure also incorporates the recommended eye and object height values of 3.33 ft and 0.33 ft. For a speed of 60 mph the required length of the curve is about the same as that needed for 75 mph in the AASHTO standard.

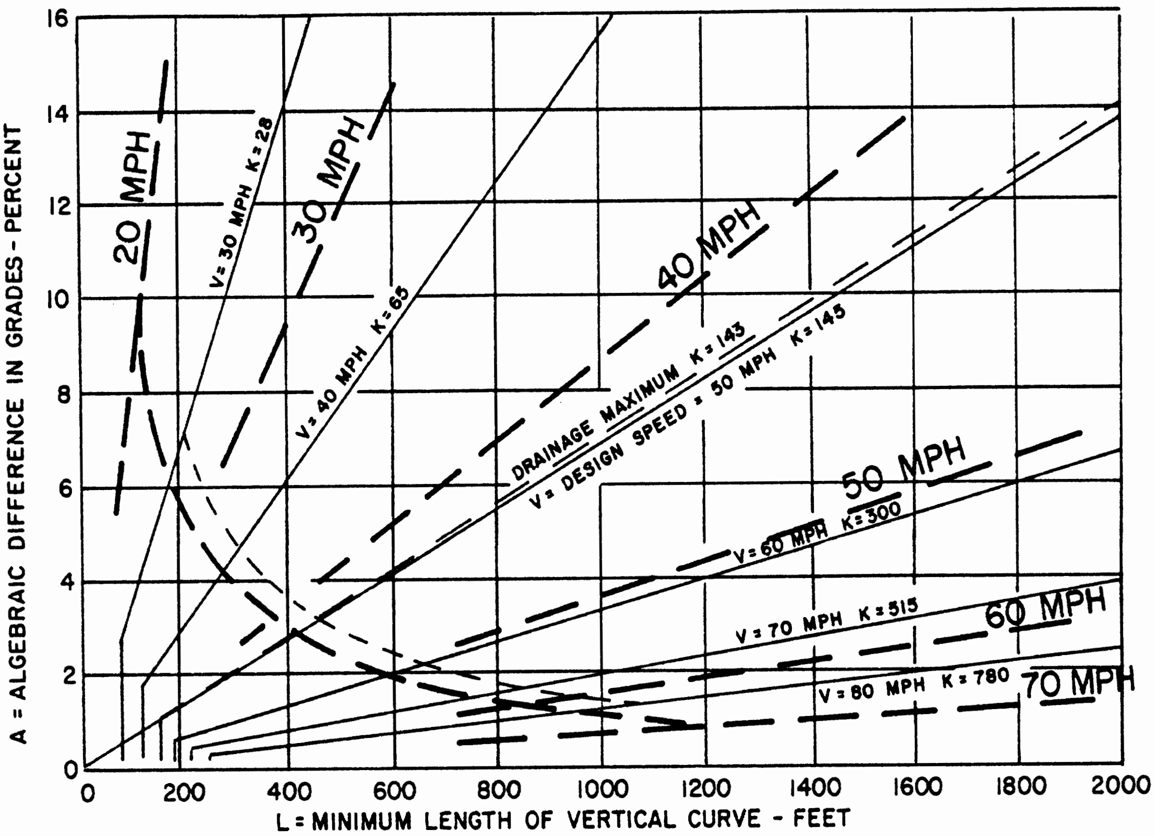
The findings of this research are surprising in the magnitude of the deviation of the recommendations from current policies. It has been shown that basic curve design elements are significantly impacted by these increases.

The key issue for highway engineering is seen to be whether the stopping is to be accomplished with the vehicle under driver control. If not, departures from the lane, spinning, and inability to negotiate a horizontal curve can be expected. It is the conclusion of this research that the existing AASHTO policy is based on braking distances that are not adequate for controlled stops from 50 mph and more. These high speed stops can only be made in a controlled manner if skid numbers of the surface are increased or better tires are required on such vehicles.

To a major extent the recommended changes in design policy are based more on the methods used for predicting braking performance than they are on changes in the characteristics of the vehicle fleet or the roadways. The

Table 27. Current policy and recommended stopping sight distances (wet pavements).

Design Speed (mph)	Comp. Speed (mph)	AASHTO POLICY SSD				RECOMMENDED SSD			
		RD (ft)	BD (ft)	Computed SSD (ft)	Design SSD (ft)	RD (ft)	BD (ft)	Computed SSD (ft)	Design SSD (ft)
20	20	73	33	106	120	73	33	103	105
30	28	103	75	178	200				
	30	110	86	196	200	110	77	187	190
40	36	132	135	267	275				
	40	147	167	314	325	147	216	363	360
50	44	161	215	276	375				
	50	183	278	461	475	183	380	563	560
60	52	191	311	502	525				
	60	220	414	634	650	220	619	834	850
70	58	213	400	613	625				
	70	257	483	840	850	257	943	1,200	1,200
80	64	235	506	741	750				
	80	293	790	1,083	1,100	293	1,365	1,658	1,650



DESIGN CONTROLS FOR CREST VERTICAL CURVES
WITH DESIRABLE STOPPING SIGHT DISTANCES

SOURCE: American Association of State Highway Officials, A Policy on Design Standards for Stopping Sight Distance, AASHTO, Washington, D.C., 1971.

Figure 18. Design controls for crest vertical curves and recommended values.

current design policy is based on a calculation in which it is assumed that the locked wheel friction between tire and road is equivalent to the average deceleration that will be achieved by drivers in emergency situations. The approach proposed for this study encompasses a more detailed look at the braking process. Tire-road friction is examined again. Then the influences of tire properties and wear are considered. Next, the efficiency of the braking system is evaluated. And finally, the control efficiency of the driver is included in the analysis. At the start of developing this detailed analysis it was not known whether the analysis would yield braking distances that are shorter or longer than those predicted by the current design formula.

The study of roadway skid numbers and tire characteristics indicates that the friction factors used in the current design policy are reasonably consistent with the findings of this research concerning the frictional potential of the tire-road interface. As shown by the braking distance curves labeled "AASHTO" and "2/32, lockup" in Figure 6, the braking distances recommended by AASHTO compare favorably with distances calculated for a passenger car equipped with tires worn to 2/32nds of an inch making locked wheel stops on a road with a skid number and skid number gradient at the 15th percentile level. Per these observations, the findings confirm that current AASHTO policy corresponds to the locked wheel braking capabilities of fully worn tires operating on a poor, wet road.

However, locked wheel performance is not necessarily indicative of the performance capabilities of the driver-vehicle-highway system. Car tires are capable of generating considerably higher peak braking forces than those achievable with locked wheels. In addition, current braking systems are very efficient, meaning that current passenger cars are capable of attaining decelerations corresponding to approximately 90 percent of the peak braking force available from the tires when operating on a poor, wet road. So far, one might anticipate that cars would be able to brake to a stop in distances shorter than their so called "locked wheel braking distances."

The problem is that the driver is not a perfect controller as needed to achieve the maximum braking performance of the highway-tire-vehicle combination. The data gathered and examined in this study show that modulating the brake pedal is a difficult task leading to braking distances considerably longer that the locked-wheel braking distances (see Fig. 6).

Given difficulties in modulating the brake pedal, what will cause drivers to avoid locking their wheels in an emergency stop? Drivers will modulate their braking to avoid locking wheels when their direction of travel is unsatisfactory or their vehicle is spinning around. The primary action that prevents drivers from being able to select their direction of travel and/or avoid spinning is locking wheels.

Once a vehicle's wheels are locked, the vehicle tends to go in a straight line except that it will "slide" downhill. Also, if the vehicle is spinning it will tend to keep spinning. With regard to traveling on roadways, it can be hazardous to operate with locked wheels in the following circumstances:

1. If the road is curved (horizontally), the locked-wheel vehicle will proceed off the road tangentially to the curve.

2. The crown or superelevation of the road will cause the locked-wheel vehicle to slide toward the downhill side of the road in a manner that the driver cannot correct by steering (for example, on a straight, crowned road the vehicle will tend to slide off the road).

3. If the road is heavily traveled with vehicles occupying adjacent lanes, the driver of the locked-wheel vehicle will not be able to correct for wind disturbances, roadway side slope, bumps, or other disturbances that cause the vehicle to depart from its lane. Furthermore, if the vehicle's rear wheels lock before its front wheels, the vehicle will tend to spin around (that is, be directionally unstable) and, once the front wheels lock, the driver will not be able to steer out of the spin and prevent the vehicle from intruding into adjacent lanes.

All of these problems with the locked-wheel vehicle are unimportant if the vehicle gets stopped in time before hitting other vehicles or roadside obstacles. In contrast, at high speeds, braking times and distances are such that an uncontrolled stop is not a reasonable course of action to expect a driver to pursue. Since the braking times are long for a high speed stop (for example, stops from 60 to 80 mph with average deceleration capabilities of from 0.2 to 0.3g take from 9 to 18 sec to complete), the driver has time to modulate the brake to regain steering control and complete a controlled stop. Unfortunately, the braking distances required for a controlled stop are considerably longer than locked-wheel braking distances (see Fig. 6). Nevertheless, safety considerations indicate that: (1) the locked-wheel stopping is not desirable, and (2) it should not be portrayed as an appropriate course of action.

Another factor, which makes locking wheels particularly hazardous, is the relatively low value of pavement skid resistance available at high speeds. The value of the friction factor, f, used in the design formula is misleading in this regard. The friction factor represents the average deceleration over the entire stop, not the value of tire-road friction available at the initial velocity at which the brakes are applied. For example, on the 15th percentile road the skid number at 40 mph is 28, but at 60 mph it is 22.3 and at 80 mph it is 17.7. For car tires worn to 2/32nds of an in., these skid numbers lead to locked-wheel friction levels of 0.20 at 60 mph and 0.14 at 80 mph, even though the average decelerations (corresponding to the "friction factors" used in the design formula) are 0.30 at 60 mph and 0.26 at 80 mph. The reason why the friction level at high speed is so much lower than the friction factor is that the friction factor is the average of the friction level over the entire stop, which starts at a friction level determined by the initial velocity and has a friction level that increases as the velocity decreases during the stop. The point of this discussion is that, even though the average friction capability at 80 mph is 0.26, the friction capability at the beginning of the stop is only 0.14, a level that approaches that of ice. If the driver of a vehicle with worn tires were to lock wheels at 80 mph on a poor, wet road, that would be like locking wheels on a snowy road, except that the driver would be traveling at 70 to 75 mph (approximately 100 ft/sec) instead of at 25 to 20 mph as would be expected for slippery conditions. Traveling sideways or heading off the road at 100 ft/sec is dramatically different from sliding at 20 to 30 ft/sec. If the vehicle is traveling at 80 mph, it is not going to stop before it has left the desired path by a substantial distance.

One might consider altering the braking distance criteria at high speeds in order to shorten the lengths of the vertical curves required on high speed roads. An approach that has been suggested is to accept a reduction in speed until it is possible to safely steer around the obstacle. However, even if this safe steering speed is set as high as 40 mph, that selection will only reduce the braking distance (and consequently the stopping sight distance) by approximately 170 ft (see Table 15). (For speeds above 40 mph, the braking distance equals the braking distance from 40 mph plus the braking distance from the initial velocity to 40 mph.) Although 170 ft may seem like a large reduction, it does not amount to much for a stop from 80 mph, which has a stopping sight distance of approximately 1,650 ft (see Table 19). In terms of the amount of excavation that needs to be done for a vertical curve on an 80-mph road, the change in cost associated with this 170-ft. change in SSD is only a fraction of the cost associated with the difference between the current AASHTO policy and the recommended SSD for a controlled stop (a 550-ft difference).

So far in this discussion, safety considerations have governed the selection of the recommended SSD. To this point, the costs associated with providing longer SSDs have not influenced the recommended distances.

In order to assess the relative costs associated with lengthening SSD's, a simple analysis has been performed to provide a means for making first order estimates of the amount of earth that needs to be moved to build a vertical curve. Assume for these purposes that the original hilltop may be roughly approximated by two symmetric grades (up and

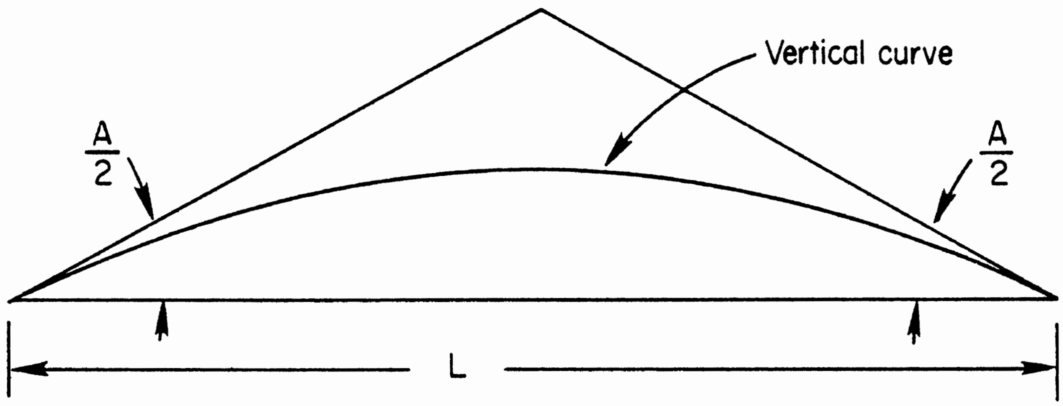


Figure 19. Sketch of a hilltop.

down) of A/2, the total difference in grades being A. Let L, the length of the vertical curve, be the base of an isosceles triangle with the equal sides making equal angles of A/2 with the base. See Figure 19, which also shows the vertical curve representing the roadway. The original cross sectional area of the hilltop equals the area of the triangle with base L. This area is equal to $L^2/4$ tan (A/2). The area between the vertical curve and the base of the triangle is equal to $L^2/4$ tan (A/2) - $L^2/12$ tan (A/2). The amount of earth to be moved (call it "E") is estimated to be approximately proportional to the difference between these two areas, that is, E E $L^2/12$ tan (A/2).

For the same hilltop, consider two stopping sight distances, S_1 and S_2, leading to two vertical curve lengths, L_1 and L_2. For this situation, the simplified analysis indicates that

$$E_2/E_1 = (L_2/L_1)^2 \tag{33}$$

where:

E_1 = the amount of earth to be moved for a vertical curve of length L_1; and

E_2 = the amount of earth to be moved for a vertical curve of length L_2. (Note that this ratio is independent of A.)

The relationship between the vertical curve length, L, and the stopping sight distance, S (see Appendix E) may be expressed as follows:

$$L = AS^2[(2h_e)^{1/2} + (2h_o)^{1/2}]^{-2} \text{ for } S<L \text{ (long curves)} \quad (34)$$

where A represents the differences in grades, h_e is the height of the eye, and h_o is the height of the object. Hence, when comparing the influences of two SSD's, S_1 and S_2, the following relationship holds:

$$(L_2/L_1) = (S_2/S_1)^2 \quad (35)$$

where L_2 is the vertical curve length corresponding to S_2 and L_1 is the vertical curve length corresponding to S. (Note that this ratio is independent of A, h_e, or h_o.)

To compare the influences of two choices of SSD on the amount of earth to be moved, Eqs. (33) and (35) may be combined to yield:

$$(E_2/E_1) = (S_2/S_1)^4 \quad (36)$$

where:

E_1 = the earth to be moved if S_1 is chosen; and

E_2 = the earth to be moved if S_2 is chosen.

Note that Eq. (36) provides a means for assessing the relative cost associated with increasing SSD. For example, at 50 mph (see Table 27) the upper SSD recommended by AASHTO is 475 ft, while the distance for a controlled stop is 560 ft, yielding a ratio of $S_2/S_1 = 1.18$ and hence $E_2/E_1 = 1.93$. In this 50-mph example, the cost of moving earth has been estimated roughly to increase by approximately 90 percent, almost double. The situation at 80 mph is much more extreme due to the 4th power relationship:

$$S_2 = 1{,}650 \text{ ft}, \; S_1 = 1{,}100$$
$$E_1/E_1 = (S_2/S_1)^4 = 5.06$$

The cost of moving enough earth to provide a 1,650-ft SSD is approximately five times the cost of moving earth to provide an 1,100-ft SSD.

Assuming that the excavation costs to build a vertical curve according to the AASHTO requirements at a particular site are significant, the cost to provide for controlled stops from high speeds is very significant (that is, 2 to 5 times the costs associated with the AASHTO design).

On the basis of the foregoing discussion, a dilemma exists. On the one hand, locked-wheel stops are hazardous, and on the other hand, providing for controlled stops can be very costly. Is there a reasonable compromise? Regardless of the underlying assumptions, roads designed to current standards do not have accidents at such a rate that they are being closed as unsafe. Possibly, the SSD's recommended in the current design policy provide an acceptable level of safety with respect to popular opinion. Nevertheless, one would like to improve highway safety if the funds are available.

To arrive at a compromise position, it is reasonable to attack the requirements applicable to high initial velocities because those are the cases in which braking distances have major influences on stopping sight distances.

One possibility is to simply eliminate design requirements based on speeds above 60 mph (or some other chosen cutoff point). This would eliminate the undesirable notion that highway engineers believe that vehicles with poor tires can safely make locked-wheel stops on poor, wet roads. For speeds above 60 mph, it is assumed that (1) the drivers will be more alert and have shorter perception-reaction times, and (2) they will have better tires. For high speed locations that are likely to have accident-causing obstacles in the road, it is expected that pavement skid numbers will be maintained well above the 15th percentile level, possibly at levels consistent with making controlled stops within the current AASHTO braking distances. Finally, it is anticipated that new and rehabilitated roads will have higher skid numbers and that vehicle designers will develop braking systems that are easier to modulate or are automatically modulated.

For speeds less than or equal to the cutoff level (possibly 60 mph), the amounts of increases in SSD's might depend on the costs involved. To illustrate this approach, assume that the costs associated with excavation could reasonably be doubled at certain sites. (Clearly, this assumption may be unreasonable. It is only a hypothetical example.) If the excavation costs can be doubled, the 4th power Eq. (36) indicates that the SSD's could be increased by about 20 percent. Since the SSD's recommended for controlled stops are less than 20 percent greater than the current AASHTO SSD's for speeds less than or equal to 50 mph (see Table 27), the recommended controlled-stop distances could be accepted as is. At 60 mph, the SSD could be increased by 20 percent, thereby increasing from the current AASHTO value of 650 ft to 780 ft.

As was true in the 1971 AASHTO policy change the determination of the amount that SSD should be lengthened must ultimately be based on the judgment and informed opinions of the groups responsible for establishing design policy (6).

The Model. It was intended that a probability based model would be developed so that the joint effect of driver reaction and braking distance variability could be used to develop the choice of specific long SSD requirements. See Appendix E for a treatment of the mathematics of such a model. Unfortunately, the available information on tire-road friction variability did not make such a model more attractive than the model developed in this research.

It is noted that the recommended values of SSD can be compared with those associated with decision sight distance, DSD (25). DSD has been defined as the process of hazard avoidance in time intervals necessary to: (1) detect a hazard, (2) recognize the hazard potential, (3) decide on a response, (4) respond, and (5) maneuver appropriately. Field validation has refined and confirmed that previously developed DSD values are appropriate for use in highway design. In one application, the DSD values recommended were substantially larger than equivalent SSD values. Additional comparisons of SSD and DSD for nonstopping situations may provide additional support for adoption of the SSD recommendations of this research.

CHAPTER FOUR

CONCLUSIONS AND SUGGESTED RESEARCH

CONCLUSIONS

The research has explored the major inputs involved in stopping sight distance in highway design. It found that only the driver-perception response time is appropriately represented by present practice. The research concluded that driver eye height and possibly object height should be lowered somewhat. The principal finding was that vehicle braking distances that have been used in design are much too short for an approach based on accommodating the poorest performing elements of the system at high speeds.

It is concluded that more attention could be given to increasing the frictional capabilities of road surfaces by highway agencies.

Policies of selecting design speeds should be reviewed to determine the effectiveness of continuing to design for the highest values. These results may cause highway administrators to adopt a changed policy with respect to design speeds. The implications of reducing these speeds are of great importance, since it primarily is the speeds of 70 and 80 mph for which the changes are the greatest and for which the most highway retrofitting would be required.

SUGGESTED RESEARCH

Additional research needs were found in several of the areas of concern in this research.

Tire Road Interface

The researchers were unable to develop a probability model that would make reliability studies possible. The primary reason for this lies in the absence of information on the variation associated with the tire-road interface. It is recommended that research be encouraged that would provide a reasonable estimate of the variance in deceleration on wet surfaces.

Driver Control Efficiency

Car-Driver Experiments. The experimental evidence used to quantify control efficiency (as defined in this study) was gathered over 10 years ago (23). The test vehicle was adjusted to have a braking efficiency of nearly 100 percent, a level of efficiency that currently manufactured vehicles are approaching. Consequently, the vehicle used in those experiments was representative of the braking performance of current vehicles. Nevertheless, driver-vehicle system tests, using current vehicles to make rapid, controlled stops, are needed to verify the previous findings.

During the last 10 years much of the brake system testing in the United States has been devoted to dry surfaces (possibly because those are the conditions employed in government standards). The development of test methods suitable for use on wet surfaces is much more difficult than that required for testing on dry surfaces. However, the wet road condition has an important safety relevance. Even though appropriate test methods will require careful control and definition of surface conditions, experimental evidence needs to be gathered to quantify driver control efficiency on poor, wet surfaces for vehicles traveling at highway speeds.

Truck-Driver Experiments. The tests performed in this study were only a fraction of the work needed to be done to understand truck braking. Further research should address the influence of friction level on the control efficiency of truck drivers. Ultimately, the matter of brake proportioning of heavy trucks should be studied to determine proportioning arrangements suitable for stopping quickly while maintaining directional control.

Combined Tire/Road Studies

During the course of this study, examination of the literature indicated a tendency for researchers interested in tires to make tests on several surfaces, but then to "average-out" the surface effects. Similarly, pavement experts tend to make studies with different tires, only to average-out the tire effects. Clearly both the grooves in the tires and the macrotexture of the pavement interact in "managing" water to allow friction to develop between the tread materials and the microtexture of the road. The highway engineer wants to know how various types of roads will perform when vehicles with various types of tires in various states of wear operate on these roads. The interaction of tires and roads needs to be explored as a total system to meet the needs of highway design and evaluation.

Research into the area of tire-road friction is, has been, and probably will be quite intense. It is an area in which an expanding number of important effects are being uncovered. The results of research are not tending to greatly simplify the theory. One way to deal with this complex array of studies and results is to support research that applies to specific areas of particular interest to the highway community.

Based on observations made during this study, research investigations emphasizing water depths typical of those occurring on highways during rainstorms are not receiving as much attention as seems to be warranted. Hydroplaning that occurs at deep water depths is much more spectacular than typical wet road phenomena. The investigation of wet traction at typical water depths requires detailed knowledge of both tires and pavements. Nevertheless, tire braking performance on roads wetted to just over the tops of the asperities appears to be the area that the highway community should stress.

The difference between truck and car tire braking performance is a major factor in predicting longer braking distances for trucks. Clearly the braking requirements for trucks may or may not be the control in a design situation depending on the ratio of car-to-truck tire friction. Data comparing car-tire braking traction with truck-tire traction are scarce. Side-by-side testing on the same surface at the same time has not been done except possibly in a few cases. Further studies in which car and truck tires are tested simultaneously under comparable conditions could provide more definitive data and better evidence for use in comparing truck-braking capabilities with those of passenger cars.

Driver Vehicle Studies at High Speeds

Currently, DOT is studying wet surface braking from an initial velocity of 60 mph. Although those tests are driver controlled, they are straight-line tests using expert drivers. The results of a study of 1983 cars will be available in March 1984. If experience gained in those studies indicates that it is practical to try less than ideal drivers and vehicles, further research could be directed at less than ideal conditions with a slight directional control challenge, for example, a path that curves in accordance with a cosine wave of 400-ft period and a peak-to-peak amplitude of 3 ft. Vehicle dynamics studies conducted at a proving ground are advocated because "emergency" events do not happen often enough to be observed in practice.

The study of high speed braking of heavy trucks is even more difficult than that pertaining to cars. Trucks are more susceptible to rolling over and articulated vehicles may jackknife, causing a severe collision within the vehicle itself. After experience has been gained in testing cars at high speed on slippery surfaces, truck stopping tests might be attempted. However, analytical results based on measured performance of tires and braking system components should provide predictions satisfactory for current use.

Accident Studies

An analysis of accident counts at a set of sites matched by traffic volumes and characteristics, road design, roadside features and abutting land use, but differing in available SSD, where one site was below 1965 AASHTO SSD standards and the other of the pair met the 1965 AASHTO SSD standard was carried out. The analysis of the accident counts showed that there was a significant difference in the accident counts between the limited site distance sites and their matched controls.

While this study demonstrated that it is possible to carry out a matched-pair study of the safety effects of available SSD, the sample was too small to obtain a reliable quantification of the safety effect. It would be useful to expand on this study and to conduct it using a large sample of matched sites from a larger range of areas in the United States. It would be also be desirable to further isolate the effect of SSD from the possible confounding effects of vertical grades by including a measure of the grades at the sample locations in the analysis.

Computer Programs

Several programs have been prepared for the research which would be of value to geometric designers if coded for microcomputers. It is suggested that these or other programs be made available through highway computer user groups after some modifications have been made.

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APPENDIX A

DRIVER PERCEPTION-RESPONSE TIME

DRIVER PERCEPTION-RESPONSE TIME

INTRODUCTION

The purpose of this study was to develop data on driver perception-response time as a function of several variables and under realistic conditions.

"Realistic" estimates of perception-response time can only be based on a scenario in which the subject is led to believe the purpose of the test is something other than it really is, so that when the roadway obstacle is encountered it will truly be a surprise. This also means that only one such data point can be collected per subject.

Trying to collect data adequate to approximate performance at various population percentiles requires a reasonably large sample of subjects. When each subject takes considerable time and yields but one data point, it is clearly impractical to use the surprise scenario to consider multiple variables. Thus, two studies were run. The first was the "surprise" study, limited to a single roadway obstacle and two age groups. The second study was a parametric investigation, run under alerted conditions.

"Surprise" Study

Independent Variables

Obstacle. The combination of variables selected for this phase of the study was designed to represent a "medium" condition. The obstacle was a piece of foam rubber 6 in. (15.24 cm) high and 3 ft (0.91 m) wide as viewed by the subject. It was light yellow in color. The color of the obstacle and the road surface combined to create a relatively low-contrast condition. Figure A-1 is a photograph of the obstacle.

Subject Age. Two age groups were used in this study. The "young" subjects ranged from 18 to 40 years of age. Useful data were obtained from 49 young subjects. The "older" subjects ranged from 60 to 84 years of age. Useful data were obtained from 15 older subjects.

Dependent Variables

The primary dependent variable was the time from first visibility of the obstacle until a response was initiated by the subject releasing the accelerator. Time from accelerator release until brake contact was also recorded. These two measures were then summed to obtain a total response time.

Equipment

The test vehicle was a 1980 Ford LTD station wagon. It was equipped with a distance-measuring system. This system used a magnetic pick up that responded to studs welded to the wheel rim. There were four such studs, equally spaced. This yielded a distance of 20.7 in. (0.525 m) per count.

The experimenter used the control box shown in Figure A-2 to collect data. The dark rectangular objects on the box are digital distance counters and timers (1/100th sec). The box was used to collect data as explained below and diagrammed in Figure A-3.

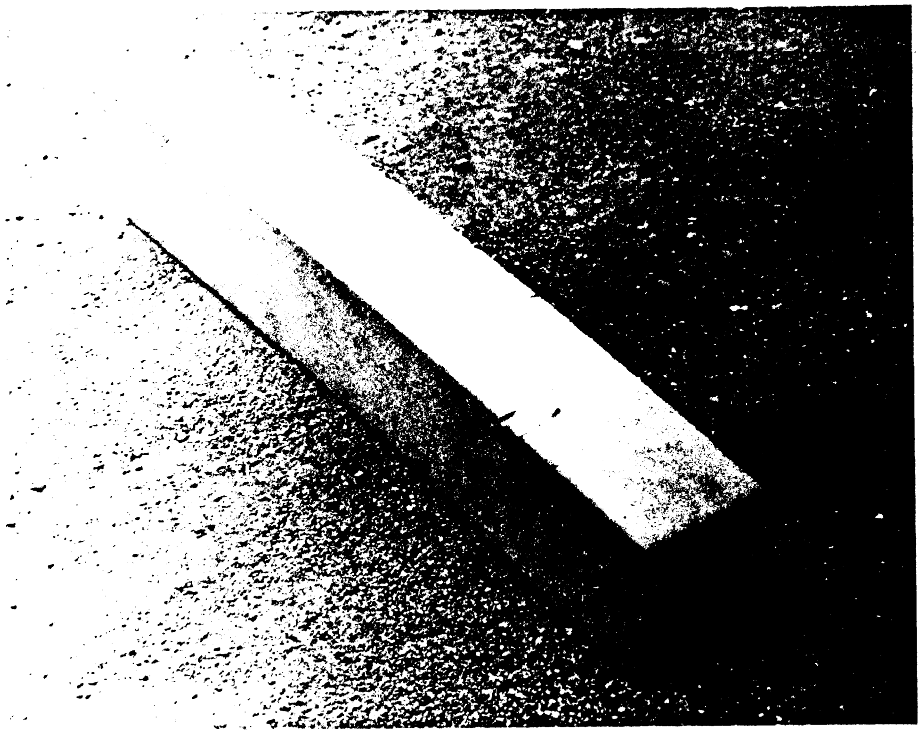


Figure A-1. Photograph of obstacle used in surprise study.

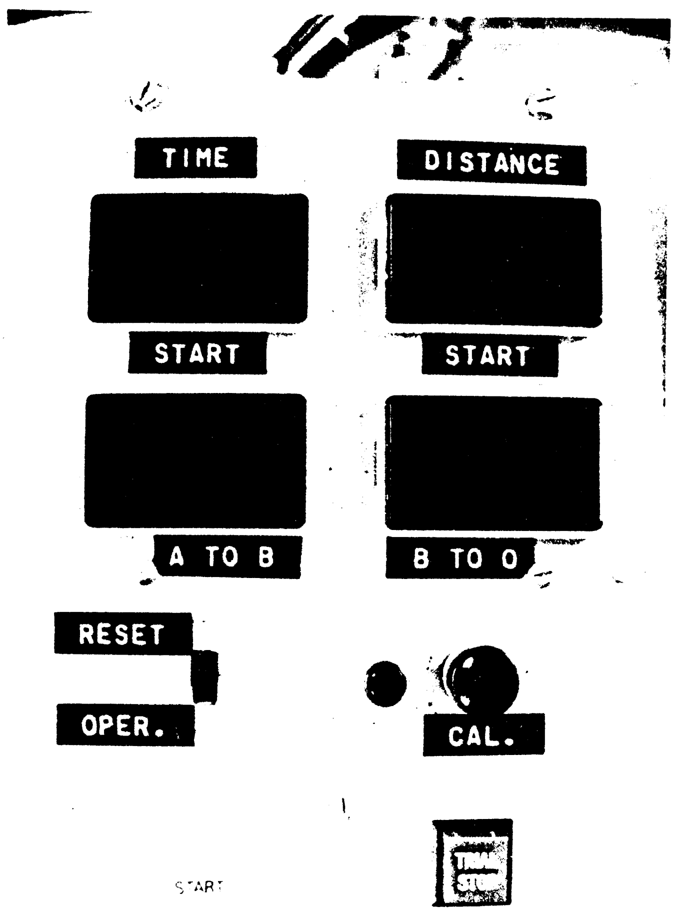


Figure A-2. Photograph of experimenter's control box.

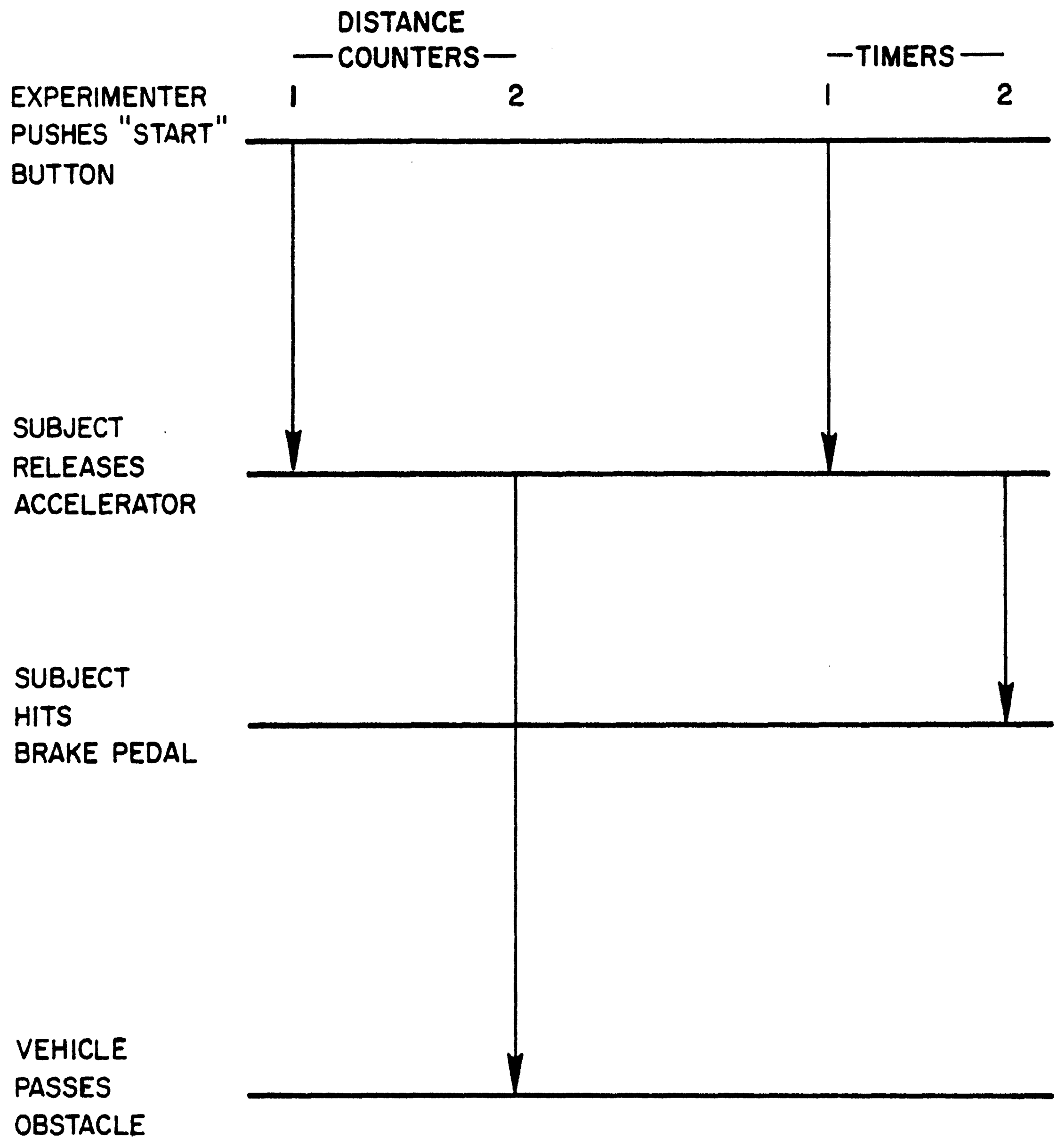


Figure A-3. Operational flow chart for timers and counters used in taking data.

A short distance before the subject could detect the obstacle the experimenter moved the switch from "reset" to "operate" and pressed the "start" button. This action started a distance counter and a timer. These continued to run until the subject responded to the obstacle by releasing the accelerator, which action started the other distance counter and timer. When the subject hit the brake pedal the second timer was stopped. The experimenter pressed the "trial stop" button as the front bumper of the car reached the obstacle, shutting off the second distance counter.

The button marked "CAL" was used in calibrating the system. Pressing this button started the distance counters. The route for each subject passed a pair of telephone poles that were a known distance (1,003 ft) apart. The experimenter measured this interval each time using the distance counters, to be sure the system was working properly.

Test site

The following criteria were used in selecting a test site:

1. Significant vertical curvature relative to the speed to provide a challenging task to the driver.

2. Light traffic to minimize interference and risk of accident.

3. Reasonably close to the Institute, to minimize driving times.

4. "Open" surroundings (no parking, pedestrians or nearby houses) to minimize distractions or interference from curious passerbys, etc.

5. Somewhere for the person who placed and removed the obstacle to hide.

6. Reasonably good quality pavement.

Finding a site that met these criteria proved difficult. A number of locations were considered and rejected, generally because the traffic volumes were too high. The study was set up and started at one site, and data were collected on 10 subjects. However, it was concluded that the sight distances were too long (average of about 5.5 sec at the 35-mph (56 km/h) speed limit). This judgment was based on the fact that the response times were much longer than expected. Many subjects did not touch the brake, they simply steered around the obstacle. So, the search for a suitable site was renewed.

The site finally selected was on a two-lane tangent road in a rural area. The total width of the road in the test area averaged 22 ft (6.7 m). There were shallow ditches on each side, and a heavy growth of trees and shrubs close to the edge of the road. Two views of the site are shown in Figure A-4. The top photograph shows the rise just before the obstacle became visible. The bottom photograph is taken from the crest, and shows the obstacle in place. (The parked car in the lower photograph was not present during the test.)

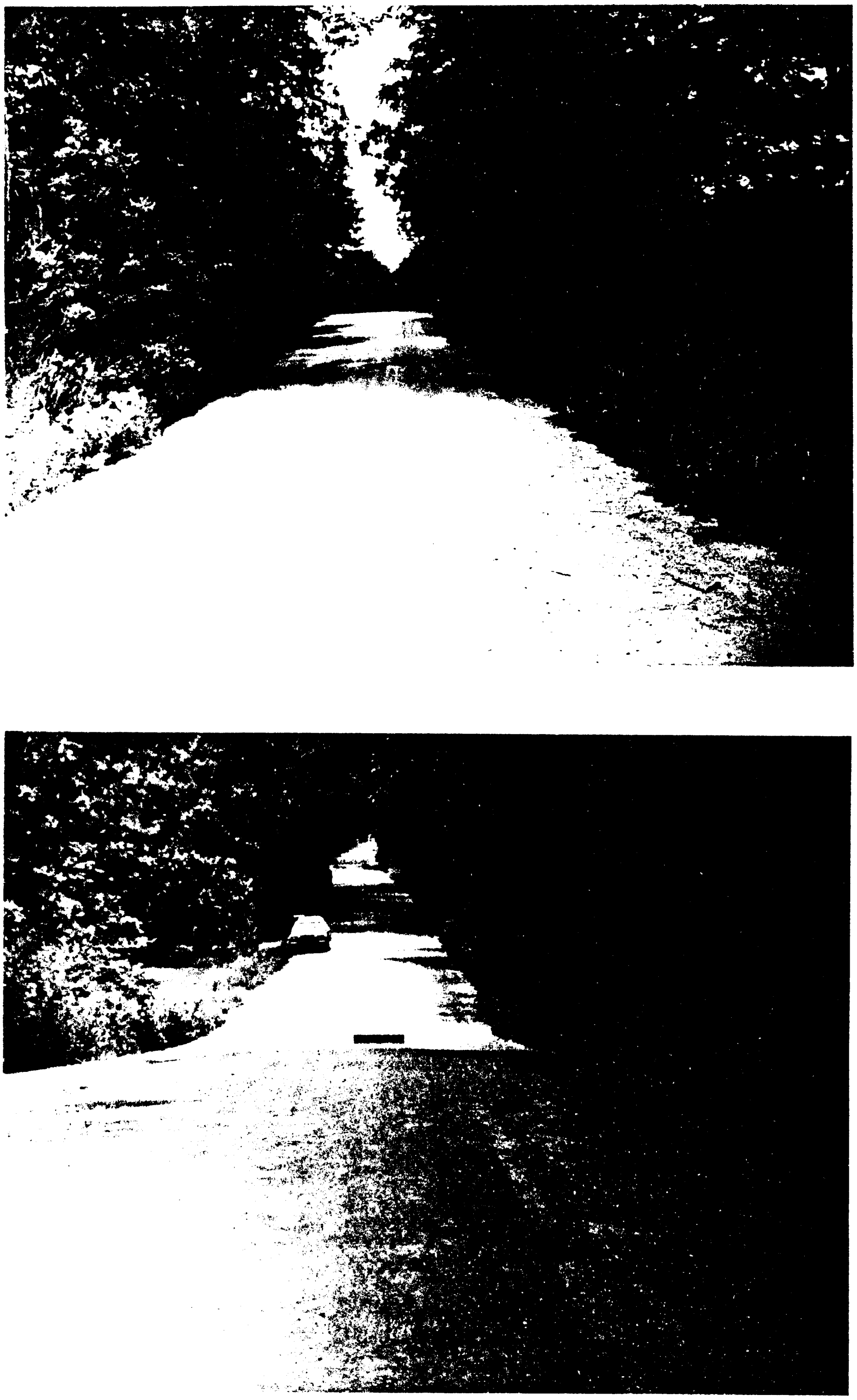


Figure A-4. Two photographs of surprise test site (top: approach to crest; bottom: at crest with the obstacle in place).

A vertical profile of the site is provided in Figure A-5. In this figure, the subjects approached from the left side (G_1 = + 5.0 percent, G_2 = -6.2 percent, L = approximately 100 ft (31 m)), and the obstacle was placed to the right of the crest. Four obstacle positions are noted as points 1 through 4 under "Comments." Position 2 was used in the first, "surprise" trial. All four were used in the alerted trials that followed.

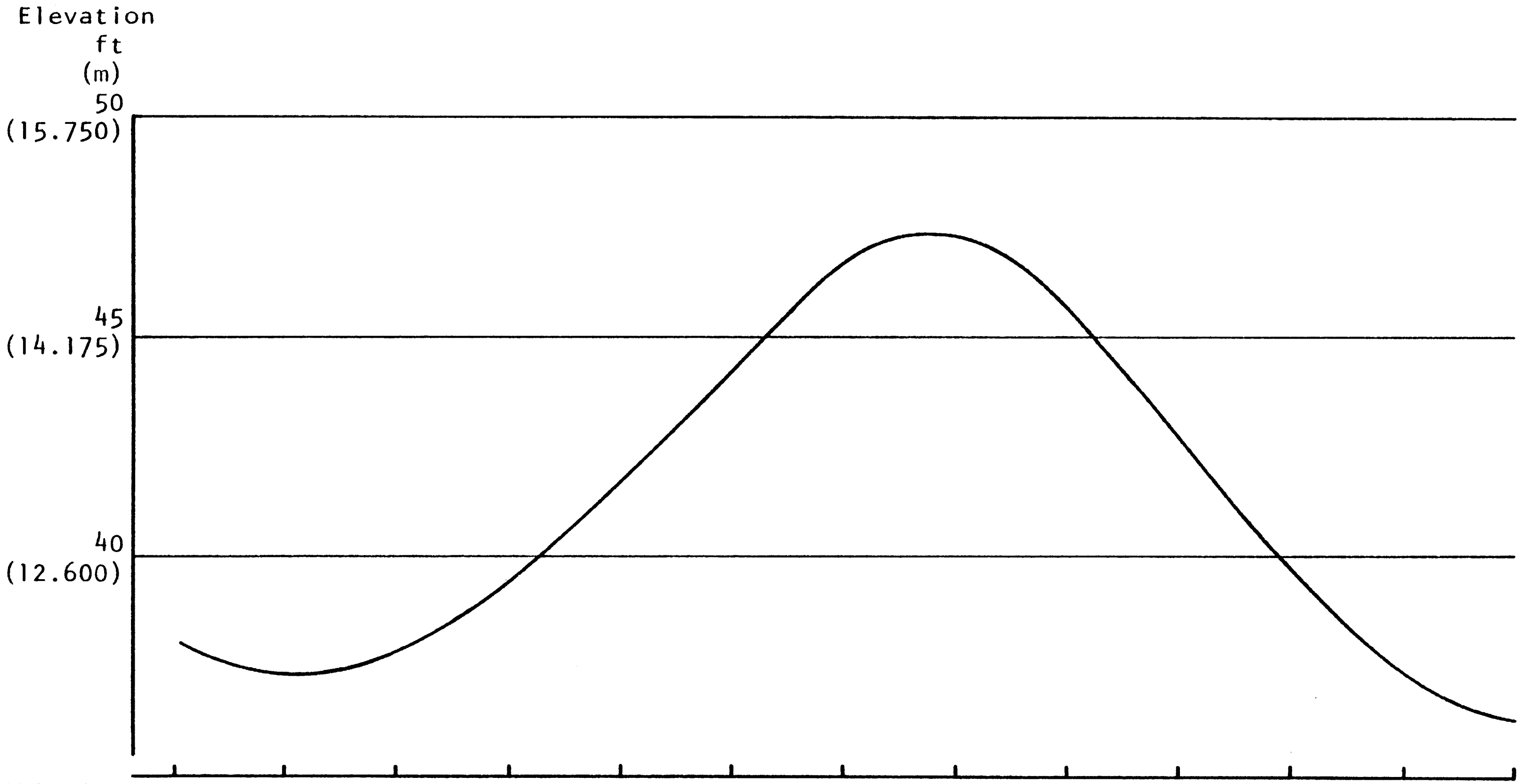
Procedure

Two experimenters were required: one rode with the subject; the other stayed at the test site to place and remove the obstacle.

Each subject reported to the Institute at an assigned time, signed a consent form, and was escorted to the test car. The subject was seated in the driver's position. The experimenter rode in the rear seat on the right side.

Before starting out, the instructions were read to the subject. Basically, they were told this was a study of driving performance, but first they were to drive the car a few miles to become accustomed to its characteristics.

After questions had been answered the drive started. The experimenter occasionally told the subject to turn, move into a particular lane, and drive at a certain speed. About half way to the test site the experimenter made a "radio check" ("mobile one to base, do you read?"), the actual purpose of which was to alert the person at the test site that the subject was approaching.



STATION													
ft	6+50	7+00	7+50	8+00	8+50	9+00	9+50	10+00	10+50	11+00	11+50	12+00	12+50
m	198.25	213.50	228.75	244.00	259.25	274.50	289.75	305.00	320.25	335.50	350.75	366.75	381.25
ELEV													
ft	38.17	37.31	37.94	39.58	41.83	44.63	47.10	47.69	45.98	42.76	39.75	37.51	36.26
m	11.642	11.380	11.572	12.072	12.758	13.612	14.366	14.546	14.024	13.042	12.124	11.441	11.059

Comments

	Bottom of Sag	Crest	Pt. 1	Pt. 2	Pt. 3	Pt. 4
Station	7+00	10+00	10+78	11+20	11+54	11+93
Elevation ft	37.31	47.69	44.19	41.50	39.49	37.76

Figure A-5. Profile of crest vertical curve used in surprise study.

After driving about 4 mi (6.4 km) the subject was on the approach to the obstacle. The experimenter put the data box into "operate" mode and pressed the "start" button. The subject released the accelerator, hit the brake, and (generally) steered around the obstacle. The experimenter pressed the "trial end" button as the front bumper of the car passed the obstacle.

With the surprise phase completed, the subject was asked to turn the car around and park. The experimenter explained the true purpose of the study and outlined the alerted data to be taken next.

The next step was to measure the distance at which the subject could just see the top of the obstacle over the hill crest. To do this the subject maneuvered the car until the desired point was reached, and stopped. The experimenter pressed the "calibrate" button on the control box and had the subject drive forward until the front bumper reached the obstacle.

Next, five trials were run under "alerted" conditions. The subjects were asked to drive up the road at about the same speed as before, releasing the accelerator and tapping the brake pedal as soon as they saw the obstacle. The obstacle position was changed on each trial.

This test proved difficult for some subjects. The natural response, knowing the obstacle was there, was to release the accelerator in anticipation of seeing it. Some subjects could not completely overcome this tendency. As a result, not all subjects produced five trials of useful data on this task.

At the completion of the alerted trials the subject was asked to park once again. A 4-in. (10.2 cm) diameter auxiliary stop-turn lamp was attached to the front of the hood, directly in front of the subject. The subject was instructed to drive back to the Institute in a normal manner, and, when the light came on, to release the accelerator and tap the brake as quickly as possible. After two practice trials, five additional trials were taken on this task. These will be referred to as "brake" trials.

This completed the test. On arrival at the Institute the subjects were paid and released.

Data Analysis

Figure A-6 is a reproduction of a data sheet from the study. This particular subject was run July 6, 1983, was a female, young, and braked with her right foot.

The first row of data, labeled "SUP," is from the surprise trial. The first two columns, headed "TIME" and "CNTR," record the output of the timer and distance counter started by the experimenter just before the subject can detect the obstacle. These were stopped by the subject releasing the accelerator. In this case the timer ran for 1.25 sec, and the counter accumulated to 28. Twenty-eight counts translate to 14.7 m, a speed of 11.76 m/sec.

Columns 5 and 6, labeled "A to B," record the time it took the subject to move his/her foot from the accelerator

SUBJECT #: U NAME:

DATE: 7-6-83 M-F Y-O R-L

	START				A TO B		A TO O			
	TIME	CNTR	METERS	M/SEC	TIME	METERS	CNTR	METERS	DIFF METERS	R.T.
SUP	1.25	28	14.7	11.76	.48	5.6	74	.	6.8	.58

CAL		CNTR	METERS		R.T.
	1	71	.		0.8
	2	87	.		1.0
	3	106	.		1.21
	4	130	.		1.49

	TIME	CNTR	METERS	M/SEC	TIME	METERS	CNTR	METERS	DIFF METERS	R.T.
A 1	2.05	43	22.6	11.01	.23	2.5	97	.	4.7	.43
2	2.08	42	22.0	10.60	.21	2.2	60	.	5.8	.54
3	2.40	51	26.8	11.15	.19	2.1	76	.	5.8	.52
4	2.49	56	29.4	11.80	.21	2.5	122	.	4.2	.36
5	2.09	52	27.3	13.06	.19	2.5	75	.	6.3	.48

	O TO A	A TO B	Σ
B 1	.32	.29	.61
2	.34	.23	.57
3	.42	.29	.71
4	.42	.28	.70
5	.43	.27	.70

Figure A-6. Reproduction of data sheet from surprise study.

to the brake (response time), 0.48 sec in this case. During that period of time the car traveled 5.6 m.

The distance from accelerator release to the obstacle is noted in column 7, labeled "CNTR." That total, 74, is subtracted from the maximum detection distance for this subject and target position (87), and converted to meters (6.8). Dividing this distance by the speed in meters/second gives the perception time, 0.58 sec.

The next four rows, labeled "CAL," record the maximum detection distance for the target in each of four positions (columns 6 and 7).

The next five rows, labeled "A," are for the alerted trials. The format is the same as that described for the surprise trial.

The last five rows, labeled "B," are for the brake trials, responding to the hood-mounted lamp. Column 1 ("O to A"), records the perception time, from the onset of the lamp until the subject lifts his/her foot from the accelerator. Column 2 ("A to B") records response time, accelerator to brake. Column 3 ("Σ") is the sum of the other two.

Results--Young Subjects. Figures A-7, A-8 and A-9 are normal probability plots of the data from this study. Figure A-7 is for perception time only; i.e., that time from the first sighting of the obstacle until the subject began to respond by lifting his/her foot from the accelerator. Figure A-8 is for response time only: i.e., the time for

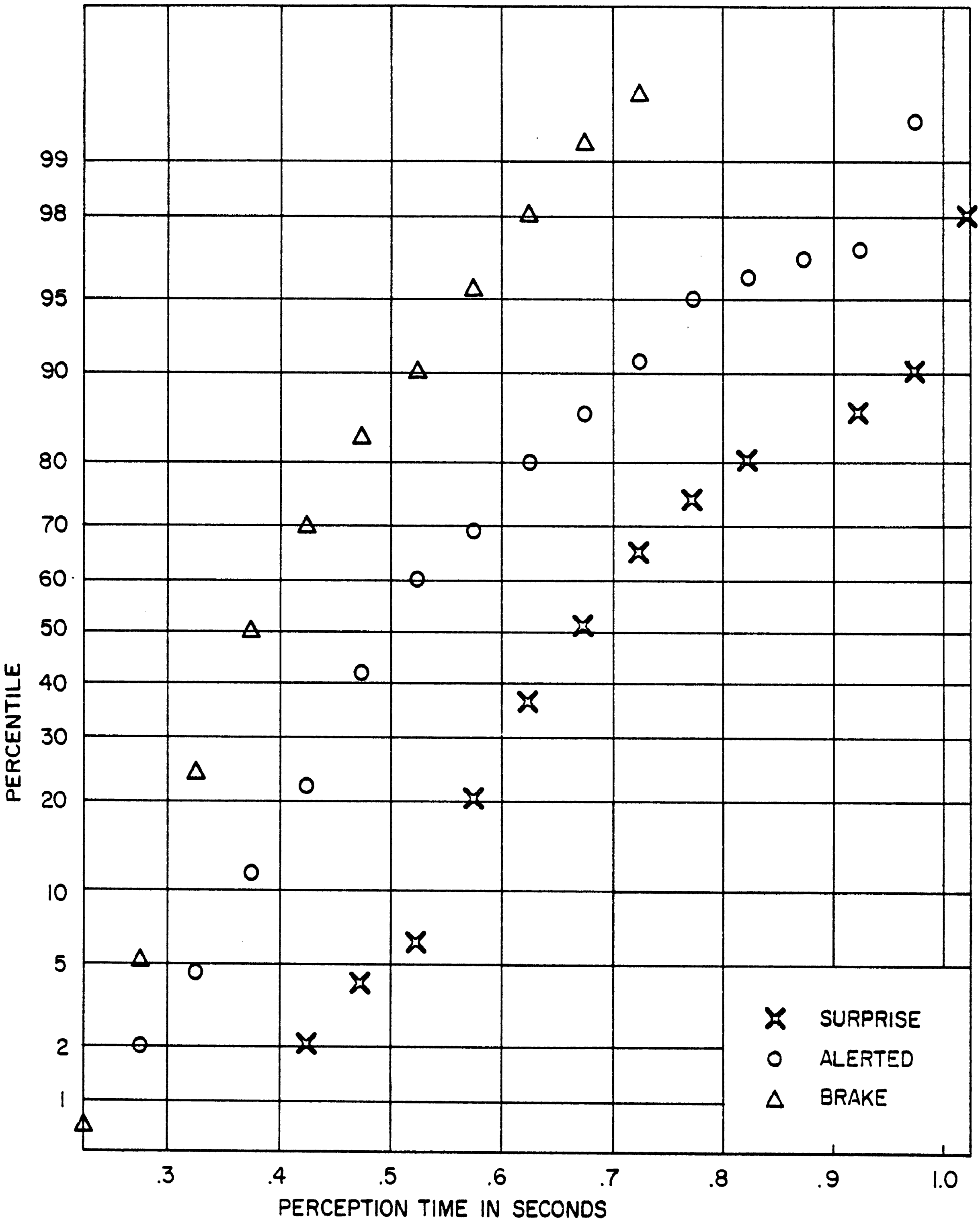


Figure A-7. Normal probability plot of perception times for young subjects.

the subject to move his/her foot from the accelerator to the brake. Figure A-9 is for the total perception-response time. Each figure shows the data from all three studies, labeled "surprise," "alerted," and "brake."

In examining Figure A-7 it is interesting to note that the distributions of perception times for the surprise and alerted conditions are almost perfectly parallel, differing by about 0.2 sec. On the other hand, the distribution for the brake trials is much shorter. The 5th and 95th percentile range for the surprise condition is about half a second, from about 0.47 to about 0.97 sec. The same percentile range for the brake condition is about 0.3 sec, from about 0.28 to about 0.58 sec.

While the distributions for perception in the surprise and alerted conditions are of almost identical width, this is not true of response (Fig. A-8). Here, the mid-90th percentile range for the surprise condition is again about half a second, from about 0.23 to about 0.73 sec. The response times for the alerted and brake conditions averaged much less and were less variable. The 5th percentile response time for both the alerted and brake conditions was about 0.10 sec. The 95th percentile response times differ somewhat, being about 0.28 sec for the brake condition and 0.40 sec for the alerted conditions.

The total perception-response times are shown in Figure A-9. These data suggest that a 95th percentile perception-response interval for a population of young to

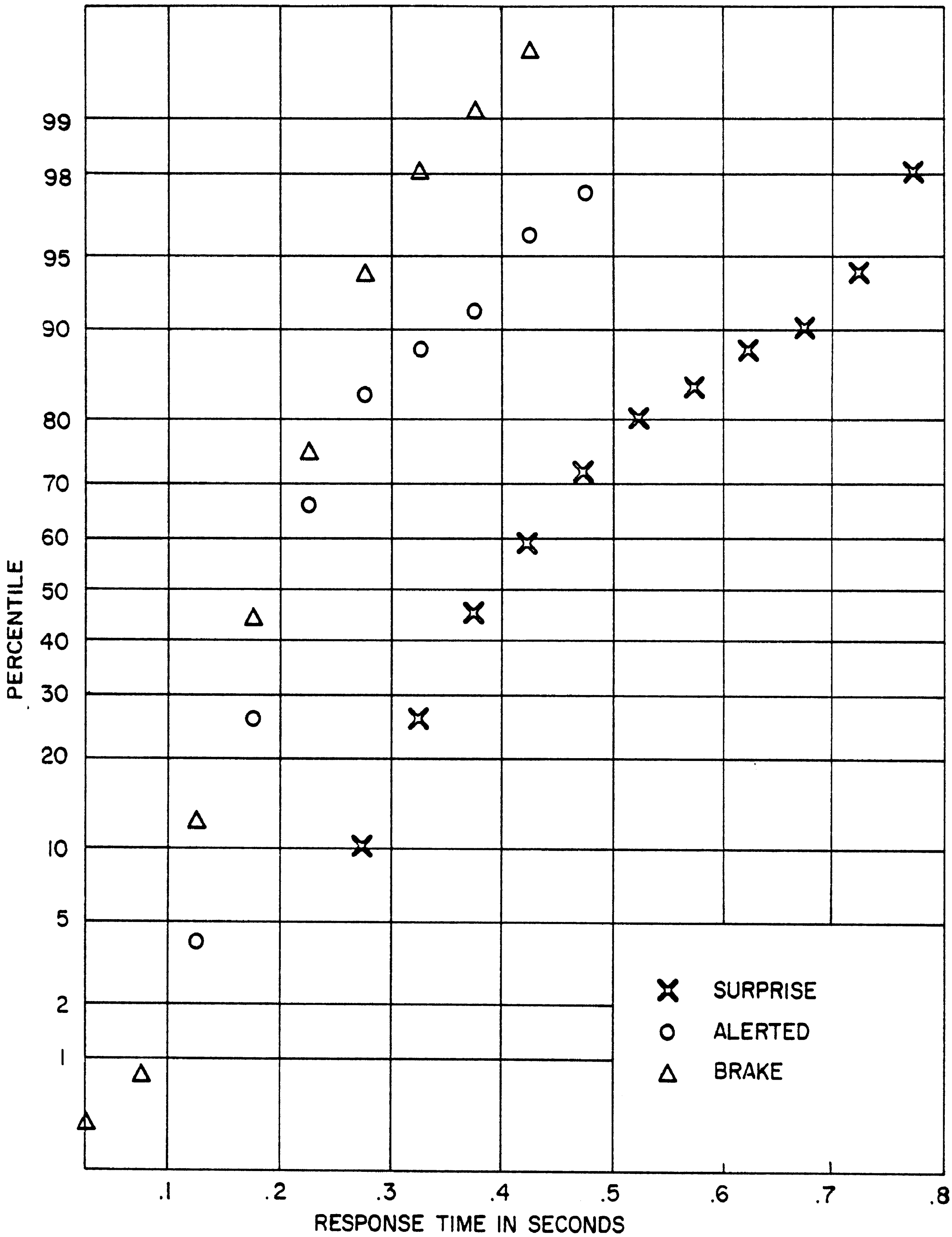


Figure A-8. Normal probability plot of response times for young subjects.

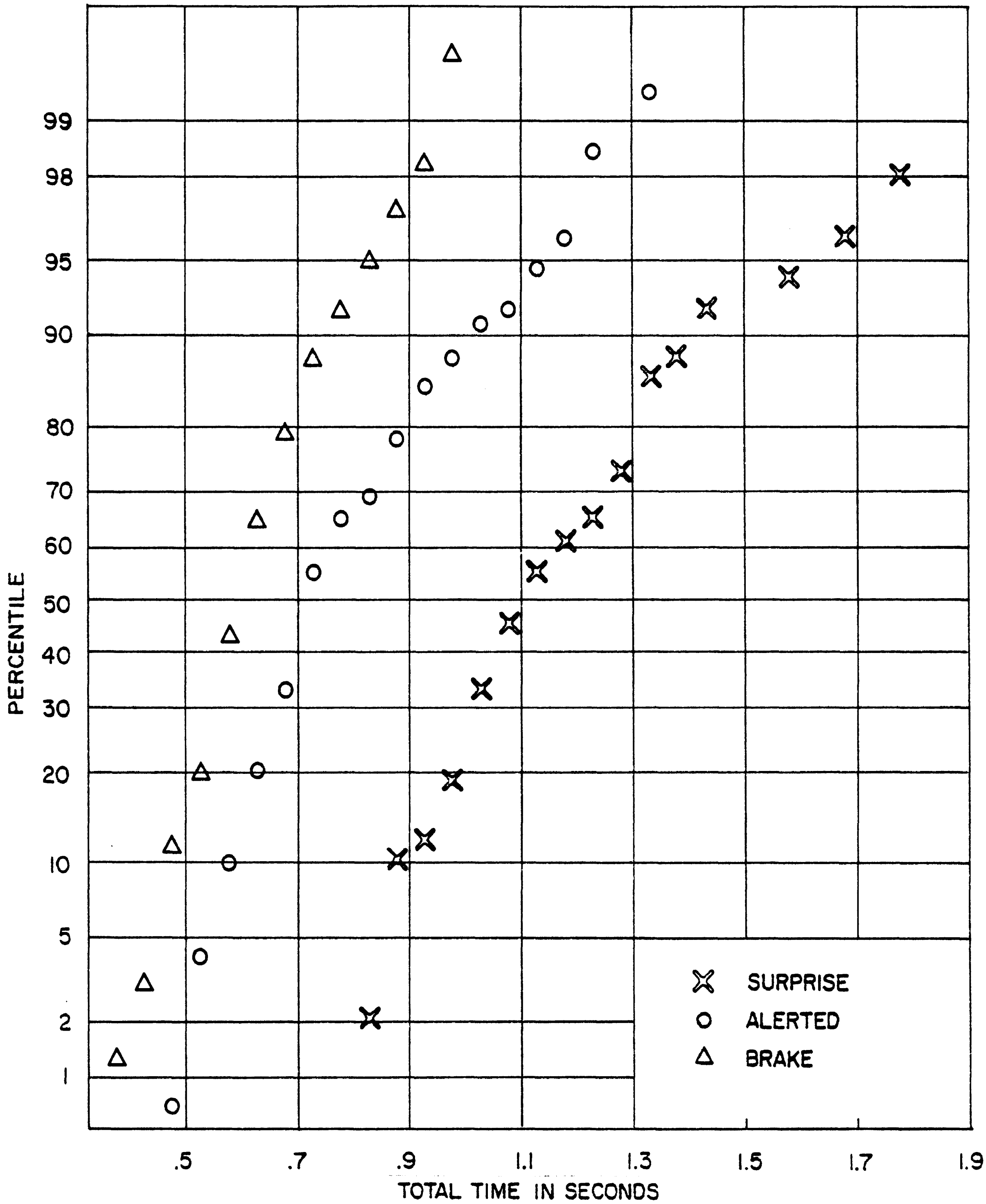


Figure A-9. Normal probability plot of total perception-response times for young subjects.

middle-aged drivers confronted with an unexpected roadway obstacle is about 1.6 sec.

Results--Older Subjects. Figures A-10, A-11, and A-12 summarize the data for the older subjects. They should be compared with Figures A-7, A-8, and A-9.

Figure A-10 shows the distribution of perception times for the surprise, alerted, and brake conditions. The older subjects' perception times in the surprise condition were somewhat longer on average than those of the young subjects. A comparison at specific percentile levels shows the following:

Percentile	Young	Older
20th	0.57 sec	0.63 sec
50th	0.67 sec	0.75 sec
90th	0.95 sec	1.00 sec

On the other hand, the distribution of perception times for the alerted condition are very similar for the two groups. A percentile comparison shows the following:

Percentile	Young	Older
20th	0.42 sec	0.42 sec
50th	0.50 sec	0.50 sec
90th	0.70 sec	0.67 sec

The major difference between the two age groups is in the brake condition. There is little difference in the distribution of the alerted and brake conditions for the older subjects, while there is a considerable difference for

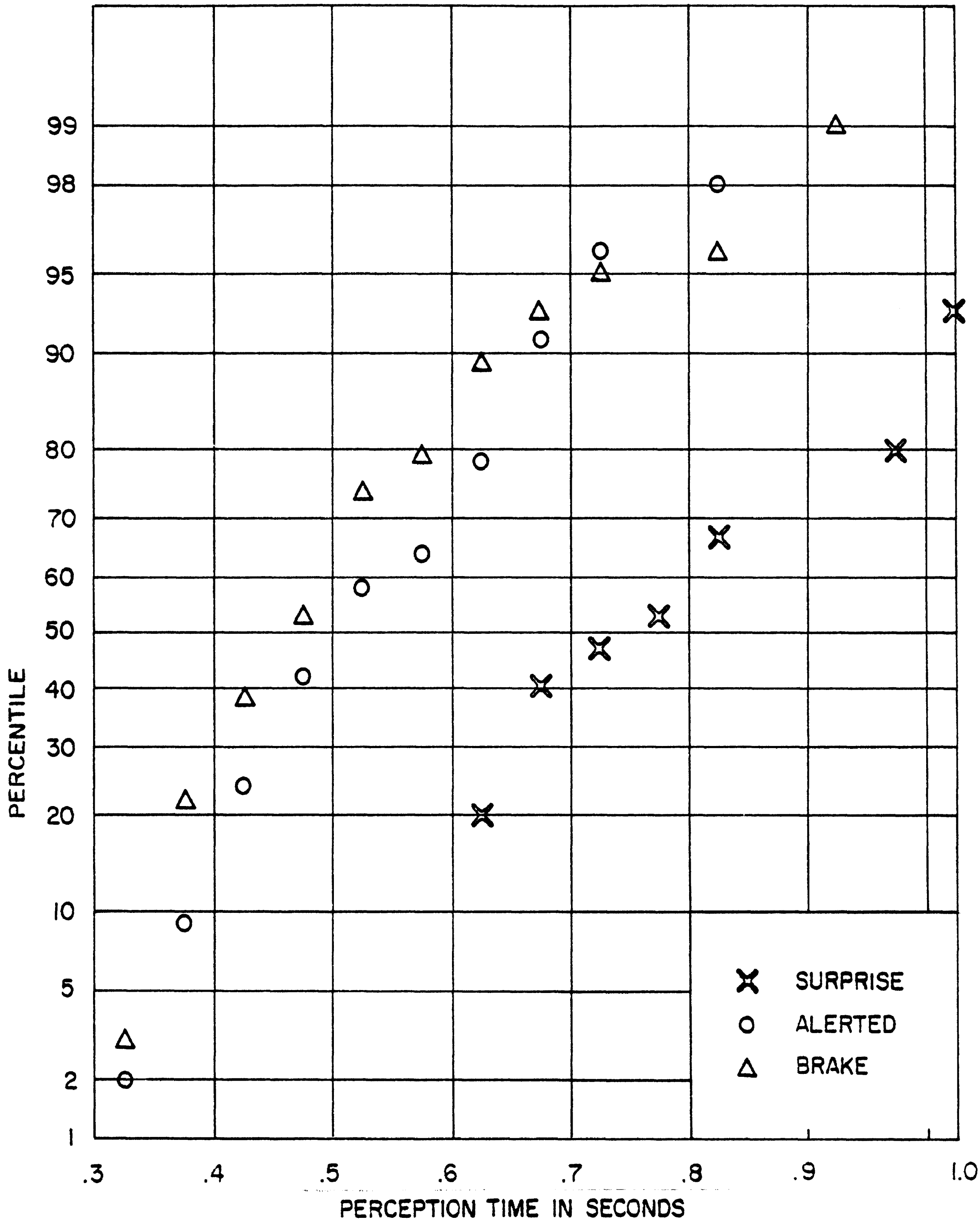


Figure A-10. Normal probability plot of perception times for older subjects.

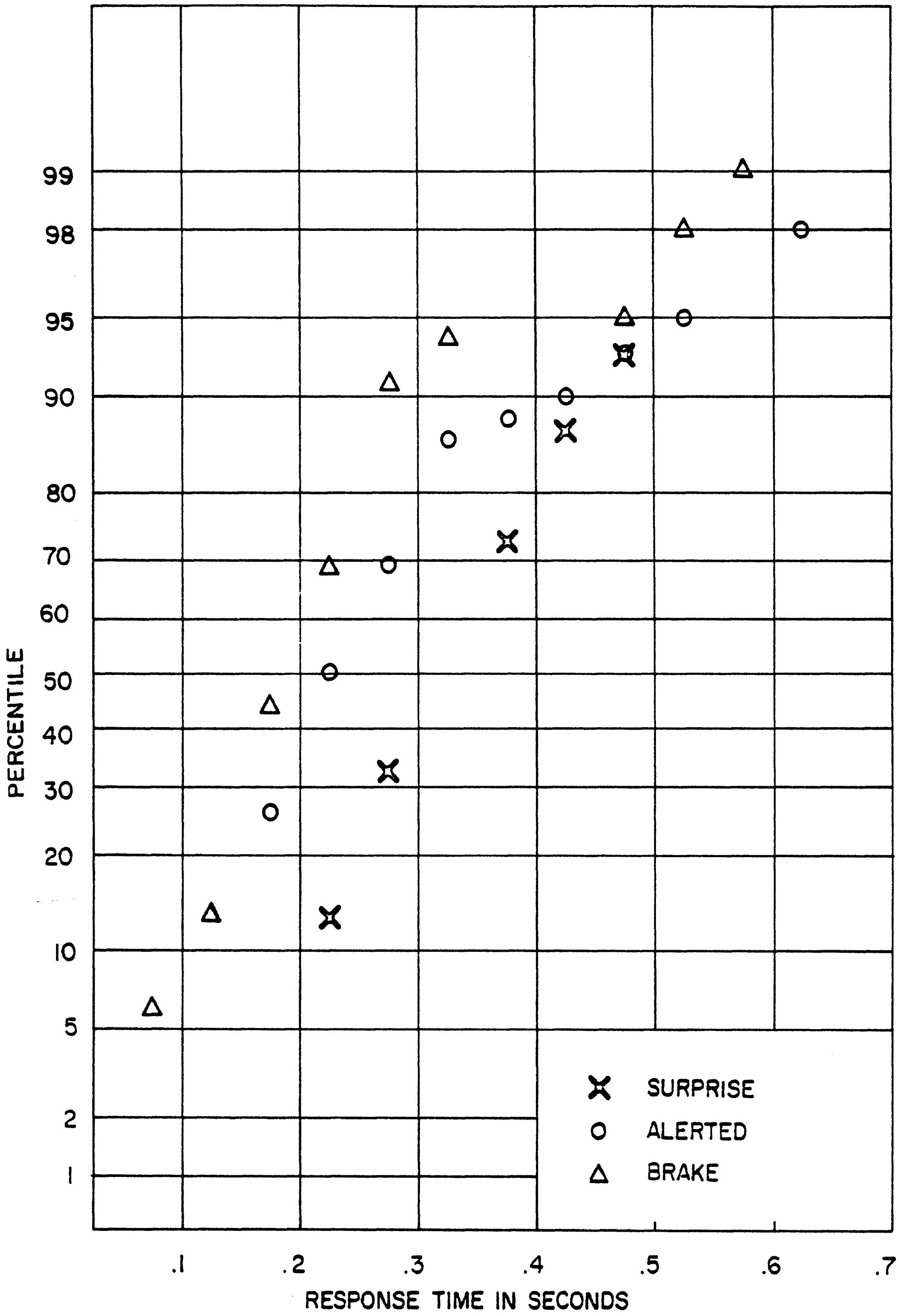


Figure A-11. Normal probability plot of response times for older subjects.

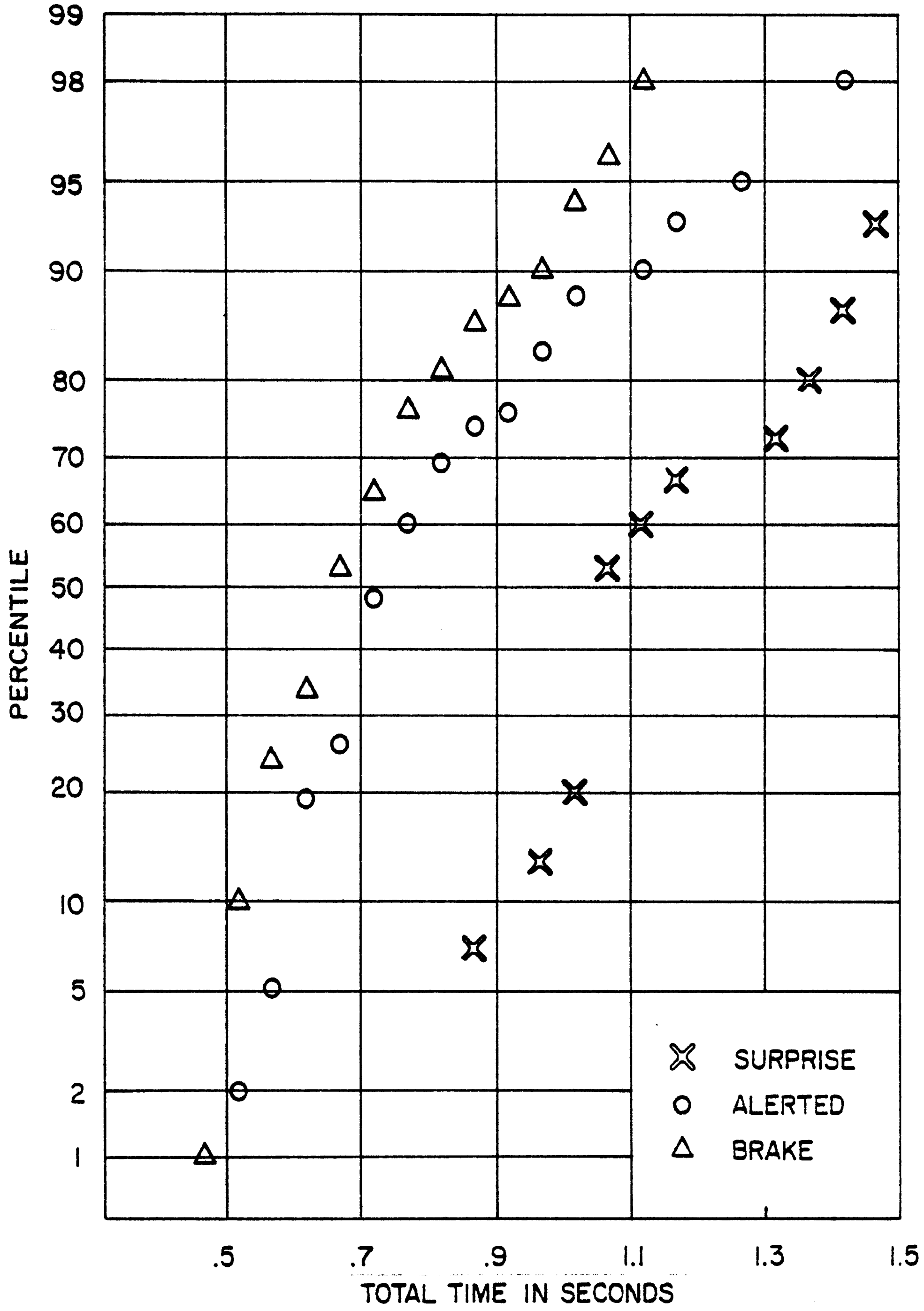


Figure A-12. Normal probability plot of total perception-response times for older subjects.

the younger subjects. A percentile comparison shows the following:

Percentile	Young	Older
20th	0.32 sec	0.37 sec
50th	0.38 sec	0.47 sec
90th	0.53 sec	0.63 sec

The response times for the older subjects are shown in Figure A-11. These data should be compared with Figure A-8. The brake response times for the older subjects were shorter on average than those of the younger subjects in the surprise condition. A percentile comparison shows the following:

Percentile	Young	Older
20th	0.31 sec	0.25 sec
50th	0.39 sec	0.33 sec
90th	0.67 sec	0.44 sec

The response times for the older and younger subjects were very similar for the alerted and brake conditions, except beyond the 90th percentile, where the older subjects' times tended to be longer.

Figure A-12 shows the total perception-response time distributions for the older subjects. This should be compared with Figure A-9. A comparison of the surprise and alerted distributions for these two groups of subjects will show that they are very close. Only the times for the brake condition tend to be longer for the older subjects.

Discussion. The results of this study suggest that a 95th percentile perception-response interval for an individual confronted with a totally unexpected roadway obstacle is about 1.6 sec. This applies to drivers of all ages and both sexes.

The subjects in this study were possibly abnormally alert. They had been driving for only about 10 to 15 minutes at the time of the surprise event, and the presence of an authority figure (the experimenter) in the back seat may have made them more cautious and attentive than usual. So far as is known, none of them were under the influence of alcohol or any other substance that might alter their response times.

This being the case, the distributions shown in Figures A-9 and A-12 are somewhat conservative. The question now becomes to what extent these distributions must be altered to better approximate real-world conditions.

In order to adequately answer this question two types of data are required. First, perception-response time distributions are required as a function of various levels of the variables of interest (e.g., fatigue, alcohol). Second, the incidence of the various levels of the variables in the driving population must be known.

A literature review of the effects of several variables on perception-response time was provided in the project Interim Report (_2_). This information is briefly reviewed, as follows:

1. Alcohol. There have been a number of studies of the effects of alcohol on perception-response time. Most of these have concerned levels at or near the usual minimum for "impaired" (i.e., 0.1 percent). The findings are quite variable. The maximum reported impairment was a 45 percent increase in response time. Two studies report an increase of about 33 percent. Other investigators report increases of 20 percent and 7 percent. One study found no differences at all.

2. Marijuana. This popular recreational drug has not been studied as extensively as alcohol. The results are also highly variable. One study found a range of increases in perception-response time from 5 to 65 percent, depending on the dosage. Another investigator reports a range of decrements from 5 to 12 percent.

3. Other drugs and chemicals. Of the host of drugs and chemicals in use today only a few have been examined in terms of their effect on perception-response time. Interactive effects resulting from combinations of these substances are almost totally unexplored. Where perception-response time data are reported the drug-related effects are generally small. Some are positive and some are negative.

4. Stress. One study looked at noise as a stressor. Perception-response times were reduced.

5. Fatigue. This is a difficult area to study because the effort to measure fatigue effects often relieves it. However, the data that are available suggest that long

exposure to driving may increase perception-response time by about 10 percent.

On a basis of the available data, it is clear that certain transient factors (e.g., use of alcohol) can increase driver perception-response time very significantly. Other factors appear to have little effect. In general, there is not nearly as much information as would be desired for accurate estimates.

The second issue is the incidence of the various factors in the driving population. There is some information on alcohol, otherwise there is little to go on.

As a result of these deficiencies in the available data, it is not possible to provide an accurate estimate of a corrected "true" perception-response interval.

It is possible to make some reasonable inferences about the distribution of values, however. For example, the 5th percentile perception-response time measured in this study was about 0.8 sec. This value would probably not change significantly. Adding in drivers who are tired, inebriated, inattentive, etc. would increase the variance of the distribution, but not its minimum end. The 5th to 95th percentile range in the surprise study was about 0.8 sec (i.e., from about 0.8 to 1.6 sec). If the range were doubled, i.e., from 0.8 to 2.4 sec, the 95th percentile would still be below the current standard of 2.5 sec.

A doubling of the response range (equivalent to increasing the 95th percentile value by 50 percent) seems like a reasonable correction, given the available data just reviewed. The largest changes noted were 45-65 percent, and generally they were much less.

A 50 percent increase in the 95th percentile perception-response time (to 2.4 sec) is very close to the present standard of 2.5 sec. Given the long experience the engineering profession has had with the 2.5-sec value, it is desirable that it be retained.

There have been a number of attempts to estimate the perception-response interval. Most of these involved subjects who were to some degree alerted to the nature of the study. However, it would be useful to compare these results with those of various conditions in the present study.

For example, in an unpublished study by one of the authors of this report, using a technique very similar to the brake condition, a mean response time of 0.15 sec and a mean total time of about 0.60 sec were reported. Lister (26) had similar results. The median response time in the present study was about 0. 18 sec and the median total time about 0.60 sec under the brake condition for the young subjects.

Johansson and Rumar (27) stopped passing motorists and asked them to tap their brake in response to a horn sound "sometime in the next 10 kilometers." An assistant measured the time from the horn blast until the brake lights came on at the test site 5 km down the road. The median response time was 0.66 sec. Practically all responses were 1.6 sec or less. On the basis of these and other data Johansson and Rumar recommended a perception-response interval of 1.5 sec as appropriate for the 90th percentile. This compares with a 90th percentile value for young subjects in the present study of about 1.4 sec.

One of the most interesting and realistic perception-reaction time studies is that of Triggs and Harris (28). Their estimates are based on first-sighting to brake application times of unsuspecting passing motorists at a number of sites in Australia. Several different "hazards" were used. Table A-1 summarizes their results.

The first eight situations were encountered after cresting a hill. The railway signals were started by the experimenters on a flat stretch as the subject vehicle approached. The car following was obtained by slipping a test vehicle in front of another car in traffic and measuring that driver's response time to a brake application. (An amphometer is a speed measuring device widely used in Australia. It consists of two pneumatic tubes across the road 25 m apart.)

Items 4 through 8 in Table A-1 are possibly similar in that the first response of many motorists may be to check their speed. This inserts another step in perception-response time and lengthens it. Such objects as police cars and speed traps are of primary interest to persons going in excess of the speed limit. Indeed, the authors note that

Table A-1. Eighty-fifth percentile response times of unsuspecting motorists to various stimuli.

Condition	P-R Time (sec)
1. C.R.B. "Roadworks Ahead" Sign	3.0
2. Protruding vehicle with tyre change	1.5
3. Lit vehicle under repair at night	1.5
4. Parked Police Vehicle	2.8
5. Amphometer : Beaconsfield	3.4
6. Amphometer : Dandenong North	3.6
7. Amphometer : Gisborne	3.6
8. Amphometer : Tynong	2.5
9. Railway crossing : Night (General Population)	1.5
10. Railway crossing : Night (Rally drivers)	1.5
11. Railway crossing : Day	2.5
12. Car following	1.3

From: Triggs and Harris, 1982 (28)

the faster their subjects were going the more likely they were to brake and the faster they braked.

Item 1, the "Roadworks Ahead" sign, should produce heightened attention and scanning behavior in drivers. However, in the absence of actual construction activity, rapid brake responses do not seem appropriate.

A railroad crossing flasher (items 9 through 11) is a danger signal that should command a prompt response. At night, in a rural area, it is a very conspicuous signal as well. The difference between day and night may be a function of conspicuity of the signal. It may also reflect the fact that the approaching driver could inspect the surround more easily to determine whether a train was nearby.

The two items that most nearly approximate the conditions of the surprise study are 2 and 3. Both of these are potential hazards that are fairly conspicuous. If true, this is encouraging. Triggs and Harris' subjects were passing motorists who were, presumably, representative in terms of factors such as alertness, drugs, alcohol, etc. The 85th percentile perception-response times recorded from these people were very close to those recorded in the surprise study (1.5 vs. 1.4 sec respectively). This suggests that accepting the 2.5-sec value is not likely to lead to significant problems.

It was anticipated that the difference between the surprise and alerted conditions would allow an estimate of the time required for the identification-decision process. For both the young and older groups this difference, for the perception stage, was about 0.2 sec. However, especially for the young group, there was another 0.2 sec difference in the response stage between the surprise and alerted conditions. This suggests that many people released the accelerator prior to completing a response decision under the surprise condition.

It was also anticipated that there would be a correlation between perception and response times. This proved not to be the case. The correlations were 0.03 and 0.12 for the young and older groups respectively. Neither value is significant ($p > 0.05$).

PARAMETRIC STUDY

Independent Variables

Obstacle Height. Three levels: 3 in. (7.6 cm), 6 in. (15.24 cm), and 12 in. (30.5 cm). All of these obstacles were 3 ft (0.91 m) wide.

Obstacle Width. Three levels: 6 in. (15.24 cm), 3 ft (0.91 m), and 8 ft (2.44 m). All of these obstacles were 6 in. (15.24 cm) high.

Obstacle Contrast. Three levels: white, pale yellow, and dark blue. As it turned out, the yellow obstacle (the standard from the surprise study) provided the least contrast against the pavement on the test road. The white and blue obstacles were, respectively, much lighter and much darker than the road surface. All of these obstacles were 6 in. (15.24 cm) high and 3 ft (0.91 m) wide.

All of the above required seven different obstacles, since each of the sets of three includes the standard obstacle from the surprise study. Four replications were run, making a total of 28 trials.

All obstacles were made of foam rubber that was light yellow in color. The white and dark obstacles were made by wrapping the target with a white and blue denim fabric respectively.

Dependent Variables

The dependent variables were the same as described previously.

Equipment

The equipment was the same as described previously.

Test site

It proved difficult to make the many passes required for this study at the site used for the surprise study. The primary problem was the lack of convenient turn-around space. Accordingly, we looked for another site. The test was finally set up at the site originally selected for the surprise study. It will be recalled that this site was abandoned because the times emerging from the surprise trials were felt to be too long. However, it was thought satisfactory for the parametric study.

The street is paved with asphaltic concrete. There are two 11-ft (3.35 m) lanes for each direction, separated by a

30-ft (9.14 m) wide grassy median. There are no shoulders; concrete curbs mark the road edges. Figure A-13 is a photograph of the approach to the site, before the obstacle became visible. Figure A-14 is a view from the crest, with an obstacle in place. Figure A-15 is a photograph of all the obstacles.

Subjects

Twenty-six persons participated in the study. All had been involved in the surprise study. We invited back individuals who had little difficulty in the alerted portion of that study. Of the total, 15 subjects were young, 11 were older.

Procedure

As in the case of the surprise study, two experimenters were required: one to place and remove the obstacles, the other to ride with the subject and take the data.

Each subject reported to the Institute at an assigned time, signed a consent form, and was escorted to the test car. One of the experimenters drove to the test site and explained the procedure to the subject on the way. Briefly, the subject was told that the procedure was identical to the alerted portion of the first study, and that whenever he saw the obstacle he should release the accelerator and tap the brake pedal as quickly as possible. The subjects were also told that our interest was in the characteristics of the obstacle itself, and they could expect to see a variety of sizes and colors of obstacles.

Figure A-13. Photograph taken approaching crest at site of parametric study.

Figure A-14. Photograph taken from crest at site of parametric study, showing one of the obstacles in place

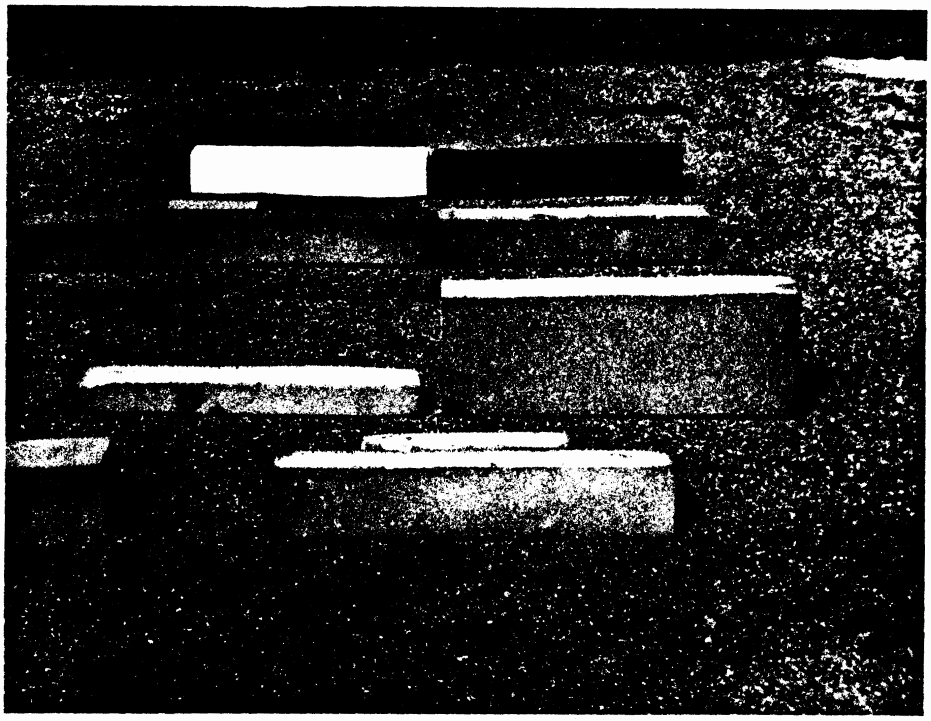


Figure A-15. Photograph of various obstacles used in parametric study.

On arrival at the site, the experimenter made one run to demonstrate the required procedures before turning the car over to the subject. Timed trials then began. Trials that were lost for any reason were repeated at the end.

Results

Young and Older Subjects. A first concern was whether performance at the two sites was comparable. To check this the distributions for perception, response, and total time were compared for the same obstacle at each site. These data are given in Table A-2.

A review of Table A-2 shows that performance at the two sites was very similar with this obstacle. The largest differences are at the 90th percentile. The alerted data are from the distributions for young subjects and the parametric study included older persons as well, who differed mainly at the higher percentiles. Hence, the comparisons are quite favorable.

Figure A-16 is a normal probability plot of the perception interval for the various obstacles. These data were subjected to analysis of variance (ANOVA) and the differences between obstacles were significant ($p < 0.01$). A post hoc test (Newman-Keuls) was conducted to compare individual means. The results of this analysis are summarized in Table A-3. An inspection of this table will show that, in general, the high and the narrow obstacles were most difficult for the subjects to detect. Not

Table A-2. Comparison of percentile response times to the same roadway obstacle at two different sites.

Variable	Percentile	Alerted Portion of Surprise Study (sec)	Parametric Study (sec)
Perception	20	0.42	0.44
	50	0.50	0.53
	90	0.70	0.83
Response	20	0.17	0.17
	50	0.21	0.22
	90	0.25	0.33
Total	20	0.63	0.67
	50	0.72	0.76
	90	0.97	1.13

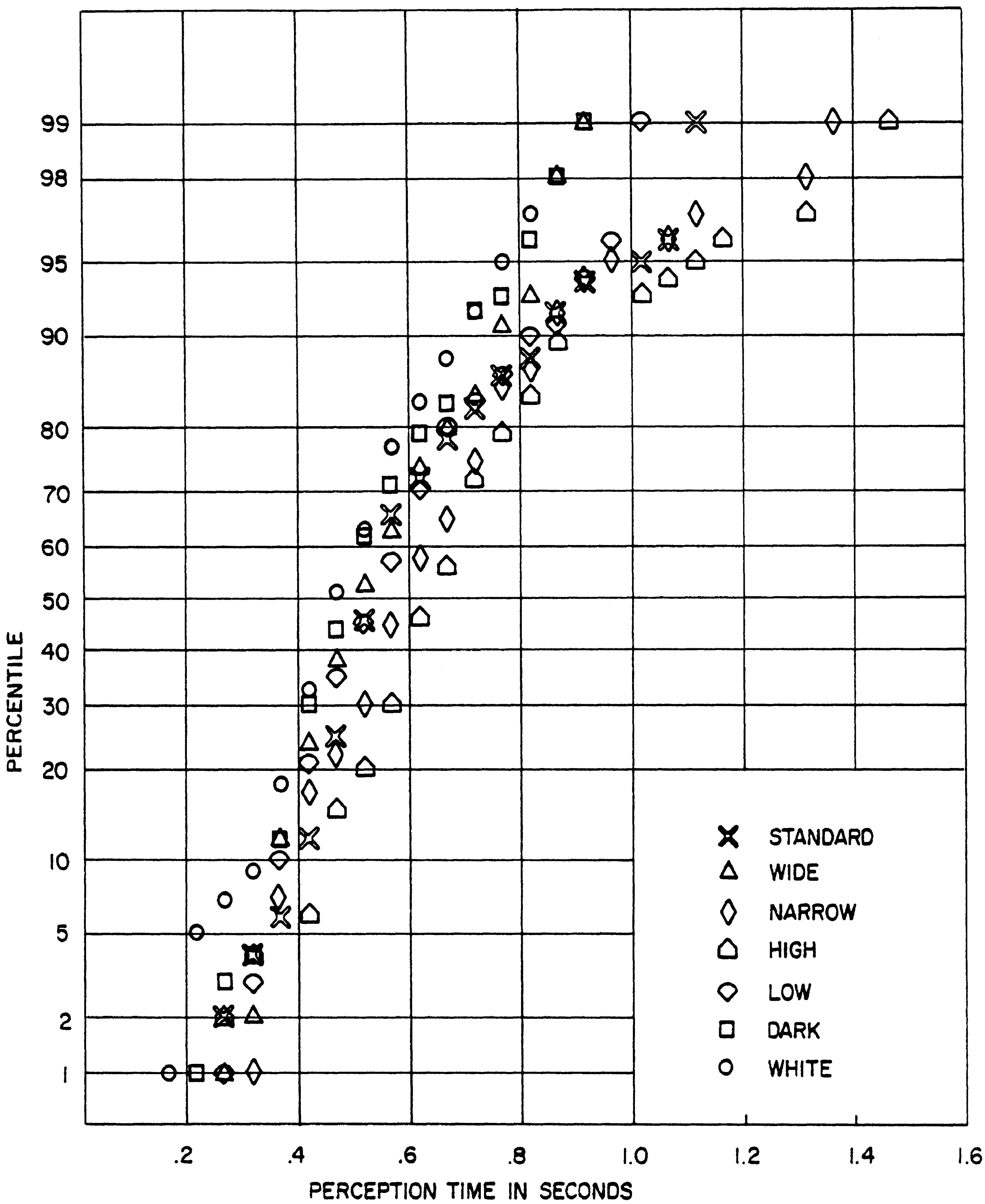


Figure A-16. Normal probability plot of perception time in parametric study.

surprisingly, the wide, white and dark obstacles were easiest to detect.

Figure A-17 is a normal probability plot of the response interval for the various obstacles. Use of ANOVA on these data showed no significant differences in response to the various obstacles ($p > 0.05$).

Figure A-18 is a normal probability plot of the total perception-response interval for the various obstacles. Since the components had been statistically analyzed and all significant variance found to be associated only with the perception interval, an analysis of the total time was judged irrelevant.

Discussion

The results of this study indicate that differences in the characteristics of the obstacle affect the driver's perception interval. Most of the differences were not surprising. For example, obstacles that take up a large share of the lane or contrast well with the road surface are more readily detected than small obstacles or those that contrast poorly.

One difference was the high obstacle. On average, it was responded to about 0.10 sec slower than the standard. The sight distance difference between the two was about 31 ft (9.5 m). At typical speeds for the subjects, this represented between 0.6 and 0.7 sec.

It is probable that the slower response to the high obstacle is attributable to the fact that, when placed in

Table A-3. Results of Newman-Keuls tests on perception times to different obstacles.

Response Times to this Obstacle	Are Significantly* Slower than Response Times to this Obstacle(s)
High	White, dark, wide, low, standard and narrow
Narrow	White, dark, wide
Standard	White, dark
Low	White

*$p < 0.05$

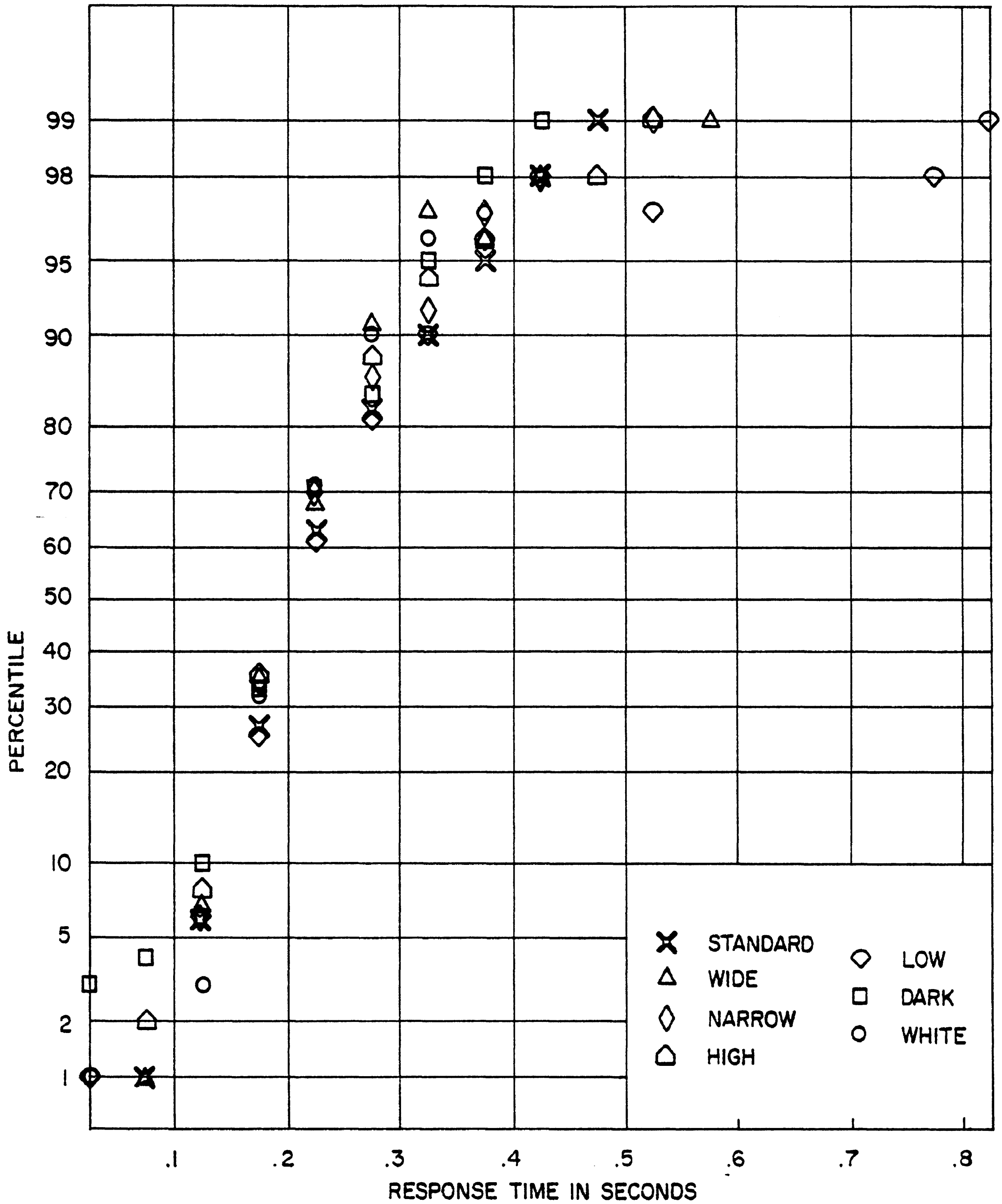


Figure A-17. Normal probability plot of response time in the parametric study.

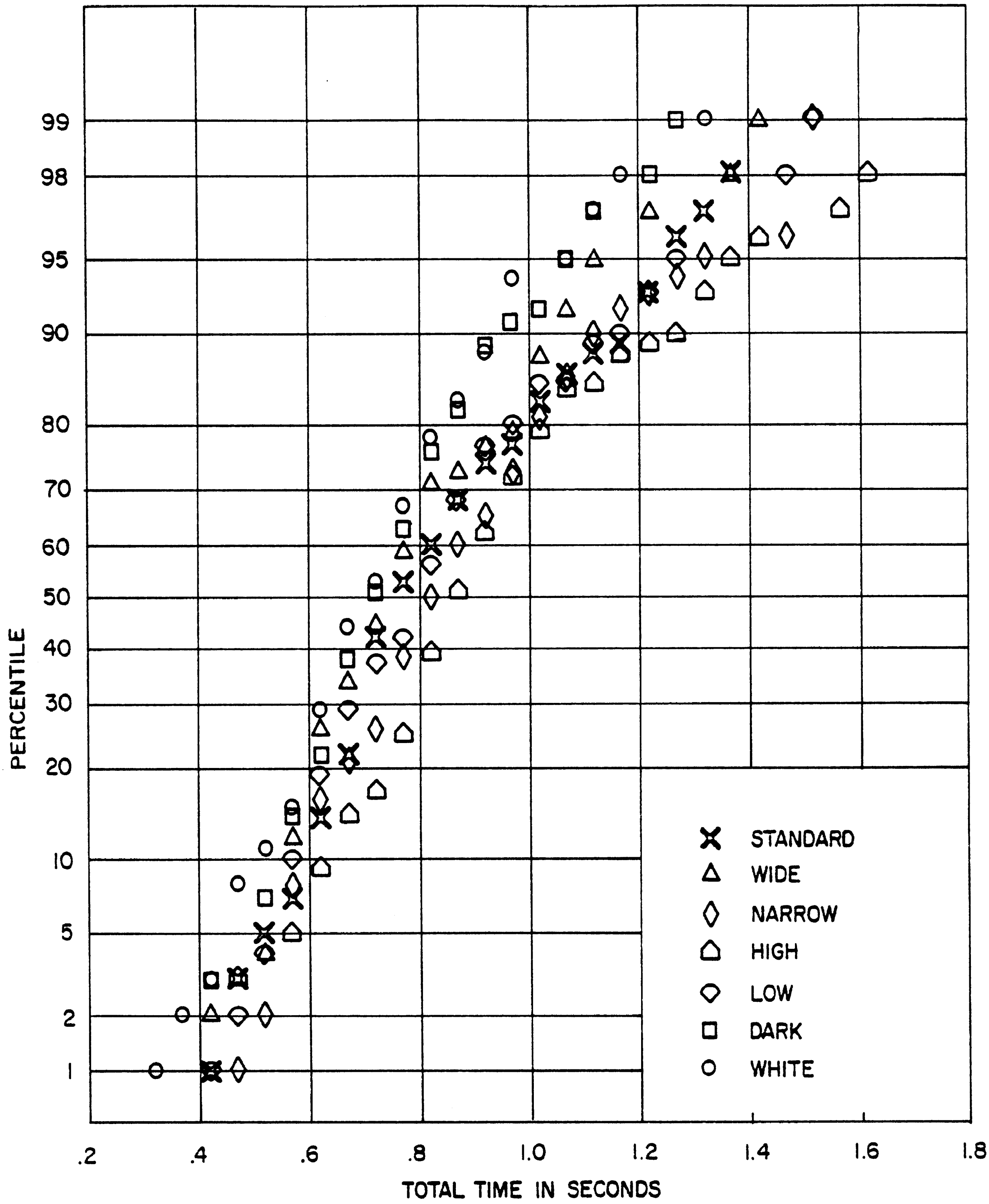


Figure A-18. Normal probability plot of total perception-response times in the parametric study.

the most forward position, it first became visible against a background including the edge of the roadway and the curb. In all other cases the targets were seen against the relatively homogeneous roadway surface. This raises an interesting point. The taller the standard object selected the more likely it is to first become visible against backgrounds other than the road surface, and the longer it will retain those backgrounds. Thus, minimum response times will be associated with objects having zero height, because they will always appear against the road surface. As object height increases, perception time will increase as well, on average, because of the variable backgrounds encountered. There are no data to indicate precisely what that relationship would be. However, if relatively short obstacles are used (e.g., the 6 in. used in the U.S.) the difference should be slight.

CONCLUSIONS--PERCEPTION-RESPONSE TIME

The results of this research indicate that the distribution of response times of ordinary drivers to a completely unexpected roadway hazard is such that a perception-response interval of 2.5 sec is appropriate.

The perception-response interval depends on several factors. One of the more important is object contrast. The data used for the estimate given here are based on a relatively low-contrast condition. However, poorer contrast conditions are possible, and would result in an upward shifting of the distribution of total time. There are no data on the contrast characteristics of roadway obstacles encountered in the real world.

APPENDIX B

STUDIES OF VEHICLE BRAKING DISTANCES

INTRODUCTION

This appendix presents technical discussions derived from studies of: (1) the frictional characteristics of pneumatic tires operating on wet roadway surfaces, (2) the efficiencies of vehicle braking systems, and (3) the abilities of drivers to utilize the deceleration capabilities of their vehicles.

As performed in this project, examinations of the braking characteristics of current highway vehicles have been directed at producing findings that either (1) support the recently adopted AASHTO policy (1) or (2) indicate the need for modifications of these recommendations. The findings, upon interpretation (see Chapter Three), suggest the advisability of considering increases in braking distances and/or pavement skid number to allow drivers a margin for keeping their vehicles under control while coming to a stop from highway speeds on wet roads.

In order to evaluate the numerical values proposed in Ref. (1), existing empirical evidence has been reduced to equations describing roads, tires, braking efficiencies, and driver control efficiencies (see Table 7 in Chapter Two). Although these equations are based on measured data, they have been examined in an attempt to ascertain that they are compatible with the laws of physics. The general idea pursued herein is that the friction number f, representing vehicle deceleration on a level road, can be computed as a function of three factors:

1. A quantity, f_x, describing the tire-road interface.

2. A braking efficiency, BE, representing the influences of the proportioning of the braking effort amongst the various wheel locations and the overall distribution of mass throughout the vehicle.

3. A driver control efficiency, CE, predicting the ability of the driver to utilize the deceleration capability afforded by f_x and BE. The "findings" are a set of equations that may be used to calculate braking distances.

The state of knowledge and certainty concerning the equations used to describe the factors influencing braking distance varies widely, particularly in the area of driver control efficiency. Further experiments are suggested in Chapter Four.

FACTORS INFLUENCING BRAKING DISTANCES

Wet Roads

Water Depths. Rainfall rates seldom exceed 0.4 in. per hour in the United States. At this level of rainfall, water depths on typically superelevated or crowned highways are not deep enough to cause hydroplaning (see Yeager(29)) unless poor drainage or rutting allows puddles to form. Example calculations indicate that pavement texture depths (See Balmer and Gallaway (30)) and tire groove capacities provide sufficient water drainage paths and storage volumes to prevent tire-road friction values from reaching extremely low values, even though the tops of pavement asperities may be covered by water on smooth roads during heavy rainfalls. A function describing the influence of water depth has not been identified in this study; however, the results presented correspond to average water depths (measured from the mean texture depth) in the neighborhood of 0.02 in., which is typical of the depths encountered in steady rainstorms of severe intensity.

Skid Numbers. Information quantifying pavement frictional characteristics exists in the form of ASTM skid numbers. Distributions of skid numbers on samples of roads have been compiled by various organizations and researchers. Researchers (Leu and Henry (31) and Taoka (19)) have employed an equation of the form of Eq. 1 (see Table 7 in Chapter One) to express experimental results. Equation 1 is easy to apply to various percentile roads. For example, we find that skid numbers for various speeds on a 15th percentile road may be approximated by the following expression:

$$SN_V = 28\ e^{-0.0115(V-40)} \tag{B-1}$$

Equations 1, 2, and 3 can be used to show that the numerical coefficients in Eq. B-1 correspond to an ASTM SN_{40} value of 28 and a mean texture depth of 0.015 in. or a skid number gradient of 0.322 skid numbers per mph.

Tires

Sliding Friction Versus Skid Number. The ASTM tire used in skid number measurements has less braking traction than that of commercially available car tires when tests are made on poor, wet roads. Although a large number of tire and pavement factors influence friction on wet roads, a number of sources indicate that new car tires on the average are approximately 1.2 times more effective than the ASTM tire. For example, see the experimental measurements of Ervin and Winkler (32), the General Motors Tire Performance Specification (Peterson, et al. (33)), and the improvement factor obtained from blading, siping, and cross-grooving stated by Yeager (25).

In contrast to car tires, new truck tires are less effective than the ASTM tire on poor, wet roads. Results from research studies (Dijks (34) and Boyd et al. (35)) have shown that truck tires are approximately 0.7 times as effective as car tires; that is, the frictional effectiveness of truck tires is approximately 84 percent of the effectiveness of the ASTM tire.

Maximum Rolling Versus Sliding Friction. Passenger car braking systems are currently designed to achieve high levels of friction utilization without locking wheels. Particularly, on poor, wet roads the maximum rolling friction is considerably higher than the level of sliding friction. Equation 6 (see Fig. 1) is a numerical fit to the rolling-to-sliding-friction-relationship developed empirically by Horne and Buhlman (36). Equation 6 approximates the experimental results with good accuracy in the range $0.2 < f_s < 0.6$.

Test data (Boyd et al. (35)) show that the maximum rolling friction of truck tires is approximately 1.45 times the sliding friction on both good and poor, wet roads. This characteristic of heavy truck tires differs from that of car tires in that ratios of "peak-to-slide" friction may reach 2 to 1 for car tires operating on poor (f_s approximately equal to 0.2), wet roads.

Tire Wear. Bald tires are dangerous to use on wet roads because their frictional capabilities can be exceptionally low at high speed. To guard against this danger, many states have laws requiring tread depths exceeding 2/32nds of an in. The change in frictional capability with tread groove depth has been shown to be non-linear in form with a tire having 2/32 in. of tread groove depth being considerably better than a smooth tire (see Eq. 9, also Dijks (34)).

The differences between the frictional capabilities of new and bald tires can be described in terms of pavement texture depth and velocity (see Eq. 8, also Dijks (37)). By combining results from Eqs. 8 and 9, one can estimate the frictional capability of a tire worn to any specified level of tread groove depth.

Truck-tire design emphasizes those factors tending to reduce wear. To some extent, this emphasis on wear accounts for the lower wet traction attributed to truck tires (Dijks (34)). However, the influence of the depth of the grooves in truck tires tends to be similar to that of car tires over the range of groove depths applicable to car tires. For new truck tires with 15/32-in. groove depths (for rib type tires) and 31/32-in. groove depths (for lug type tires), the influence of tread wear is very small until groove depths wear to levels below 12/32nds of an inch. Below 12/32 in., the same wear characteristics as those attributed to car tires are believed to apply to truck tires.

Braking Systems

Major changes in passenger car characteristics between 1970 and 1980 have brought about major changes between the braking performance of current (1982) vehicles and vehicles manufactured several years ago (38). The braking systems employed in new passenger cars are designed to be very efficient in utilizing the peak friction available at the tire-road interface. Through the use of special valves and selected proportioning arrangements, this efficiency can be maintained at high values over a wide range of friction levels (Radlinski and Flick (39)). Specifically, the average efficiency of a sample of 1982 passenger cars has been found to be approximately, 91 percent, based on using Radlinski and Flick's data (39) to calculate braking efficiencies when the vehicles are operated with a driver or a driver plus another person in the front seat (i.e., the loading states that apply at least 80 percent of the time (Johnson and Segel (40)).

Experimental data (Radlinski (41)) show that the braking performance of current heavy trucks is not greatly different from that of vehicles manufactured prior to 1975, even though government braking regulations applying between 1975 and 1978 caused vehicle manufacturers to install anti-lock braking systems on vehicles manufactured during that period. The braking systems now installed on heavy trucks are proportioned according to the gross axle weight ratings specified for the various axles. This arrangement results in braking characteristics such that the rear wheels of heavy trucks are likely to lock up on slippery surfaces. Under these circumstances, the classical equations for braking efficiency are useful for estimating the efficiency of heavy straight trucks (see Eq. 11 pertaining to the case where the rear wheels are approaching lock up). Test results (Radlinski (41)) show that unladen straight trucks are likely to have the longest braking distances of all heavy vehicles except "bob-tail" tractors. When typical values are employed in Eq. 11, we find that the braking efficiencies of unladen heavy trucks will range from 0.55 to 0.59 as peak friction varies from 0.43 down to 0.21.

Fully laden heavy trucks, including both straight trucks and articulated vehicles, are brake torque limited because at 100 psi (the highest pressure available for applying the brakes) the maximum capabilities of the individual brakes are generally not high enough to lockup wheels on good, unslippery road-surfaces. In addition, surveys of trucks in use indicate that many pneumatically actuated brakes are either significantly out of adjustment or defective in service. Consequently, fully laden heavy trucks are often incapable of making more than a 0.4g stop, even on the best roads. Nevertheless, when the road is slippery, the vehicles have superior performance in their laden state as compared to their performance in an unladen state.

Current trends indicate that braking performance, as measured by adhesion (friction) utilization, may increase for all classes of trucks in the future. Light trucks are starting to employ proportioning devices, thereby approaching passenger car levels of braking efficiency. Medium trucks in the United States are increasingly being outfitted with hydraulic disk brakes on all wheels. The proportioning of these braking systems provides higher efficiencies than those attained by heavy trucks. If the philosophy of no front wheel lockup becomes less popular in the United States, the braking efficiencies of heavy trucks can be expected to increase in the future.

Braking Control Efficiency

In experiments conducted by Mortimer et al. (23), automobile drivers attempted to follow a slightly curving 10-ft lane while stopping as quickly as possible. The desired path was a cosine wave with a peak-to-peak amplitude of 3 ft and a period of 400 ft. The results showed that drivers could not utilize the ultimate braking capabilities of their vehicles because they could not modulate their brake pedals well enough to use most of the peak-tire-road friction available while simultaneously avoiding loss of directional control due to wheel lockup. These experiments were conducted on two wet surfaces with peak frictional characteristics of 0.4 and 0.71. Equation 13 is based on the results of Ref. (23), and it provides a means for estimating the influence of tire-road friction and vehicle braking efficiency on the control efficiency of car drivers.

In the literature review conducted during this project the research team did not find any information on the braking control efficiencies of truck drivers. A limited set of experiments was performed in this study to evaluate the braking control efficiency of truck drivers. The results of these studies indicate that professional drivers of heavy trucks could usually achieve more than 62 percent of the braking capabilities of their vehicles in an unladen condition during a braking-in-a-turn maneuver. However, in so doing, this group of drivers failed to stay within a 12-ft lane in approximately 1/6th of their attempts to stop quickly from 40 mph.

Braking Distance Calculations

The Influence of Velocity. As discussed by Henry (24), calculation of braking distance requires integration of the equation of vehicle motion because the frictional characteristics at the tire-road interface change as the vehicle's velocity changes during a stop. Also, aerodynamic drag is changing throughout the stop. A simple numerical integration routine has been used in this study to estimate the distance traveled while slowing from a higher velocity, V_i, to a lower velocity, V_f (see Eq. B-2, below):

$$\Delta D = (V_i^2 - V_f^2)/(f_{\bar{v}}\ 29.94) \qquad \text{(B-2)}$$

where:

ΔD = the distance traveled; and

$f_{\bar{v}}$ = the average deceleration in units of gravity applying to velocities between V_i and V_f. (Equations 1 through 12 in Table 7, Chapter One, are used to evaluate $f_{\bar{v}}$).

The Role of Control Efficiency. The calculated control efficiency (Eq. 13 or Eq. 14) is to be applied to the ideal braking distance capability of the vehicle because the original data on which control efficiency is based were gathered as a ratio of actual to ideal braking distance. Control efficiency is not to be included in the numerical integration algorithm specified by Eq. B-2. Equation B-2 is to be applied to results obtained from Eqs. 1 through 12 to estimate braking distances from initial speeds ranging from 20 to 80 mph (see Chapter Two).

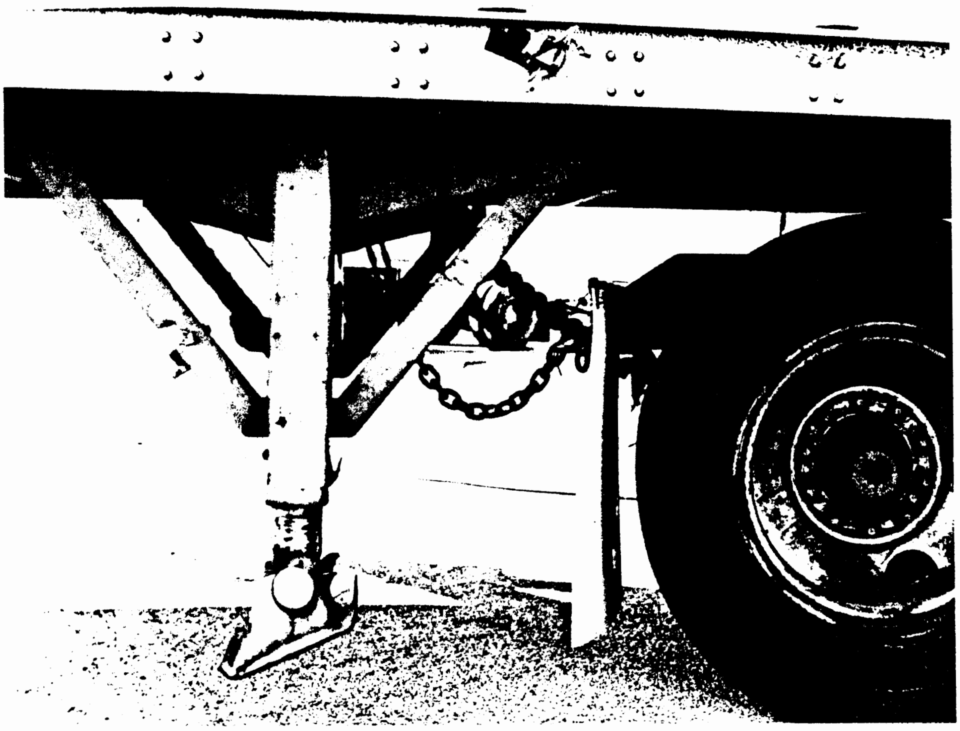
The value of control efficiency is a very important quantity in determining braking distances if vehicles are to be kept under control during sudden, rapid stops. The amount of experimental evidence available for quantifying control efficiency on poor, wet roads is quite limited. This study has used test results from Ref. (23) for passenger cars and results from experiments conducted as part of this project for trucks. As indicated in Chapter Four, these studies of the efficiencies with which drivers can control cars and trucks need to be repeated to confirm the previous results. In addition, the entire subject of control efficiency would benefit from further research. With regard to cars, braking efficiencies are close to 1.0 (100 percent) for new models operating on slippery surfaces. However, the advantages of this high level of efficiency can only be realized if drivers are capable of modulating their brakes effectively. With respect to trucks, drivers and owners in the United States seem to prefer little or no front braking--a situation that provides poor braking efficiency and may lead to jackknifing or vehicle spin even though the lateral force capabilities of the front wheels are retained during stopping. Trucks in other parts of the world are proportioned to achieve better braking efficiency. Apparently, the U.S. philosophy reflects a belief that drivers of trucks are better off trying to steer around obstacles rather than trying to stop for them. Further research into truck braking may provide the basis for improved stopping performance of heavy trucks.

Control Efficiencies of Truck Drivers

An experimental program aimed at assessing the control efficiencies of truck drivers when stopping heavy vehicles was completed as part of this study. The results of these experiments contribute to the information needed for use in estimating braking distances for heavy vehicles. Prior to the completion of these tests, there did not exist (to our knowledge) any data indicating the efficiency with which truck drivers can modulate braking effort in order to stop quickly without encountering problems with directional stability and control.

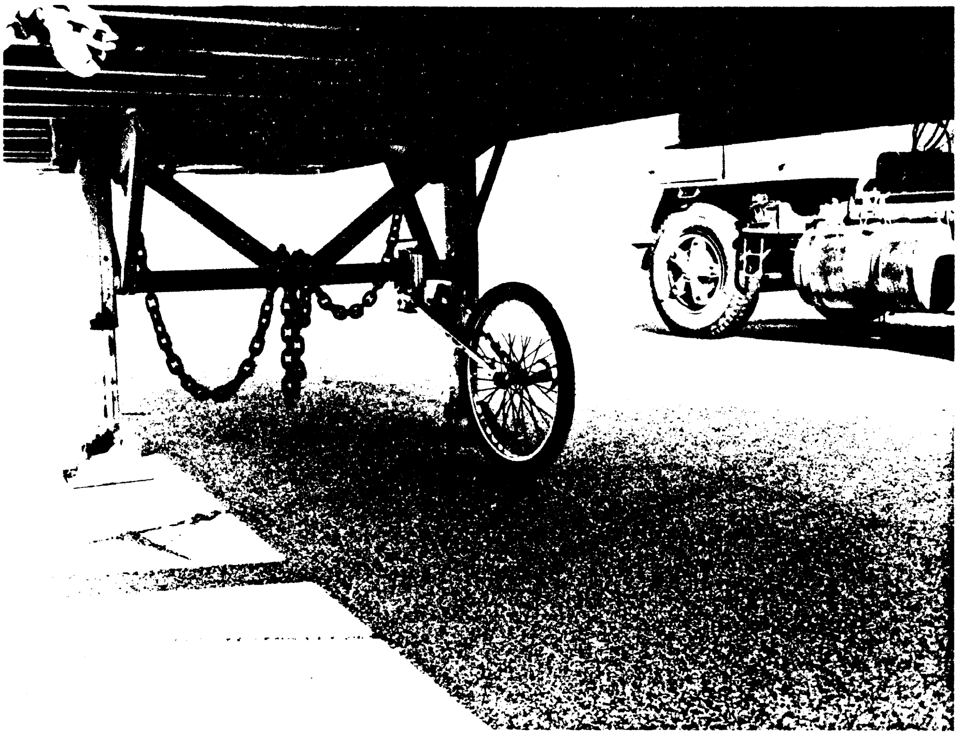
<u>Test Preparations and Procedures</u>. The preparations for testing included: (1) borrowing a highway tractor and two semitrailers; (2) outfitting the semitrailers with safety devices (chains to prevent extreme jackknifing and outriggers to prevent rollover); (3) installing a system for automatically controlling brake pressure in order to evaluate the braking capability of the vehicle alone,that is, without any degradation due to driver limitations (it is noted that the system for controlling brake pressure was only employed to evaluate the performance of the system without the driver modulating the brake pedal (treadle valve); this device was switched out of the braking system during the driver/vehicle tests, and in the driver/vehicle tests, data were obtained with the vehicle and its equipment in a normal condition for highway operation); (4) instrumenting the vehicles to measure initial velocity and braking distance; (5) loading one of the semitrailers, thereby providing the capability for testing loaded and empty vehicles; (6) arranging for the services of a group of truck drivers; and (7) developing test procedures and driver instructions. (Pictures of the test equipment are shown in Figure B-1.)

(a) Test tractor and semitrailer
(note: outriggers are used to prevent rollover if control is lost.)

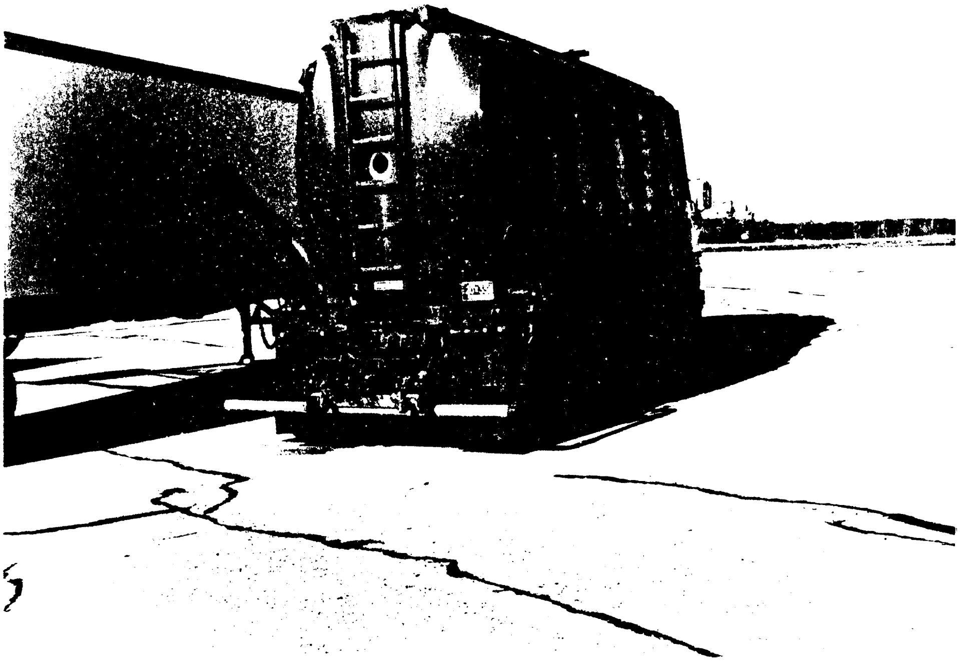


(b) Safety chain installed to prevent jackknifing

Figure B-1. Test equipment used in measuring control efficiency.



(c) "5th wheel" used to measure initial velocity
and braking distance



(d) Tanker with spray bar for watering the test area

Figure B-1. (continued)

The experiment consisted of stopping both a fully laden and an empty tractor-semitrailer combination on a wetted section of the vehicle dynamics area at the Chrysler Proving Grounds. The stops were initiated at 40 mph. The drivers were instructed to stop safely and quickly while keeping the vehicle within a 12-ft lane on a curve with a 1,000 ft radius. (The purpose of the curve was to provide a consistent source of directional control challenge to the driver without significantly altering the braking capabilities of the truck's tires.)

The drivers were given time to familiarize themselves with the vehicles. Then each driver made four stops with the loaded vehicle and six stops with the empty vehicle from 40 mph while trying to follow a curve with a radius of 1,000 ft. The order of testing (starting with either the empty or the loaded vehicle) was varied in an attempt to balance the influence of transfer effects on the test results.

In total, 16 drivers participated in the test program. Six of these drivers were from research organizations engaged in testing heavy trucks. Two of these six test-drivers had been performing braking tests recently. The remaining 10 drivers were members of the teamsters union. These 10 professional drivers all had at least 10 years of experience in driving heavy trucks.

Test Results. The results of the tests showed that the two test-drivers with recent experience in running braking tests were very proficient at achieving close to all of the braking capability of the test vehicles. However, the other drivers were not as skilled at keeping the test vehicles within the lane boundaries and modulating the brake pressure. (The drivers with recent test experience served more as a check on the maximum capability of the test vehicle than as an example of typical braking skill.) The test results for the unladen vehicle (see Fig. B-2) provide bases for estimating braking control efficiency.

The data displayed in Figure B-2 can be interpreted in various ways. For example, an estimate of "good" performance could be obtained from the average of the three best stops performed by the professional drivers. In this case, the average of the performance of the driver/vehicle system is 0.40 g and, since the vehicle's braking capability was measured at 0.5 g, the braking control efficiency is 80 percent.

However, in highway situations the driver clearly does not have the opportunity to "try again." Even though some of the initial attempts of the drivers were either at low deceleration levels, or resulted in jackknifing or trailer swinging motions exceeding the lane boundaries, these control actions may represent misjudgments that could occur in emergency situations. It is interesting to note that out of the 60 runs performed by professional drivers, eleven of these runs resulted in a loss of directional control of sufficient magnitude that, at least, some part of the vehicle departed from the intended lane of travel. These

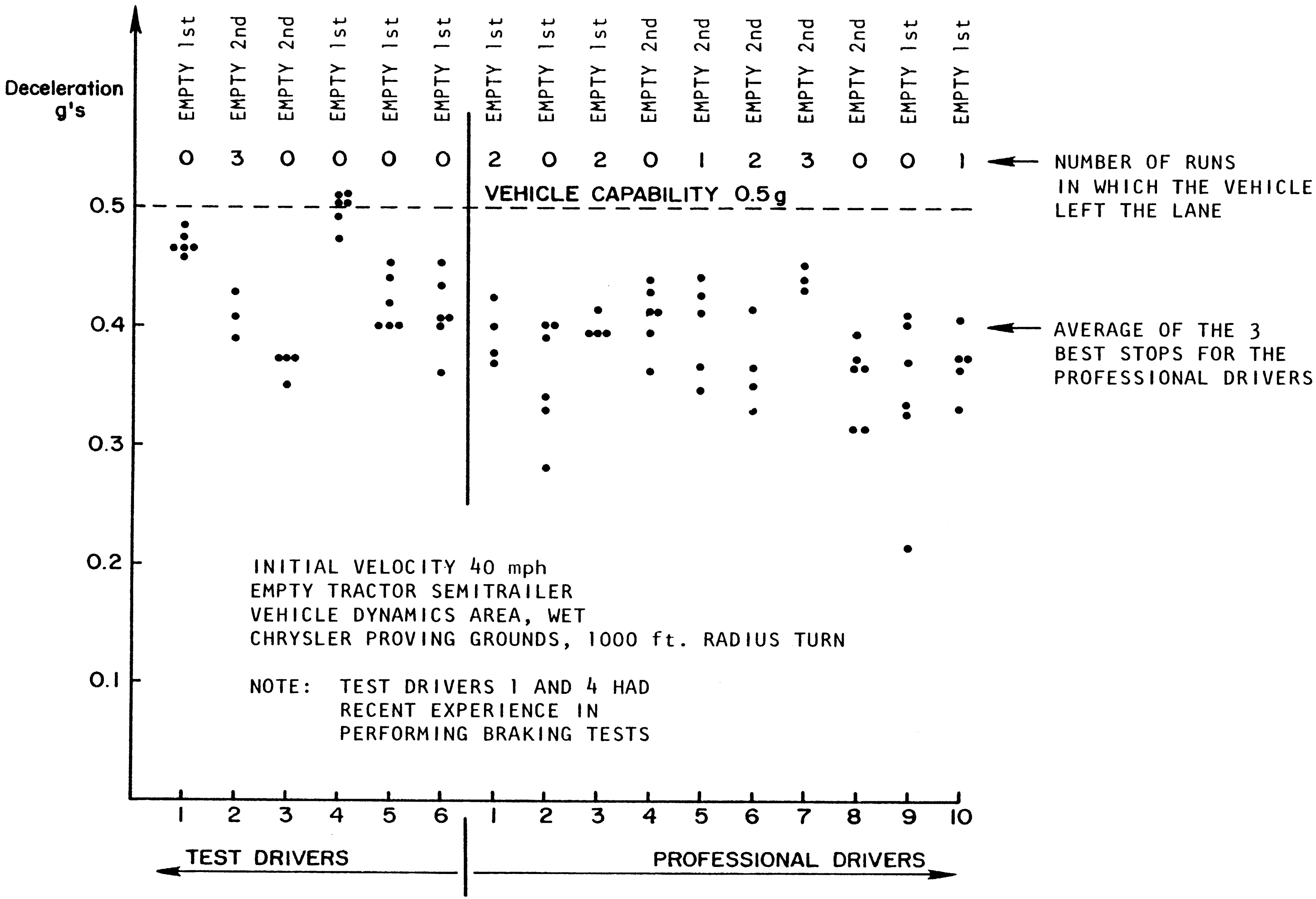


Figure B-2. Test results for an unladen heavy vehicle.

runs might be combined with cases with relatively low deceleration to determine an overall efficiency level. For example, as indicated in Figure B-2, thirteen of the stops made by the professional drivers either (1) resulted in poor directional control or (2) had deceleration levels less than 0.31 g. The braking control efficiency based on this boundary (i.e., 0.31 g), is 0.62 (or 62 percent). This value of 62 percent has been chosen for use in the body of this report.

Although the test data obtained with the empty truck may present a challenge with regard to selecting a reasonable means of interpreting it, it nevertheless clearly indicates that truck drivers on the average are substantially less than 100 percent efficient in utilizing the capabilities of the vehicle's braking system.

The test data obtained using the loaded truck (see Fig. B-3) exemplify an important facet of heavy truck braking. In this case, the vehicle was brake torque limited, that is, the braking system did not have enough capability to lock the wheels regularly. (One wheel on the front trailer axle locked up in approximately one-half of the stops performed with the loaded vehicle. It was impossible to lock either front or rear wheels on the tractor.) In this situation, the optimum driver strategy is to apply full air pressure to the brakes. When full pressure was applied, the loaded vehicle attained a deceleration of approximately 0.42 g, the braking capability

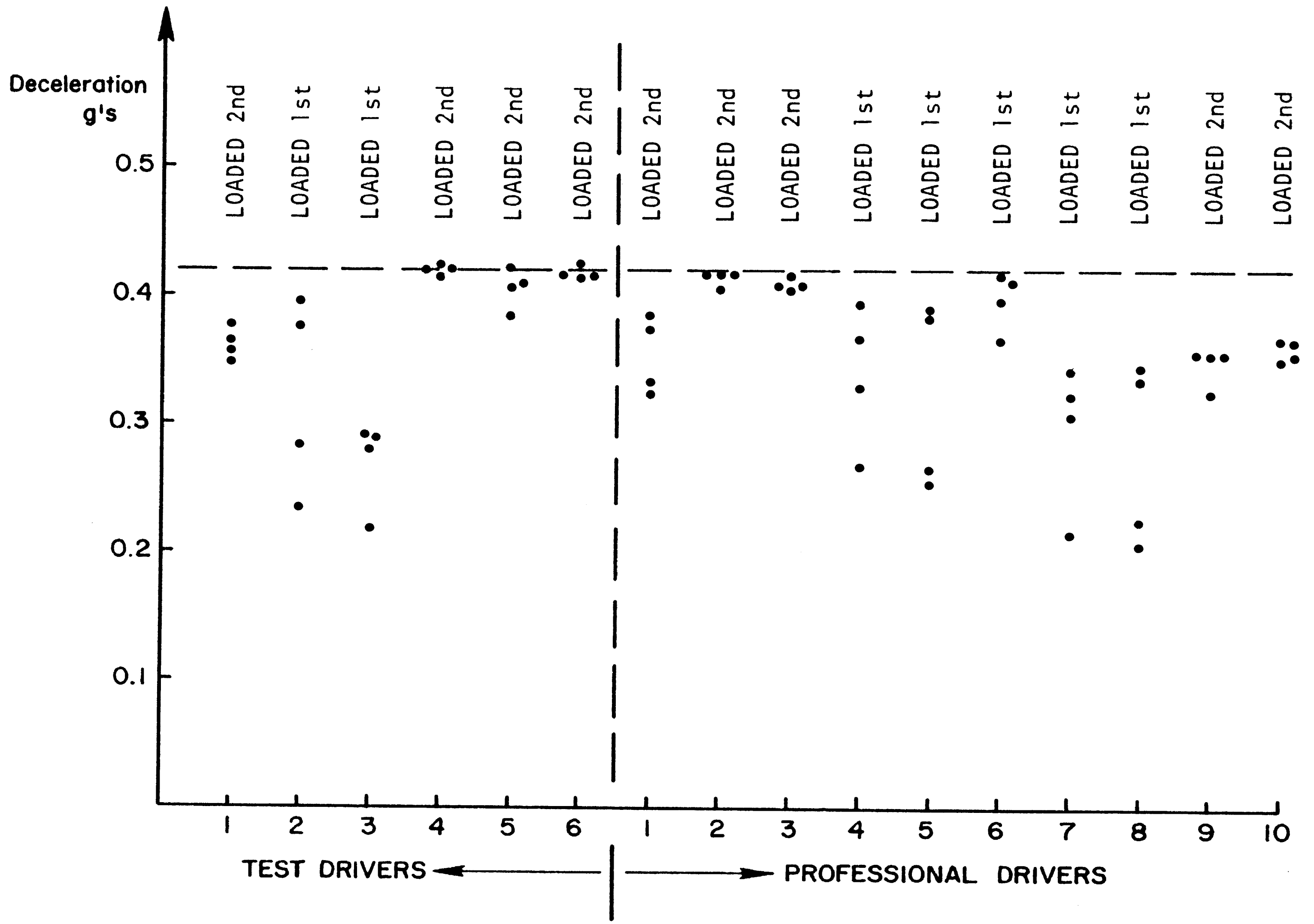


Figure B-3. Test results for a fully loaded heavy vehicle.

of the loaded vehicle. Many of the drivers achieved close to this maximum level of deceleration on at least 3 of their 4 repetitions of the experiment. There were several drivers out of the 16 that were overly cautious and did not achieve the maximum possible deceleration with the loaded vehicle.

The test results for the loaded vehicle were made under conditions which could not seriously challenge the driver's ability to maintain directional control (because no wheels locked up). Under these circumstances, the driver merely needed to apply maximum pressure to achieve best performance. However, the drivers did not know this on their first attempt and, depending on the amount of caution that they exercised, some of the drivers needed 2, 3, or even all 4 trials to realize that they could apply maximum pressure.

The test results for the professional drivers may be separated into two categories (see Table B-1): (1) those who drove the empty vehicle first, and (2) those who drove the loaded vehicle first. Those who had already driven the empty vehicle knew that they could employ high pressure and, as a group, they achieved approximately 95 percent of the available braking capability. In contrast, if their first experience with the test was with the loaded vehicle, the drivers started out cautiously. In this case, the drivers (on the average) achieved approximately 67 percent of their best performance. These results show the need for careful interpretation of the relationship between driver/vehicle performance on the test pad and in highway service.

The test results for the unladen vehicle did not show the sensitivity to the order of testing exhibited by the results for the loaded vehicle; and, hence, the empty vehicle results can be interpreted without concern about the order of testing.

BIBLIOGRAPHY RELATED TO BRAKING DISTANCE MATTERS

The specific references enumerated in this appendix represent a fraction of the material reviewed to develop a simplified model of the frictional interaction between tires and pavements. The following annotated bibliography lists additional books, conference proceedings, research compendiums, and reports that contain information that helped in formulating the equations pertaining to frictional properties as presented in Figure 6:

Meyer, W.E. and Walter, J.D., editors, Frictional Interaction of Tire and Pavement, ASTM special publication 793 (1983). In addition to the specific papers 30, 36, and 24 directly referenced in this study, this publication contains other papers of theoretical and practical interest.

Clark, S.K., editor. Mechanics of Pneumatic Tires, U.S. Department of Transportation NHTSA, U.S. Government Printing Office. This is probably the most comprehensive document ever published on the subject of tire mechanics.

Table B-1. Loaded truck average deceleration.

	Driver Number	Lowest	Highest	Ratio	
Loaded 2nd Empty First	1	.324	.382	.85	First Week
	2	.408	.421	.97	
	3	.403	.416	.97	
Loaded First	4	.267	.397	.67	
	5	.253	.388	.65	
	6	.366	.414	.88	
Loaded First	7	.210	.341	.62	Second Week
	8	.203	.344	.59	
Loaded 2nd Empty First	9	.322	.353	.91	
	10	.350	.364	.96	

Hays, D.F. and Browne, A.L., editors. The Physics of Tire Traction. Plenum Press (1974). This book contains sections presenting (1) an overview of the tire traction problem, (2) aspects of rubber friction, (3) the role of the tire, and (4) the role of the pavement. The articles in this book provide a wealth of useful ideas. Worth special note are the graphs presented by Gough on pages 287 through 290. These figures summarize the influence of pavement texture, tread pattern, tread materials, and speed on both sliding and peak friction.

Skidding Accidents: Tires, Vehicles, and Vehicle Components. Transportation Research Records 621 through 624 (1977). Articles by Gauss, Dijks, and Williams provide pertinent insights derived from European research.

Interaction Between Vehicles and Pavement Surfaces. Transportation Research Record 788 (1980). Articles pertaining to skid number and skid number gradient are presented here.

Hayden, C.M., editor. Pavement Surface Characteristics and Materials. ASTM special publication 763 (1980). This publication contains interesting studies of pavements some of which discuss the sandpatch method for measuring texture and others which discuss wet traction.

Pavement Surface Properties, Evaluation, and Shoulders. Transportation Research Record 666 (1978). This publication

contains articles on pavement texture and skid resistance, also, Ref. (31).

Pavement Surface Properties and Vehicle Interaction. Transportation Research Record 584 (1976). Two studies on pavement friction are contained in this document.

Proceedings International Automobile Tire Conference. Sponsored by SAE and U.S. Department of Transportation (1974). In addition to Refs. (33 and 37), these proceedings contain several articles on handling.

Ervin, R.D. "The State of Knowledge Relating Tire Design to Those Traction Properties Which May Influence Safety. UMTRI Report No. UM-HSRI-78-31. Prepared for Bolt, Beranek, and Newman, Inc. (1978). This report presents a structured review of the traction responses to braking and cornering slip and the sensitivity of those responses to operating and design variables.

APPENDIX C

STOPPING SIGHT DISTANCES UNDER NIGHTTIME DRIVING CONDITIONS

INTRODUCTION

The detection portion of the SSD model is assumed to start when the uppermost portion of the obstacle becomes visible over a crest vertical curve, as depicted in Figure C-1. It is for this reason that an accurate representation of population eye heights is important.

At night, illumination necessary to make the obstacle visible is generally supplied by the vehicle's headlamps. Headlamps are mounted below the eye level of the driver. As a result a situation like that depicted in Figure C-2 can arise, wherein the obstacle is shadowed by the crest vertical curve until sometime after it would have become visible in daylight.

Under the assumption that the principle of SSD is as important at night as it is during the day, it has been argued (e.g., Hills, (42)) that lamp mounting height is a more appropriate parameter than driver eye height.

Electing headlamp mounting height over driver eye height would have a significant impact on SSD recommendations because it would reduce the vertical dimension at the vehicle to 24 in. (61 cm), the current minimum mounting height under Federal Motor Vehicle Safety Standard (FMVSS) 108. If petitions now before the National Highway Traffic Safety Administration (NHTSA) are approved, the minimum mounting height will be reduced to 22 in. (56 cm).

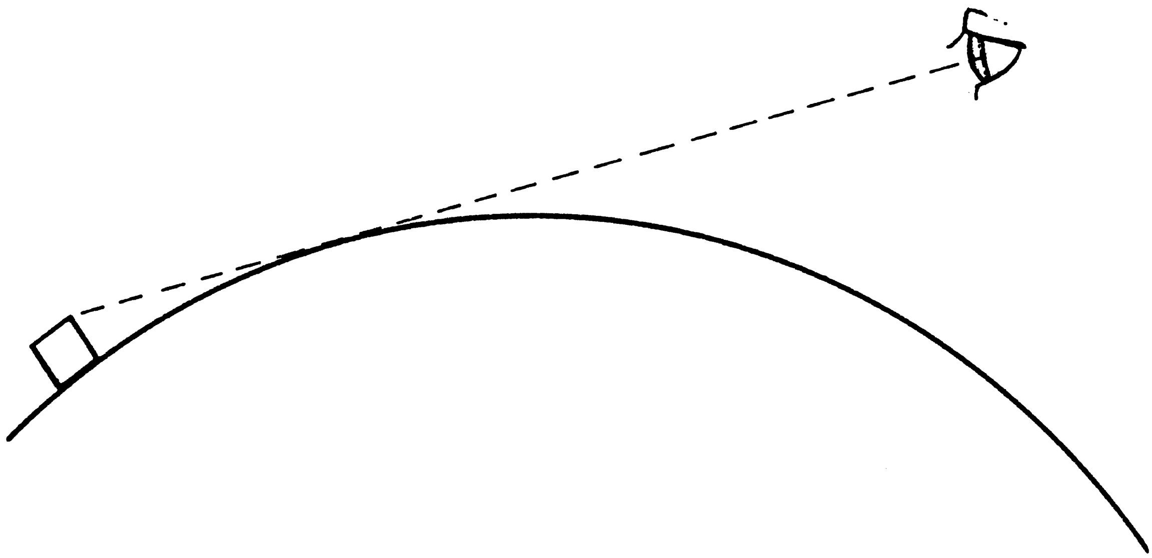


Figure C-1. Schematic representation of the start of the perception interval.

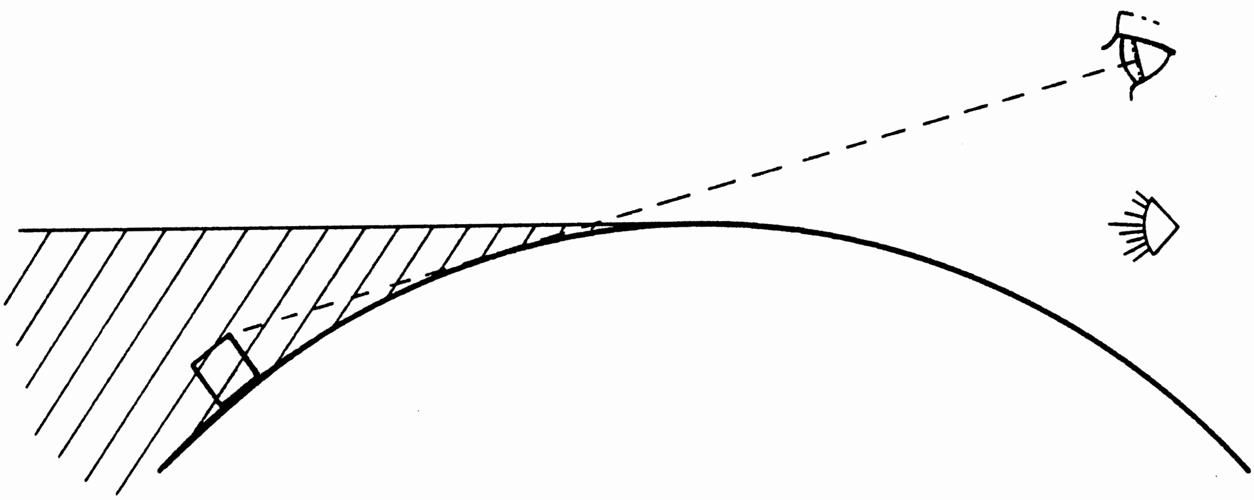


Figure C-2. Schematic representation of the start of the perception interval at night with illumination provided by the vehicle's headlamps.

In defense of using a driver eye height criterion, it has sometimes been argued that (1) less driving is done at night, and (2) people tend to drive slower at night. Both of these points have merit. However, by ignoring the problem, we are assuming, without empirical support, that these two factors compensate for what is possibly a significant loss of sight distance at night.

Another side of the question relates to headlamp performance. There are limits to the visibility that can be achieved with headlamps, and there is no point in providing additional sight distance, at appreciable cost, if the obstacle would be beyond the range of headlamp illumination.

In this section we will explore the question of visibility with headlamps. In doing so we will rely on data generated as part of a recently completed study of vehicle headlighting (43).

HEADLIGHTING STUDY

In the field evaluation conducted as part of the headlighting study, three different targets were used. One of these was a block of foam rubber 4 in. (10.16 cm) tall and 13 in. (33 cm) wide as viewed by the subject. Like the other targets, it was wrapped in blue denim to provide low contrast. Subjects had to detect the target, discriminate it from the other two, and then press one of six buttons to identify it by type and location (right or left of the car). The distance from the button press until the target was passed was recorded.

The data resulting from the procedure described are response distances. That is, the subjects had gone through the perception-response process as defined in the SSD model prior to starting the distance counters. To work back to the start of the perception process it is necessary to add a distance equal to perception-response time. Given the heightened state of alertness of these subjects, and the artificial nature of the task, the perception-response times measured in the surprise study described earlier are probably too long. However, the alerted times may be appropriate. The 95th percentile perception-response time under altered conditions was about 1.1 sec. This is the value that will be used in the comparisons later.

The data that resulted from the field visibility study are reproduced in Figures C-3 and C-4. These figures are normal probability distributions of results from six different lighting systems, including a representative U.S. low beam. Note the range of responses. With the object to the right of the car (Fig. C-3) the 5th to 95th percentile range is from about 25 to 200 ft. With the object to the left of the car (Fig. C-4) the same percentile range is about 15 to 110 ft. These data are for young subjects (i.e., < 35 years of age). Older subjects were much worse. Their response distances were about half those of the younger subjects, on average.

These data were collected under nearly ideal conditions, i.e., proper aim and voltage, all glass clean,

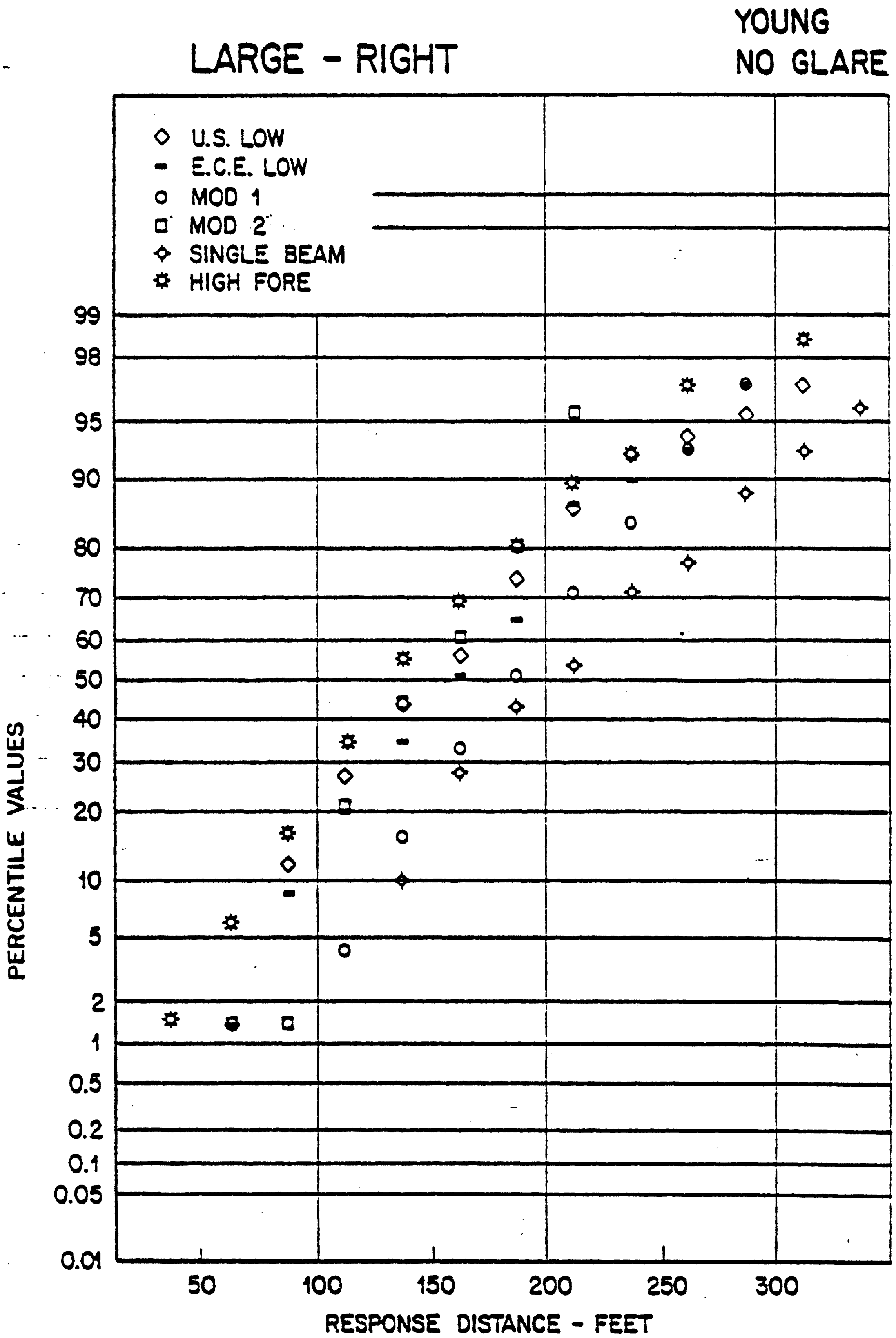


Figure C-3. Normal probability plot of six lighting systems for small targets on the right side. No glare, young subjects. (From Olson and Sivak (43)).

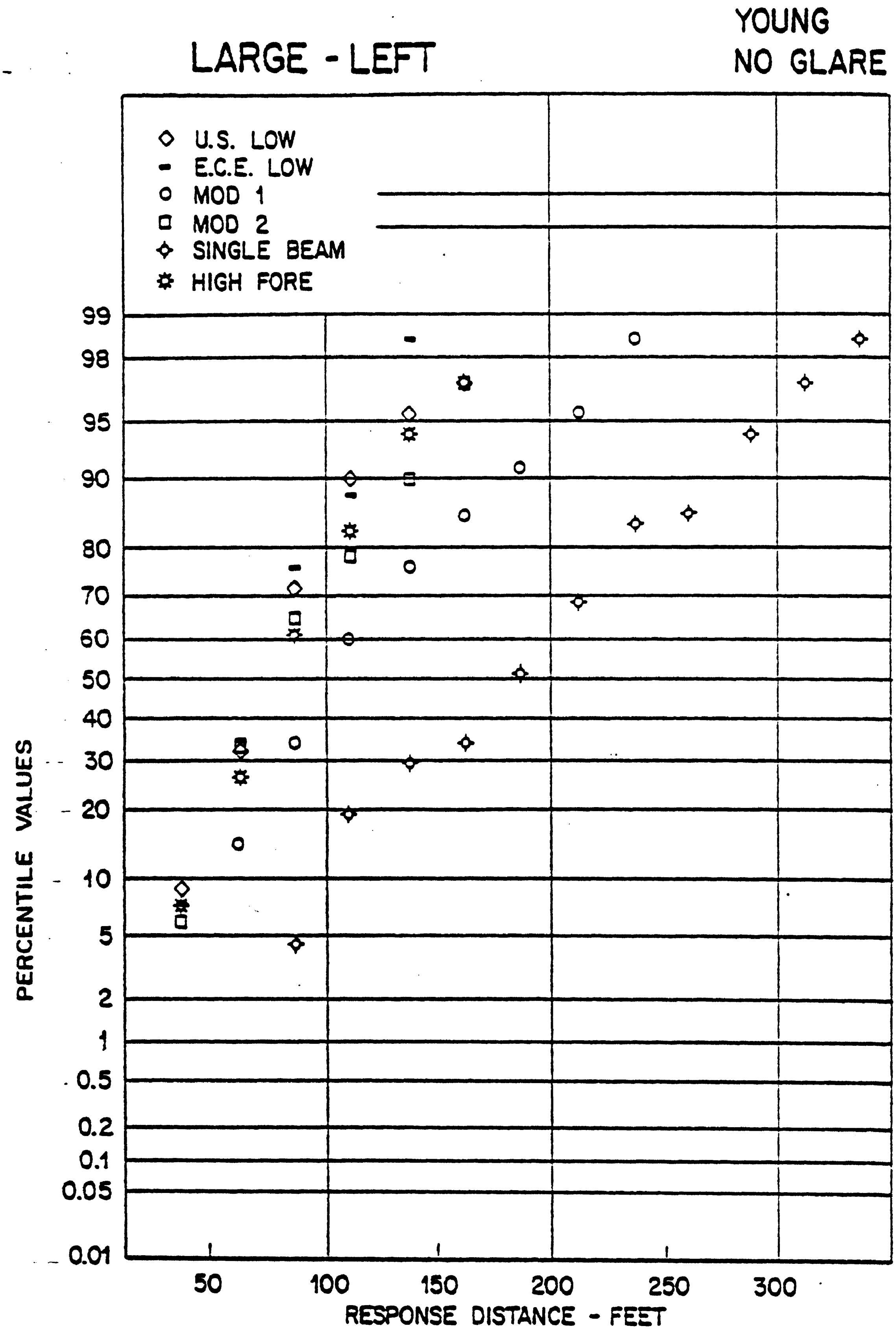


Figure C-4. Normal probability plot of six lighting systems for small target on the left side. No glare, young subjects.

clear atmosphere, no glare. Problems with any of these could significantly affect visibility.

Because the data were collected under artificial conditions, and the subjects used were aware of the purpose of the test and nature of the targets, there is a question whether and to what degree the results overestimate visibility under real-world conditions. To provide some guidance on this issue, a comparison was made between the mean response distances in this study and those recorded for the same targets in an earlier study (Olson and Abrams, (44)). In the earlier study subjects were simply told to drive a specific route on public roads and respond when they noted a "potential hazard." A variety of targets likely to be regarded as potential hazards were placed along the route by the experimenters.

Although the subjects were probably more alert than normal in the earlier study, the fact that it was run on public roads, using "natural" targets, the nature and location of which were unknown to the subject, made the resulting data more realistic than a more controlled approach. A comparison between the data for the two studies suggests that the results described in Figures C-3 and C-4 significantly overestimate real-world response distances, apparently by about 25 percent. This problem will be referred to again in the recommendations.

At the same time it must be conceded that some factors could increase visibility. For example, if the obstacle were white, rather than dark blue, the median response distance would be about twice that shown in Figures C-3 and C-4. If the car were using high beams the response distances would increase by about 100 percent for the left side obstacle and 50 percent for the right side obstacle.

In conducting the analysis to follow, a worst-case scenario was assumed (i.e., dark obstacle, low beams). It does not appear to be an unreasonable one, however. Roadway obstacles may be of any reflectivity, but certainly many are of low reflectivity. Similarly, drivers may use high beams, but are often using low beams.

The analysis required that an "effective sight distance" be determined for each condition of interest. The effective sight distance is simply the sight distance that would result if the driver's eyes were at the same height as the headlamps (24 in.). It assumes that an obstacle will not become visible until it emerges from the shadow cast by the intervening vertical curve.

The minimum AASHTO stopping sight distances, based on the 1971 blue book, which assumes an eye height of 3.75 ft, are as follows:

30 mph	200 ft
40 mph	275 ft
50 mph	350 ft

The effective sight distances for crest vertical curves for the same speed conditions, but reducing eye height to 2.0 ft are as follows:

30 mph	160 ft
40 mph	220 ft
50 mph	280 ft

For purposes of this analysis it will be assumed that the obstacle is on the right side of the lane. Thus, Figure C-3 will be used to estimate the percentile response. The response distances in Figure C-3 must be increased by an amount equal to 1.1 seconds in order to approximate the start of the perception interval. For the three speeds listed above this distance is as follows:

30 mph	48 ft
40 mph	65 ft
50 mph	81 ft

By adding the appropriate perception-response distance to the values given in Figure C-3 and noting the percentile values associated with each of the effective sight distances, it is possible to estimate the percent of drivers who will experience shorter perception-response intervals than they would otherwise because of the intervening vertical curve:

30 mph	50%
40 mph	20%
50 mph	5%

RECOMMENDATIONS

The analysis provided above suggests that using driver eye height rather than headlamp mounting height effectively reduces nighttime SSD on crest vertical curves for a significant fraction of drivers at speeds up to about 40 mph.

However, as noted earlier, the conditions assumed for the analysis, in terms of the lighting equipment, obstacle position, and driver alertness, are such as to produce overestimates of performance. Thus, the apparent deficiency at 40 mph is actually of no consequence.

If there is a problem, it appears to be at 30 mph. At this speed use of driver eye height as the vertical dimension at the vehicle will produce vertical curve profiles that will cause the obstacle to be shadowed at distances where a significant fraction of drivers would otherwise have detected it.

Just how serious this problem is from a practical point of view is uncertain. Roadways designed for 30 mph are typically in urban areas and often lighted. In addition, the speeds are low, so the consequences of a collision are less serious.

In sum, headlamp mounting height as a restriction to sight distance of crest vertical curves does not appear to be a significant problem except at very low speeds. While it is true that roadway vertical curvature will shadow an object for some time after it would have been detected under daytime conditions, headlamps simply do not have enough output on low beams to reliably reveal a low-contrast object until the car is relatively close. The minimum sight distance criterion for 30 mph does appear to present a problem for unlighted roads. It is recommended that this value be increased to 250 ft.

APPENDIX D

STUDIES OF DRIVER EYE HEIGHT

Determination of driver sight distance on a curved road surface requires a knowledge of both object height and driver eye height. The latter will clearly vary with several factors, including the vehicle type, seat characteristics and size, and position and posture of the driver. To thoroughly and accurately determine the distribution of driver eye heights for the vehicle and driver populations on today's and tomorrow's roads is therefore a complicated task involving both experimental and analytical efforts that are beyond the scope of the current project. Over the past 20 years a number of studies have been conducted to determine driver eye height in selected vehicles and to determine the present SAE J941C standard, which defines the driver population of eye positions relative to the seated H-point. In determining the driver eye height criteria for the present study these previous studies have been utilized, modified, and updated as appropriate and necessary.

Stonex (45) showed a clear trend of decreasing driver eye height from 1936 to 1957. He predicted that the average driver eye height would fall to 43 in. (3.58 ft), but would not go much lower than that value.

Stonex (46) in his second study, took a representative driver population sample that included females. He found that although the median driver eye height for new cars in the late 1950's was similar to the median for the car fleet, the 15th percentile was lower.

Meldrum (10) used a photographic technique in a static driving simulation that is still being verified in real-world driving tasks. Visitors were photographed in three 1963 automobiles looking at a straight-ahead target. With 2,300 subjects, this is the largest direct sampling ever done for eye height, and subsequent refinements in eye height placement have attempted to utilize these data with mathematical modifications to take into account variations not measured by Meldrum.

Meldrum's study consisted of three simultaneous photographs taken of the subject while seated in one of the cars looking straight ahead at a street scene. Using photogrammetry, it was possible to locate the subject's eyes in three-dimensional space. The eye distribution data approximated an ellipsoidal shape that was termed an "eyellipse." An ellipsoid could be drawn for various percentile distributions that would locate eye positions in space. These "eyellipses" became the Society of Automotive Engineers (SAE) standard SAE J941a for positioning the driver's eye in the package design.

The SAE J941 tangent cut-off ellipses are based on driver's eye locations. They represent probability contours that are the locus of straight lines, any one of which subdivides the spatial distribution of drivers' eyes into two groups: the P group and (1-P) group. For example, drawing a line tangent to the 95th percentile eyellipse will define two areas such that 95 percent of the drivers' eyes will be on the ellipse side of the line (but not necessarily inside the ellipse), and 5 percent will be on the other side.

Devlin and Roe (12) developed a procedure for modifying use of the eyellipses based on rotation of the driver's head such as would occur in actual driving situations. This led to the SAE J941b standard for eyellipse application.

The Meldrum data were all collected in vehicles that had similar seatback angles. The question was raised as to whether the SAE J941a/b standards could be used in vehicles with other seat back angles. Hammond (47) developed an "eyellipse locator line," with previously collected data from several studies using anthropomorphic test dummies in static vehicles simulating back angles from 10 to 40 degrees. The eyellipse of the J941b standard could now be repositioned based on seat back angles (J941c).

Cunigan and Abrahamson (48) took actual on-road pictures to determine eye height. They chose a fleet including pickups and sex mix that they felt resembled that of the driver population. The 15th percentile eye height was found to be at 3.5 ft, a reduction of 3 in. from Lee's 3.75-ft 1963 value. They also noted that 89 percent of compact and small cars had an eye height less than 3.75 ft, and 73 percent of intermediate and full-size cars have eye heights of less than 3.75 ft. This study is the most recent that actually gathered on-road data. The authors had a rather small sample (n=161), so confidence intervals may be quite wide. In another recent on-road study, Boyd (49) found that 68 percent of the passenger cars in the study had an eye height below 3.75 ft. He recommended that the design eye height be 3.45 ft.

Recently, Lee and Scott (50) analyzed current automobile sizes and sales for 1969 through 1979. Using eyellipse data, they found the average driver eye height to still be approximately 10 in. below the top of the auto. This value was modified to reflect the 5th percentile rather than the average, and was used with the vehicle dimensions to establish trends in driver eye height. Figure D-1 shows some of their findings. They recommended a design driver eye height of 1 m (3.28 ft, 39.4 in.) based on results shown in Figure D-2. These indicate that for the 5th percentile driver, 63.9 percent of the passenger vehicles sold in 1979 had an eye height less than 3.5 ft and 25.9 percent of the 50th percentile drivers would have an eye height of less than 3.5 ft, the current AASHTO policy (1). McGee (25) recently estimated the 5 percent eye height to be 40.2 in.

Experimental measurements in the field were determined to be beyond the scope of effort desired for this phase of the study. Therefore an analytical approach previously used and recommended by Hammond of Ford Motor Company (9) was selected. As shown in Figure D-3 this approach makes use of the SAE J941 eyellipse data, which provide vertical distances from the seating reference point (SgRP) to various population percentiles of eye height. The initial eyellipse

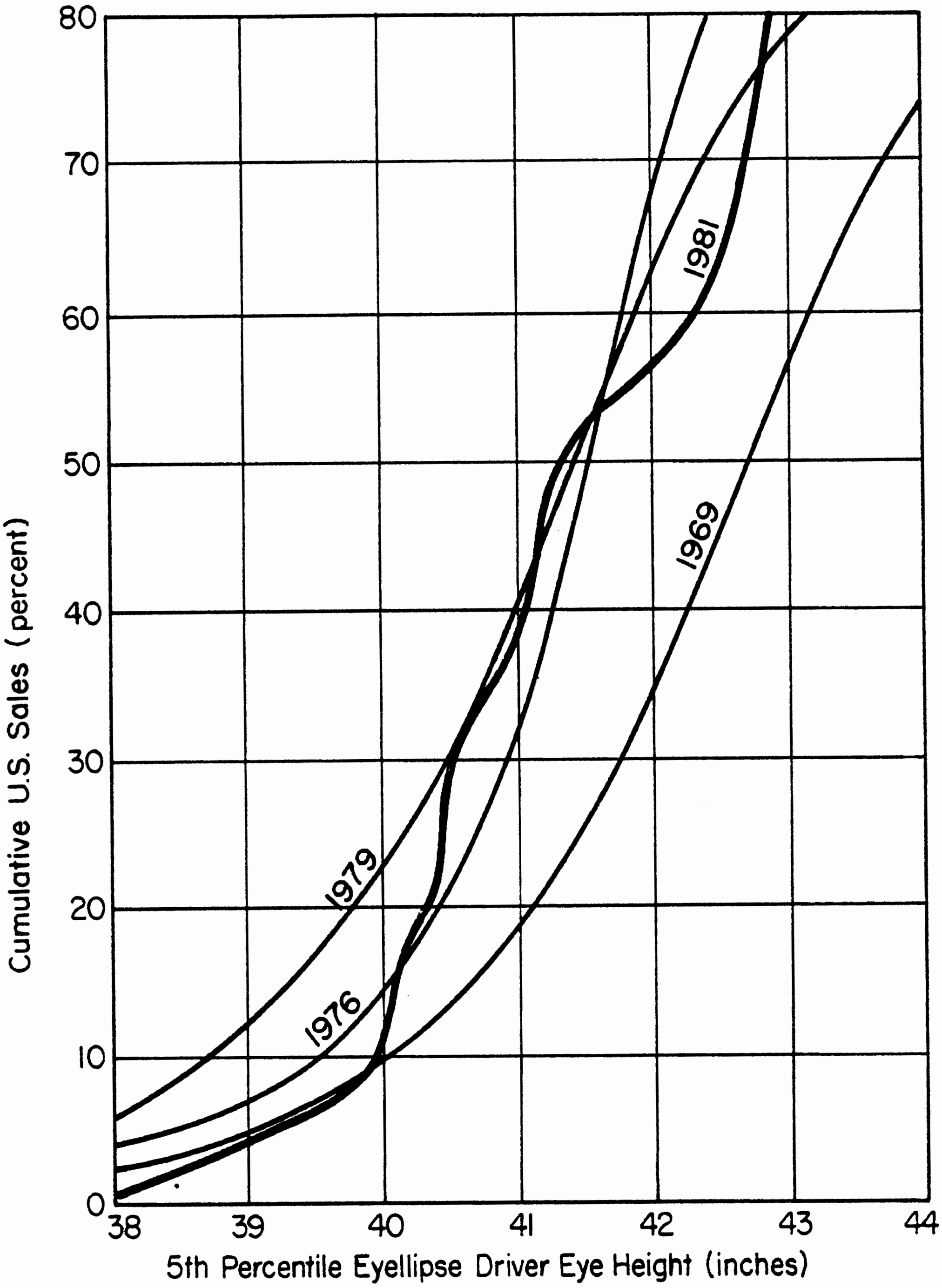


Figure D-1. Fifth percentile eyellipse driver eye heights for passenger cars sold in the United States (1969, 1976, 1979, 1981).

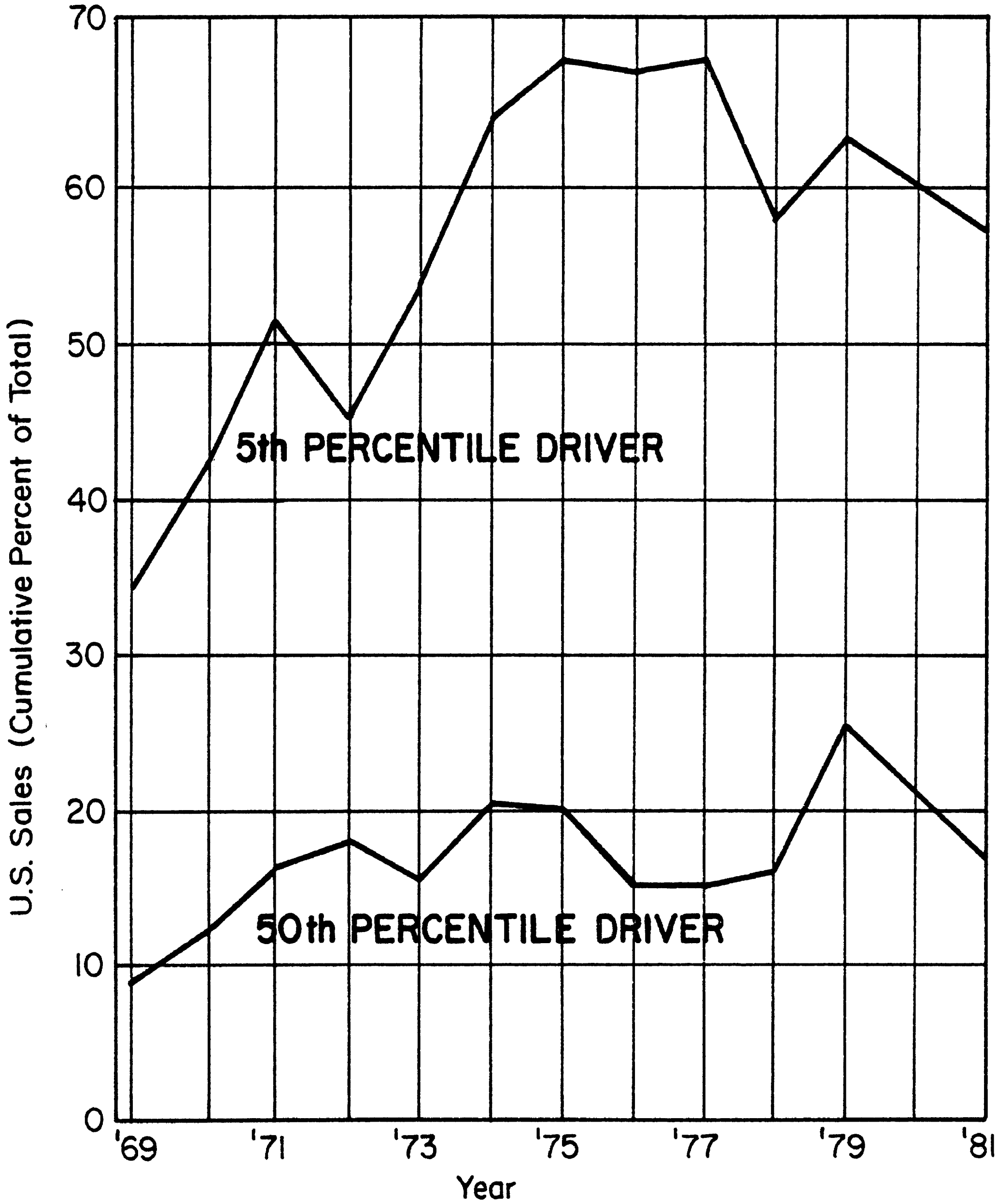


Figure D-2. Percentage of passenger cars sold in the United States with driver eye height less than 3.50 ft.

data were generated from subject measurements of actual eye position using a variety of vehicles (10). These data have been modified and improved over the years to account for differences in seat back angle (11) and head rotation (12) Essentially, the eyellipse is a locus of tangent cut-off points, such that for any line tangent to the 95th percentile eyellipse, for example, 95 percent of the eyes will be located on the ellipse side of the line (but not necessarily within the ellipse) and 5 percent will be on the other side of the tangent line. For our purposes, it was desired to determine the vertical distance from the SgRP to various percentile eyellipses (see Fig. D-4). This was accomplished by determining the distance from the SgRP to the horizontal tangent line to the various percentile eyellipsoids as illustrated in Figure D-4. Table D-1 shows the resulting percentile values for this distance.

In order to determine the eye height to ground measure desired, it is necessary to add to the SgRP to eyellipse distance, the distance of the SgRP to the ground This measure is vehicle specific and is not directly available. As recommended by Hammond and illustrated in Figure D-3, this measure was calculated from other vehicle dimensions as follows:

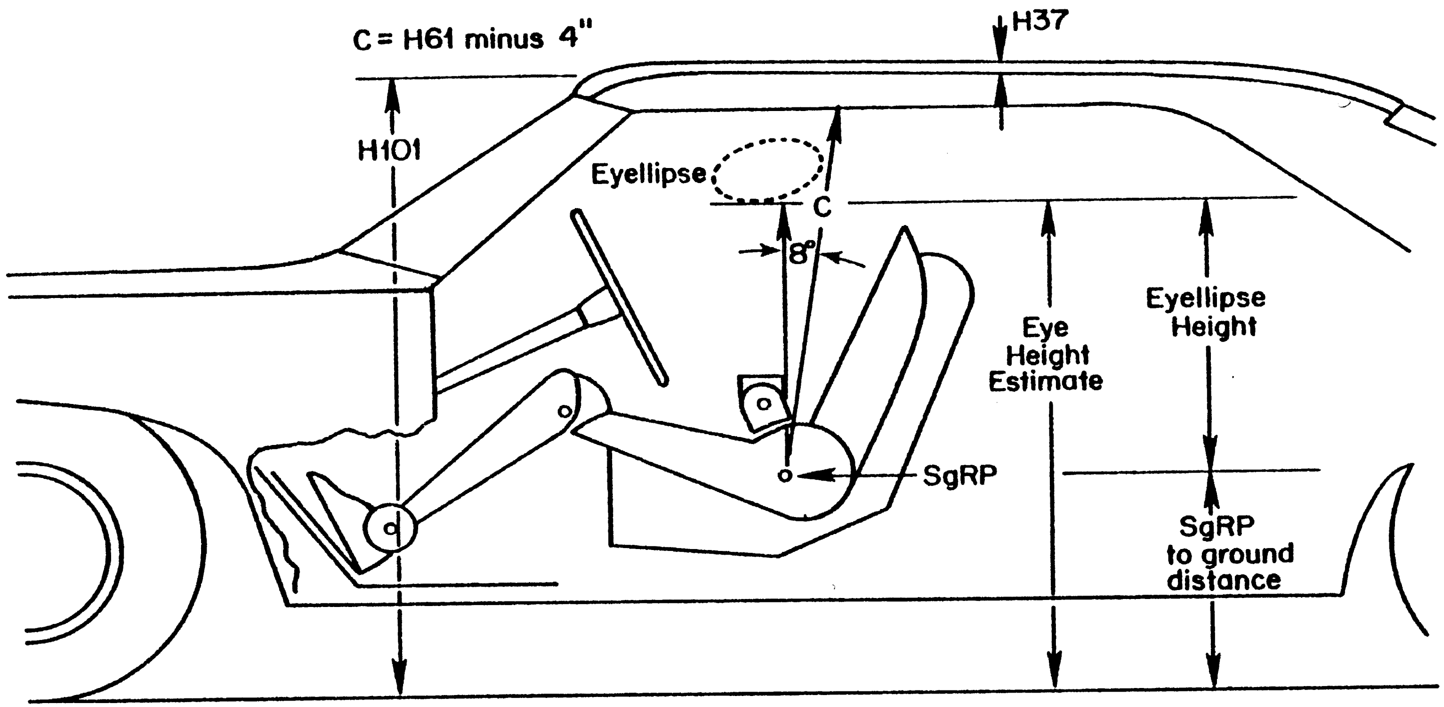


Figure D-3. Illustration of specifications used to estimate SgRP height and driver eye height.

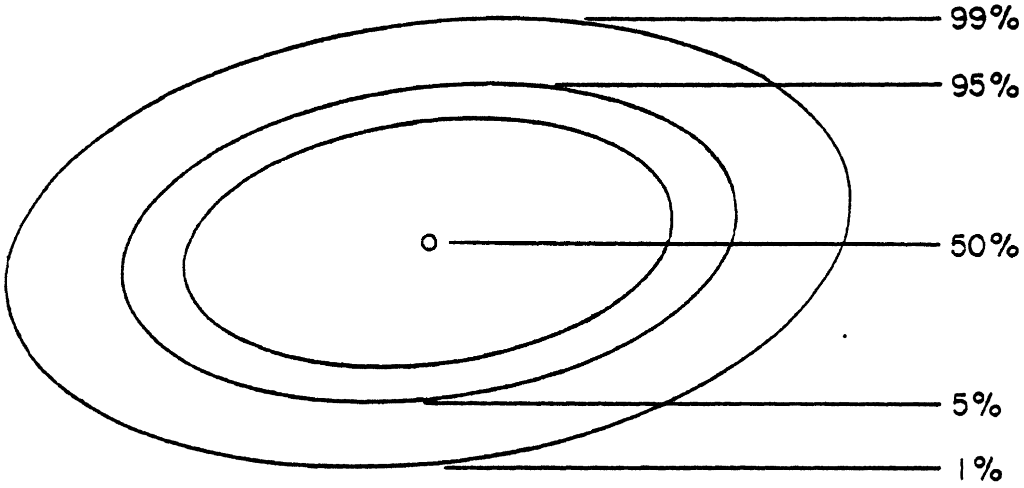


Figure D-4. Percentile tangent lines used in measuring vertical distance of eyellipse to SgRP.

$$\text{SgRP to Ground} = H1091 - (H61 - 4") * \cos 8^{\circ} - a$$

where:

H101 = vehicle height, as defined by SAE, in the "design attitude," i.e., loaded with fuel, passenger, etc.;

H61 = effective head room, a measurement along a vector 8 deg rear of vertical from SgRP to the vehicle headlining, plus 4 in.;

a = adjustment factors estimated at 1.5 in. which include curvature of roof, headlining thickness, seat track adjustments and varying back angle.

Ground to SgRP distances were calculated for all vehicle models (both foreign and domestic), which comprise a measurable volume of the current sales market in the United States as sold in 1981 according to Wards Automotive Yearbook, 1982. These SgRP heights were ordered from smallest to largest and a cumulative summation of all models sold was made from this list. SgRP heights for various selected percentiles (e.g., 5th, 10th, 20th, etc.) were calculated by linear interpolation between these cumulative percentages. Table D-2 gives the resulting percentile values of SgRP heights above the ground and Figure D-5 shows the histogram.

The distribution of eye height to ground distances along with various percentiles of driver eye height was calculated by assuming independence of the two distributions

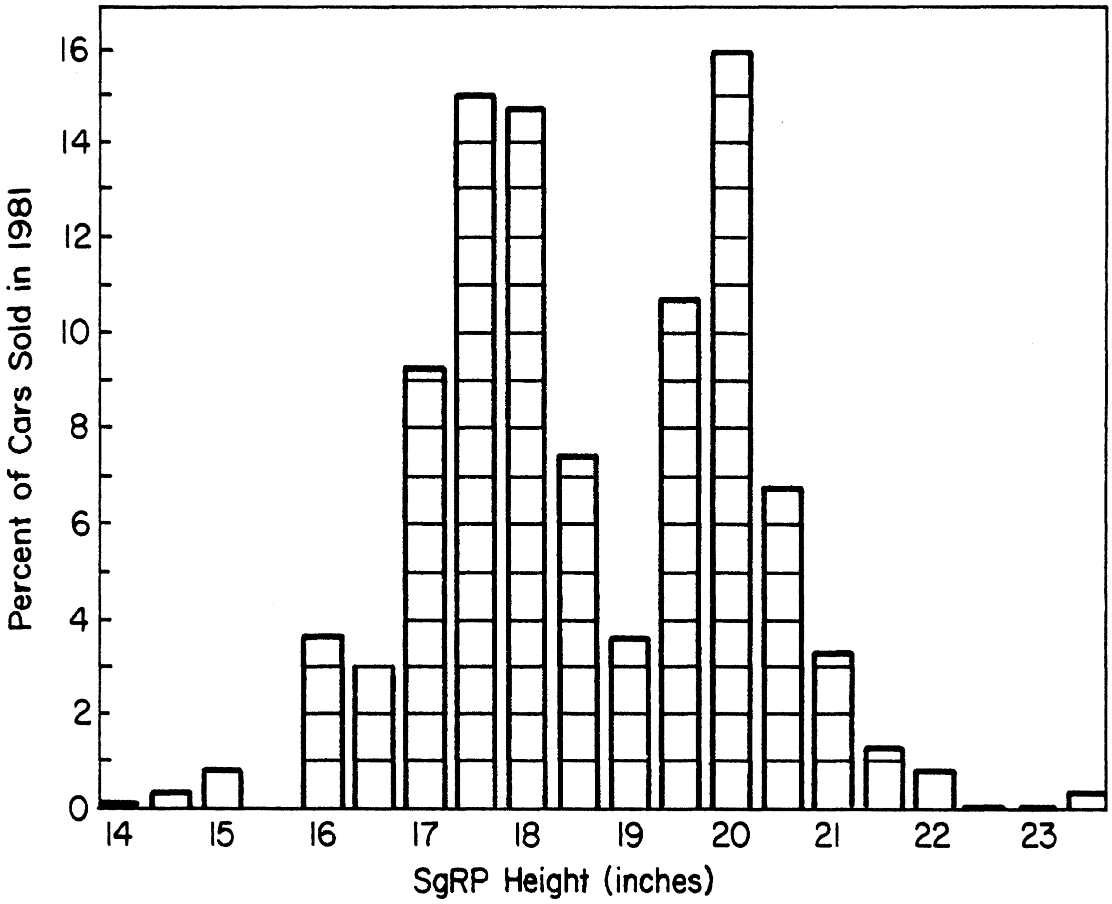


Figure D-5. Histogram of SgRP height of cars sold in 1981.

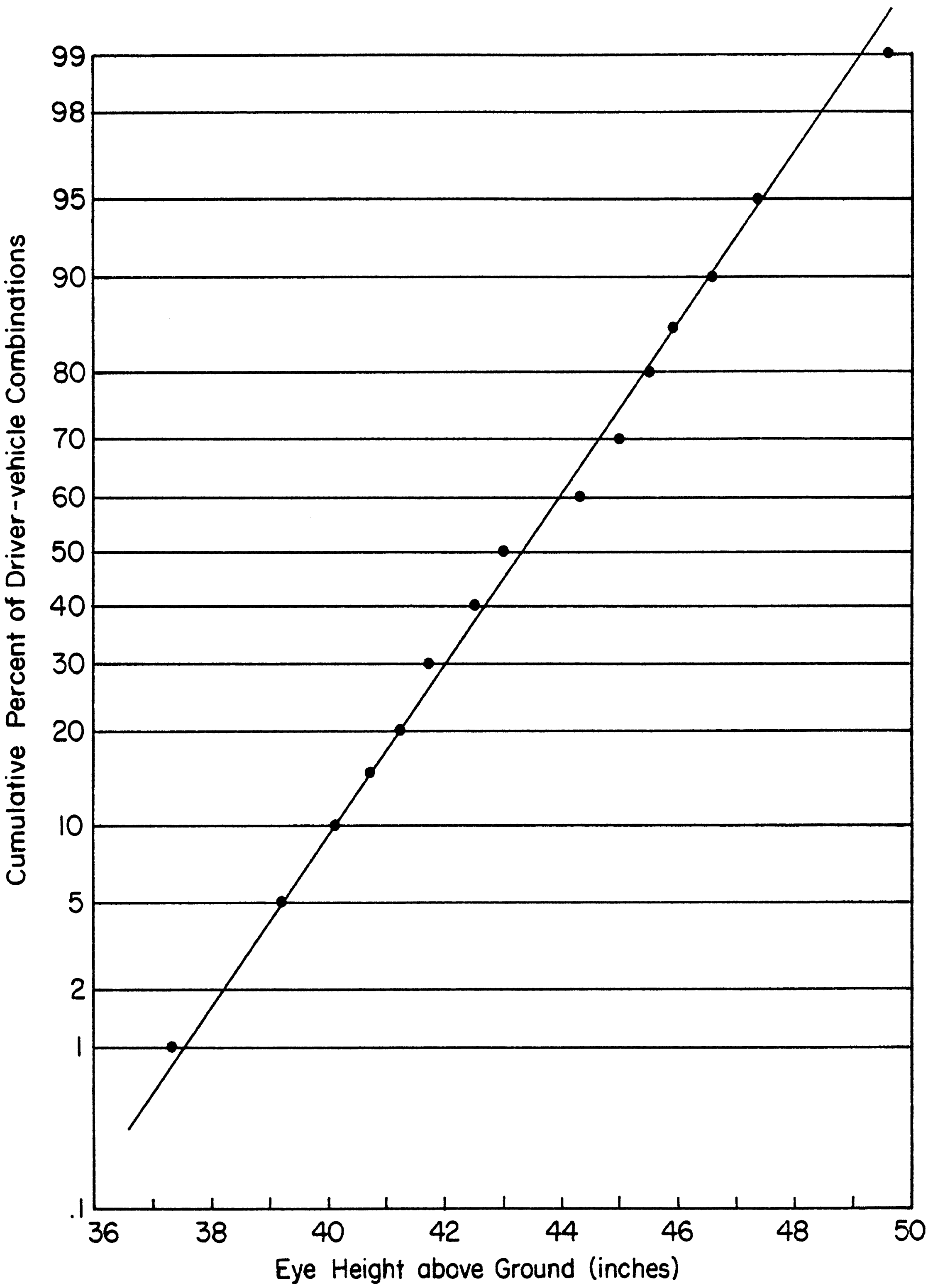


Figure D-6. Cumulative distribution of eye height above ground for driver-vehicle combinations.

Table D-1. SgRP eye heights cumulative percentile eye heights above SgRP.

SgRP Eye Height (in.)	Percentile
22.2	1
23.0	5
23.3	10
23.6	15
23.8	20
24.1	30
24.4	40
24.7	50
25.0	60
25.3	70
25.6	80
25.8	85
26.1	90
26.4	95
27.7	99

Table D-2. SgRP Heights--1981 sales.

Ground SgRP Height (in.)	Percentile
15.1	1
16.2	5
16.8	10
17.1	15
17.4	20
17.6	30
18.1	40
18.3	50
19.3	60
19.7	70
19.9	80
20.1	85
20.5	90
21.0	95
21.9	99

and appropriately combining them using the percentiles in Tables D-1 and D-2. It was found that this distribution is nearly normal as shown in Figure D-6. The resulting percentiles given in Table D-3 provide an estimate of driver eye height based both on 1981 car fleet characteristics as given by SgRP to ground data and driver population characteristics based on eyellipse to SgRP distance. These results have been added to Figures D-1 and D-2.

An estimate of SgRP height was then made for late in the decade. Projected sales for 1990 in small and large car categories were obtained (14). These data were based on weight of vehicles, and the 1981 vehicle weights were obtained (15). The 1981 distributions of SgRP were then weighted by small and large car fractions and used to develop an estimate for this factor for late in the decade.

Table D-4 presents the SgRP data for the 1990 estimate. Comparing Table D-4 with Table D-2 reveals that changes from 1981 sales will affect the eye heights given in Table D-3 by less than 0.2 in. Accordingly, the 1981 values are adopted as the estimate of this study.

Table D-3. Eye heights for 1981 passenger cars.

Eye Height (in.)	Cumulative Percentile
39.0	1.1%
40.0	3.8
41.0	11.0
42.0	24.9
43.0	43.5
44.0	64.1
45.0	78.8

Table D-4. SgRP heights 1990 cars.

Ground SgRP Height (in.)	Cumulative Percentile
15.1	1
16.3	5
16.9	10
17.1	15
17.3	20
17.7	30
18.1	40
18.5	50
19.0	60
29.4	70
19.9	80
20.2	85
20.6	90
21.0	95
21.9	99

APPENDIX E

HIGHWAY SAFETY, SPEED AND GEOMETRIC DESIGN

SAFETY STUDIES

This section describes a study of the effects of SSD on safety. A review of previous work is summarized, and the methodology, data, and analyses are presented.

Previous Studies

There have been a number of studies of the relationship between sight distance and safety. However, it is extremely difficult to maintain proper controls and many of the results are not well supported. A recent synopsis by Jorgensen (51) concluded that the vehicle-mile accident rate decreases as the available sight distance increases. Jorgensen cited studies on two-lane and multilane highways, at rural and urban locations, bridges, intersections, interchanges, and railroad grade crossings to support these conclusions. SSD research has been summarized most recently by the FHWA. (Bissell et al., (52)). The FHWA review of reported research described none that specifically attempted to relate SSD to accident experience. However, it was concluded that most accidents occur on straight-level surfaces and that curves have been found to be more hazardous in terms of vehicle-mile exposure.

Among specific studies that have used SSD as an input to an accident prediction model, Behnam and Laguras (53) found that SSD, horizontal curvature, and road grade were the best predictors of accident frequency at bridge locations with wet pavement conditions.

Most of the work done with SSD was concerned with horizontal curvature. SSD often varies with sharpness of horizontal curvature. Shah (54), Kipp (55), and Baldwin (56) all found that as the degree of horizontal curvature increased the accident rate increased for curves with safe speeds of 50 mph or greater. Smith (57) recently reported an analysis of horizontal curves with high accident rates in which increasing sharpness was associated with higher accident rates. In another early study, Raff (58) found contrary results for rural highways with an ADT less than 5,000.

Gupta and Jain (59) and Dale (60) reported that significant accident reductions followed improvements in horizontal alignment and visibility. Dale found a 40 percent reduction in accidents and a greater than 50 percent reduction in injuries following horizontal alignment changes. Sparks (61) studied 7,500 accidents on 12,000 m of rural Oklahoma highways and found little safety relationship with geometric design features, including sight distance.

Both Bitzel (62) and Kihlberg and Tharp (63) found that particularly dangerous roadway sections involved combinations of steep grades and horizontal curves. Hillier and Wardrop (64) suggested that the effects of uphill grade along with poor sight distance contributed to accident occurrence.

Gupta and Jain (59) found that restricted sight distance had a significant effect on single vehicle accidents in both urban and rural areas and on multiple vehicle accidents in urban areas. Mullins and Keese (65) reported that rear-end accidents on freeways were prevalent at high-accident-frequency vertical curve locations where there were unfavorable sight conditions. Both Schoppert (66) and Baldwin (56) reported correlations between sight distance and accident experience. Raff (58) explored accidents for the density of sight distance restricted sections where the critical value was 400 ft in mountainous terrain and 600 ft elsewhere (eye and object height used were not indicated). He found that the vehicle-mile accident rate increased approximately 50 percent as the number of such restrictions range from none up to 6 per mile.

Earlier, Hilts (67) had observed a contrary finding. However, Baldwin (56) reported that the relatively infrequent sight distance restriction produces a higher incidence of accidents. Tanner (68) found that visibility improvements at five English curves resulted in a 65 percent reduction in injury accidents. Young (69), in an early California study, reported that the vehicle-mile accident rate on 500 mi of two-lane roads decreased by more than 50 percent where sight distance increased from 800 to 2500 ft.

The Cirillo et al. (70) FHWA study on the Interstate freeway system provides a primary modern-day support for the desirability of increased sight distance from a safety viewpoint. It was concluded that an increase in SSD was associated with a decrease of about one accident per year for each 4,000 ADT.

A problem with all of these studies is the obvious correlation that exists between sight distance and alignment. Where roads have no horizontal or vertical curvature, available SSD is unlimited. Recent analyses of the effect of sight distance separate from curvature have not been found.

Project 15-8 Methodology

The objective of this study was to isolate the effect of available SSD on safety. A study design where the effects of alignment would be controlled was proposed. This design called for the comparison of a figure of merit of safety for a set of road segments with limited available SSD relative to the 1965 AASHTO standard with that of similar sites with adequate available stopping sight distances. The road segments were to be tangent sections, thus eliminating all effects of horizontal curvature. The effects of vertical curvature were to be controlled by selecting pairs of crest vertical curves matched for traffic volume, abutting land use, lane widths, shoulders, and ditches as well as the same algebraic difference in grades. The only difference between the two sites in each pair would be the length of curve and thus a difference in the available SSD. The figure of merit of safety selected was the police-reported accident count over a period of several years on these road segments. Since all traffic and design factors at each pair of sites with the exception of the length of the vertical curve were the same, any significant difference in accident counts could be attributed to the difference in length of the vertical curve as the controlling factor of the available SSD. However, in actual practice it was impossible to identify such a set of matched sites, and the requirement that the matched sites be identical with respect to the algebraic difference of the grades was relaxed. The control sites would be matched to the limited available SSD sites for traffic volume, abutting land use, lane widths, shoulders, and ditches. The available SSD on the limited available SSD sites would be below that recommended by AASHTO in 1965, and the available SSD on the control sites would more than meet the 1965 AASHTO standard. Although it is recognized that in such a study the accident experience difference between the limited available SSD and control sites would be attributable to the differences between the sites with respect to the available SSD and to the grades, the sampling problem in the original study made the modification of the study design necessary and the inseparability of SSD from grades supported proceeding with the modified study design.

Sites on paved two-lane rural roads in Washtenaw and Oakland Counties, Michigan, where limited sight distance signs were posted were selected as candidates for this study. After review of photologs and site visits, road segments where the sight-distance limitation was attributable to horizontal curvature or hidden driveways were eliminated, leaving only road segments with restricted sight distance due to crest vertical curves. Any sites that were near horizontal curves were further eliminated. Only locations where the vertical alignment had not been changed during the last 7 years were retained. For each limited available SSD segment, another segment of the same road was identified as a matching control segment. Each control segment was the same length as the test segment, was within 1 mi of, but not adjacent to, the limited available SSD site, nor was it near a horizontal curve. The traffic volumes, abutting land use, vegetation, road geometry, lane widths, and shoulders were the same for the limited available SSD segment and its matching control segment.

A check was made on the reconstruction history of these segments. The vertical alignment of these sections had not been changed in the past 7 years. Such changes as pavement edge or shoulder improvements, and pavement markings occurred at the same time as on the matched site. The available SSDs were measured at each limited SSD site and at its control. An object height of 6 in. and an eye height of 3.75 ft were used. The final sample consisted of 10 matched pairs of sites with an average length of 0.30 m. Table E-1 presents the description of the limited available SSD sites and their controls.

Table E-1. Description of the limited SSD and control site pairs.

Site Pair	Site Type	Length (mi)	Speed Limit (mph)	Advisory Signs	Min. available SSD (ft)
OAKLAND COUNTY					
1	LSD*	.50	45	LIM. SIGHT DIST. 40 MPH	118
	CONTROL	.50	45		>700
2	LSD	.23	50	LIM. SIGHT DIST. 40 MPH	276
	CONTROL	.23	50		536
3	LSD	.40	50	LIM. SIGHT DIST. 25 MPH	188
	CONTROL	.40	50		>700
4	LSD	.25	45	LIM. SIGHT DIST. 30 MPH	174
	CONTROL	.25	45		>700
5	LSD	.22	45	LIM. SIGHT DIST. 30 MPH ROUGH ROAD 30 MPH	263
	CONTROL	.22	45	ROUGH ROAD 30 MPH	>700
6	LSD	.25	45	LIM. SIGHT DIST. 30 MPH	250
	CONTROL	.25	45		>700
7	LSD	.24	45	LIM. SIGHT DIST. 35 MPH	262
	CONTROL	.24	45		>700
WASHTENAW COUNTY					
8	LSD	.15	50	LIM. SIGHT DIST. 40 MPH	308
	CONTROL	.15	50		>700
9	LSD	.17	50	LIM. SIGHT DIST. 40 MPH	280
	CONTROL	.17	50		>700
10	LSD	.20	25	LIM. SIGHT DIST.	223
	CONTROL	.20	25		>700

*LSD; Limited SSD Site

The county of the site, length, speed limits, advisory signs, and the minimum available SSD are given.

Accident records for each limited available SSD and control segment were obtained from the Washtenaw County Road Commission and the Traffic Improvement Association of Oakland County. The number of accidents at the Washtenaw County sites were counted from 1977 to 1982. The number of accidents at the Oakland County sites were counted from 1978 to 1983. The only exception to this is site seven in Oakland County where the segment with limited sight distance was widened in 1982. For this site the accident counts reported here are from 1978 to 1981. The data cover 30.28 mi-yr. The total number of accidents is 136. Of these, 82 accidents occurred on the limited available SSD sites and 54 accidents occurred on the control sites. Table E-2 presents the summary of the accident data collected.

A comparison was also made to determine whether there was an obvious pattern in the types of accidents occurring on the limited available SSD sites and the matching controls. None was found.

Analysis

The accident counts on the sites with sight distance limitations and on the matched control sites were analyzed by standard contingency table techniques. Hypotheses were tested concerning whether the accident count on the limited available SSD segments was different from the count on the control segments, whether the proportion of accidents

Table E-2. Number of accidents at limited SSD and control sites.

Pair	LSD*	Control	County
1	11	3	Oakland
2	1	0	Oakland
3	2	2	Oakland
4	7	1	Oakland
5	13	6	Oakland
6	17	26	Oakland
7	24	13	Oakland
8	5	2	Washtenaw
9	2	1	Washtenaw
10	0	0	Washtenaw

* Limited SSD Site (LSD)

occurring on limited SSD or on control sites was independent of the county of site location, and whether the total accident experience considering pairs of sites was different.

Table E-3 shows the test of the hypothesis that the total number of accidents in the study period on the limited SSD sites and the control sites is the same by the standard χ^2 test. The hypothesis was rejected.

Table E-4 shows the test of the hypothesis that the number of accidents during the study period on the limited SSD and control sites is independent of county of site location, by Fisher's exact test. The hypothesis of independence was not rejected.

The next hypothesis tested whether there is a difference in accident experience during the study period between each site in a limited available SSD and matched control pair. Table E-5A presents accident counts expected under H_o. Some sites have been randomly grouped together so that the expected number in each cell is at least 5, which is necessary for a meaningful χ^2 test. Since the previous test shows that there is no county effect in the proportions of accidents in limited available SSD and control sites, sites from both counties could be included in such a group. Table E-5B shows the observed accident counts for the same groupings of sites and the results of the χ^2 test. The hypothesis of no difference was rejected.

Table E-3. Total accidents by type of site.

LSD	Control	Total
82	54	136

H_o: The total number of accidents in the study period on the limited available SSD sites (LSD) and control sites is the same.

$\chi^2 = 5.76 > 3.81$

df = 1

Reject H_o at $\alpha = 0.05$

Table E-4. Number of accidents by type of site and county.

	LSD	Control	Row Total
Oakland	75	51	126
Washtenaw	7	3	10
Column Total	82	54	136

H_o: The number of accidents during the study period on the limited available SSD (LSD) and control sites is independent of county of site location.

p value for rejection of H_o using Fisher's exact probability of observed arrangement = 0.222

Therefore do not reject H_o at $\alpha = 0.05$

Table E-5A. Accidents by site and type of site expected under Ho.

Site(s)	LSD	Control	Row Total
1	7	7	14
2,3,4	6.5	6.5	13
5	9.5	9.5	19
6	21.5	21.5	43
7	18.5	18.5	37
8,9,10	5	5	10
Total	68	68	136

H_o: There is no difference in accident experience during the study period between each site in a limited available SSD and matched control pair.

Table E-5B. Observed accident counts by site number and type of site.

Site(s)	LSD	Control	Row Total
1	11	3	14
2,3,4	10	3	13
5	13	6	19
6	17	26	43
7	24	13	37
8,9,10	7	3	10
Column Total	82	54	136

$\chi^2 = 17.67 > 12.59$

df = 5

Reject H_o at $\alpha = 0.05$

The results of the analysis indicate that there is a significant (α=0.05) difference between the accident counts at the set of sites with limited available SSD and the matched set of control sites. The accident experience at the sites with limited available SSD and their matched control sites is independent of the county of location at the 0.05 level. The hypothesis that there is a significant difference in accident occurrence between each available limited SSD site and its matching control site could not be rejected at the 0.05 level. Sign tests were also conducted and they support these findings.

From this analysis it can be concluded that there are significantly fewer accidents at sites where available SSD is not limited and meets the 1965 AASHTO standard than there are at sites that are similar in every respect except for a crest vertical curve and an available SSD below the 1965 AASHTO standard.

Although the sample of sites is not large, the data cover 30.28 mi-yr. Great care was taken in identifying the sites and in matching the control sites. There are no effects of horizontal curvature. Effects of traffic volume, road design, roadside features, and land use on accident occurrence are identical at each pair of sites. If there are any accident reporting biases, they should be consistent for both the limited available SSD and control sites. Thus the differences in accident experience can be attributed to the effect of the available SSD and the differences in vertical alignment at the pairs of sites. In this sample the sites with SSD meeting the AASHTO 1965 Standard experienced 1/3 fewer accidents during the study period than did sites with limited available SSD.

SPEED STUDIES

Two studies concerned with speed distributions of vehicles on rural roads were carried out. The first study was concerned with the nature of the distribution of speeds on rural roads during daylight, with the objective of providing a probabilistic description of speed distributions should it be required for the safe stopping sight distance model. The second study investigated the differences between speed distributions on wet and dry pavements. It was once assumed that highway speeds were somewhat lower on wet pavements than on the same pavements in dry conditions (5). It is now generally accepted that many operators drive as fast on wet pavements as they do on dry surfaces (6, 1) It was the objective of the second speed study to determine if speed distributions are significantly different under wet and dry pavement conditions during the day.

Speed Distribution Study

Data for the analysis of speed distributions were obtained from the State Departments of Transportation of Illinois,Indiana, Kentucky, and West Virginia. These data had been collected for the FHWA Speed Monitoring Program for the National Maximum Speed Limit (75). Although these data are only collected on highways with 55-mph speed limits, they are obtained by a standardized procedure (71) and provide a large bank of recent information on the behavior of drivers.

Only data from locations that are called "control locations" in the speed monitoring program were used. These are sites that are sampled every quarter of a year, and more detailed information about speeds is collected at these sites than at other types of sites.

The site locations are selected by a sampling plan detailed in the FHWA Speed Monitoring Manual (71) and are free from features that encourage or discourage vehicle speeds. The speed monitoring sites are not near or on sharp horizontal curves, on grades greater than 4 percent, or near important intersections. The speed data are not collected under extreme weather conditions or in the presence of nonroutine enforcement, construction, maintenance or other activity that may affect the speed of vehicles passing the site. For the purposes of the speed monitoring study, highways are classified into six categories. Only the three rural categories are of interest to this study; thus only data for the rural Interstate, rural principal and minor arterials, and rural major collectors were acquired.

Because this study was concerned with daytime visibility it was important to get daytime speed distributions. Inasmuch as some states collect the speed information totaled over a 24-hour period, such daily distributions would necessitate making many assumptions about which portions of traffic traveled during daylight. Thus, only data from locations where hourly speed distributions were available were used. To ensure that the speeds were from daylight conditions only, speeds between 8 AM to 5 PM were used. The speed data were available as vehicle counts in 5-mph speed intervals.

Hourly speed distributions from a sample of 106 sites from the available data were tested for normality. Table E-6 presents the number of sites and hours from each road category by the State of location. Over 900 hourly distributions were plotted, and 90 distributions were formally tested for normality by the chi-square test. The tested distributions could be considered normal at the 5 percent level. The remaining speed distributions were compared visually to those that had been formally tested. They were very similar and it was concluded that normality was also a reasonable assumption for these remaining distributions.

Figure E-1 shows an example of one such hourly speed distribution. Shown is the cumulative speed distribution for one hour from an Interstate highway site in Kentucky, where 698 vehicles were observed. The theoretical normal distribution is also presented for comparison.

Table E-6. Sample of hourly speed distributions.

State	Interstate Sites	Arterial Sites	Collector Sites	Sites in State	Total Hours
Illinois	8	5	1	14	126
Indiana	9	4	2	15	135
Kentucky	12	16	15	43	387
West Virginia	11	17	6	34	306
No. of Sites in Road Category	40	42	24	106	954

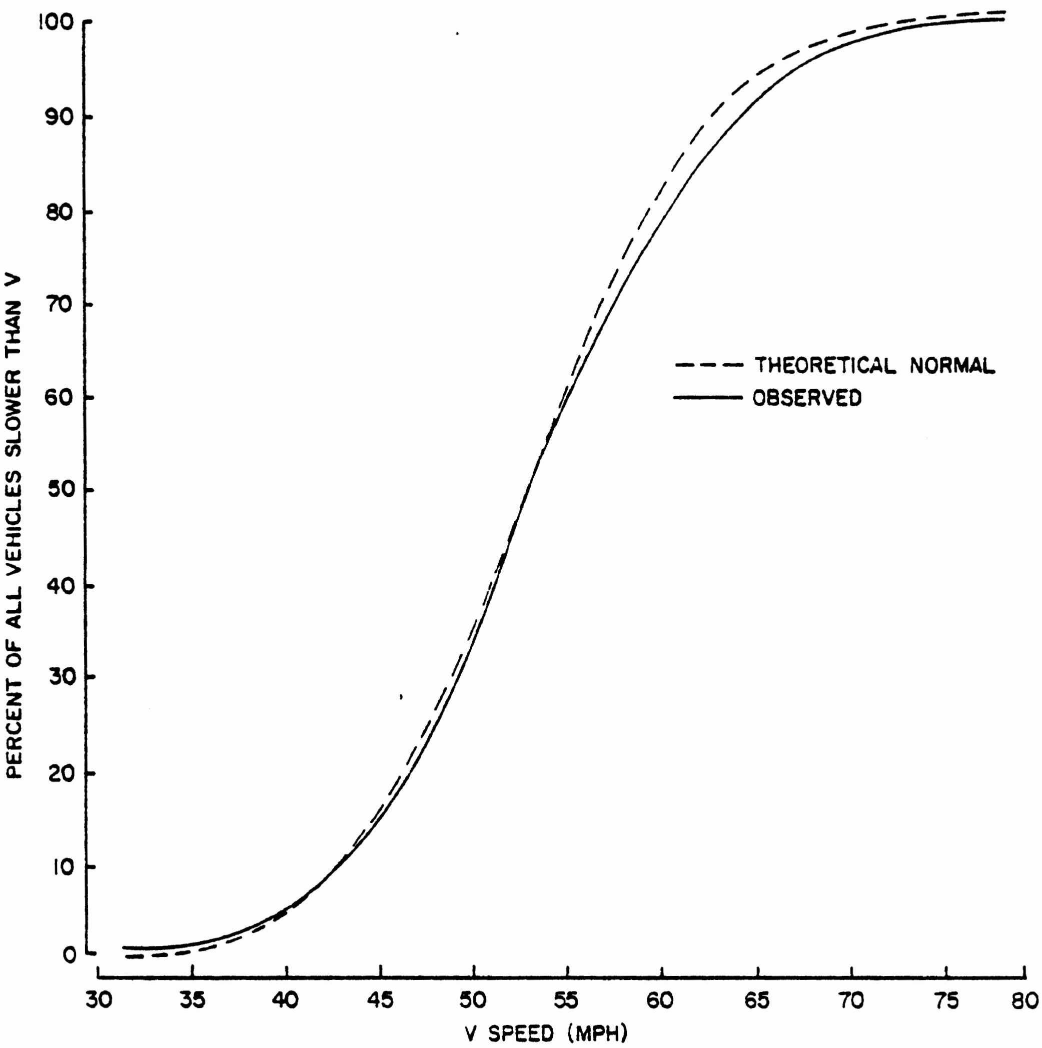


Figure E-1. Cumulative speed distribution from Inter-
state highway site for one hour.

The normality of the distributions allows the analysis of variance. Analysis of variance on subsets of the data identified site effects within the three categories of rural highways. This of course is not surprising, considering the many possible differences in sites within such broad categories.

The results of this analysis indicate that the normal distribution is a good assumption for speeds on rural roads. Specific distributions for the three road categories were not obtained since these categories were much too broad for their distributions to be captured by a single distribution.

Speeds on Wet and Dry Pavements

The next analysis was concerned with investigating the difference between speed distributions on wet and dry pavement conditions. For this analysis it was necessary to have reliable information about the weather at the speed monitoring site. All speed data are from April and May of 1983 and are limited to Illinois. The Meteorological and Oceanographic Sciences Department in the University of Michigan's College of Engineering provided the research team with several dates over the past year for which the probability of rain over a large portion of Illinois for most hours of the day is high. Similarly, dates with a low probability of rain were also provided. Since the State of Illinois maintains permanent counting stations which continuously monitor speeds, data are available for each day, and the speeds on wet pavements may be conveniently compared to speeds on dry pavement on "nearby" days.

Table E-7 gives the number of sites, wet and dry days, and wet and dry site hours used in this analysis. It should be noted that speed distributions for all the sites were not available for all the days. This is reflected in the number of site hours for the highway category.

The hourly distributions for each of the sites were tested for time-of-day effects by one way analysis of variance. Table E-8 shows an example of the test for time-of-day effects in the speed distributions for one of the sites from Illinois for two wet and two dry days. There were no significant time of day effects at this site and at the other sites tested.

Since there were no significant time-of-day effects, the speed data at each site for one day were aggregated for further analysis. Thus at each site the speed distributions for the daylight hours of a day would be compared with similar data from other days. The days are identified as wet or dry days.

Table E-9 presents the speed summaries for the seven Interstate highway sites at which data were collected. For each site, for each day the wet/dry condition is identified, and the number of vehicles in the sample is given. The mean and standard deviation as well as the 15th, 50th, and 85th percentile speeds are given. The same statistics are summarized for all wet and all dry days for each site. The

Table E-7. Sample for speed comparison on wet and dry pavements.

Highway Category	No. of Sites	Maximum No. of Days at Site *		No. of Site Hours	
		Wet	Dry	Wet	Dry
Interstate	7	2	2	126	126
Arterial	15	3	3	351	342
Collector	4	3	3	90	81

Data were not available for all sites for all days.

Table E-8. Testing for time of day effects of speed distributions at a site by analysis of variance.

Hour	9	10	11	12	13	14	15	16	17	p-Value for Rejection
Dry Day 1	94 61.9 8.3	78 60.3 10.1	47 62.0 6.5	34 63.5 6.9	82 60.8 6.7	119 62.9 7.3	176 61.7 6.1	114 62.3 7.0	77 62.1 8.6	.2770
Dry Day 2	95 61.8 8.4	79 60.2 10.2	89 58.4 10.0	46 60.2 8.6	71 61.3 9.0	114 62.3 8.7	106 60.9 8.2	196 61.9 6.8	143 61.3 8.6	.1088
Wet Day 1	96 61.0 7.7	67 61.3 8.0	29 61.2 6.5	24 60.6 6.6	63 61.8 6.9	95 62.8 6.4	107 59.9 9.2	143 61.7 7.2	99 62.9 7.3	.1466
Wet Day 2	48 61.5 7.3	58 60.0 6.5	58 60.2 8.7	62 61.0 5.9	58 61.6 6.8	58 60.9 7.9	47 61.5 4.8	61 62.6 7.3	73 61.5 7.6	.6520

Note: 1st row = sample size
2nd row = sample mean in mph
3rd row = sample standard deviation in mph

Table E-9. Speed (mph) summary at 7 Interstate highway sites by wet dry conditions.

	Sample Size	Mean	Standard Deviation	Percentile Speeds 15%	50%	85%
SITE 1*						
Dry Day 1	3698	62.1	5.3	58.0	61.8	66.2
Dry Day 2	3514	60.7	6.5	56.1	60.3	64.7
Dry Days	7212	61.4	5.9	56.8	60.6	65.3
Wet Day 1	4501	63.0	7.0	57.4	61.1	68.9
Wet Day 2	1446	59.6	7.2	56.1	61.3	65.4
Wet Days	5947	62.2	7.2	57.1	61.2	67.4
SITE 2*						
Dry Day 1	3657	61.2	5.5	56.6	60.2	65.2
Dry Day 2	2357	59.5	5.4	55.7	59.6	63.7
Dry Days	6014	60.5	5.5	56.0	61.0	64.5
Wet Day 1	2883	59.7	5.9	55.4	59.1	64.5
Wet Day 2	4627	62.2	4.3	58.6	61.7	64.2
Wet Days	7510	61.3	5.2	57.8	60.2	64.3
SITE 3						
Dry Day 1	8518	61.8	5.3	56.9	61.0	66.8
Dry Day 2	5901	61.2	4.9	56.9	60.5	64.7
Dry Days	14419	61.5	5.2	56.9	61.0	65.8
Wet Day 1	6572	61.7	5.5	57.3	61.5	66.1
Wet Day 2	4511	62.7	5.5	58.2	61.7	66.8
Wet Days	11083	62.1	5.5	57.9	61.6	66.0

*Hypothesis of no difference rejected by K-S test at α=0.05

Table E-9. Speed (mph) summary at 7 Interstate highway sites by wet dry conditions (cont.).

	Sample Size	Mean	Standard Deviation	Percentile Speeds		
				15%	50%	85%
SITE 4*						
Dry Day 1	3281	60.1	7.0	55.0	60.0	65.3
Dry Day 2	3093	61.2	7.0	56.1	60.8	65.4
Dry Days	6374	60.6	7.0	55.5	60.4	65.3
Wet Day 1	3508	60.1	5.2	55.9	60.2	63.7
Wet Day 2	7412	61.3	4.4	42.8	50.2	58.3
Wet Days	10920	60.9	4.7	47.0	53.4	60.0
SITE 5*						
Dry Day 1	5140	62.7	4.7	58.9	62.8	66.6
Dry Day 2	4419	62.3	4.8	58.0	61.8	66.0
Dry Days	9559	62.5	4.8	58.3	62.2	66.3
Wet Day 1	4338	62.2	5.4	59.0	61.7	66.2
Wet Day 2	3864	60.9	6.0	56.4	60.5	64.8
Wet Days	8202	61.6	5.7	56.8	60.2	65.1
SITE 6						
Dry Day 1	2741	59.8	7.0	55.6	60.0	64.8
Dry Day 2	2810	59.0	7.5	55.2	60.2	64.1
Dry Days	5551	59.4	7.3	55.4	59.6	64.8
Wet Day 1	3105	57.3	7.6	54.3	59.0	62.2
Wet Day 2	6523	62.7	4.6	56.6	60.8	65.2
Wet Days	9628	61.0	6.3	56.6	60.8	65.2

*Hypothesis of no difference rejected by K-S test at a=0.05

Table E-9. Speed (mph) summary at 7 Interstate highway sites by wet dry conditions (cont.).

	Sample Size	Mean	Standard Deviation	Percentile Speeds		
				15%	50%	85%
SITE 7						
Dry Day 1	5460	59.5	7.7	54.0	61.2	65.7
Dry Day 2	5420	59.4	8.2	52.0	60.8	66.1
Dry Days	10880	59.5	7.9	53.0	61.0	65.8
Wet Day 1	6974	59.2	5.3	54.6	59.3	63.8
Wet Day 2	3440	59.2	5.9	56.2	58.7	62.6
Wet Days	10414	59.2	5.5	54.6	59.2	63.4

difference between the total dry and total wet day distributions was tested by the Kolmogorov-Smirnov test for each site. The total dry day distribution was considered as the known distribution. The null hypothesis that the observations made on the wet days could have been observed from the distribution of the dry day speeds, or in fact that there is no difference between the two distributions, was rejected at five of the seven sites at the α=0.05 level. Further examination showed that the maximum difference between the total dry and wet day speeds at four of the sites is below 2.5 mph. At the one site where these differences exceed 2.5 mph, one of the two sets of wet day speed observations was very different from the other which was very similar to the observations from the two dry days. Furthermore the sample sizes of the wet day observations for the Interstate sites are quite large, requiring almost a complete overlap of the wet day speed observations to the dry day speed distribution to not fail the Kolmogorov-Smirnov test. Thus it was concluded that no practical difference exists between the dry and wet day speed distributions at the Interstate sites. Figures E-2 and E-3 show the cumulative speed distributions for sites 1 and 2 in the Interstate highway category.

Table E-10 presents the speed summaries from the 15 sites on rural arterials. For each site for each day the wet/dry condition is identified, the sample size is given, and the descriptive statistics are presented. Although the

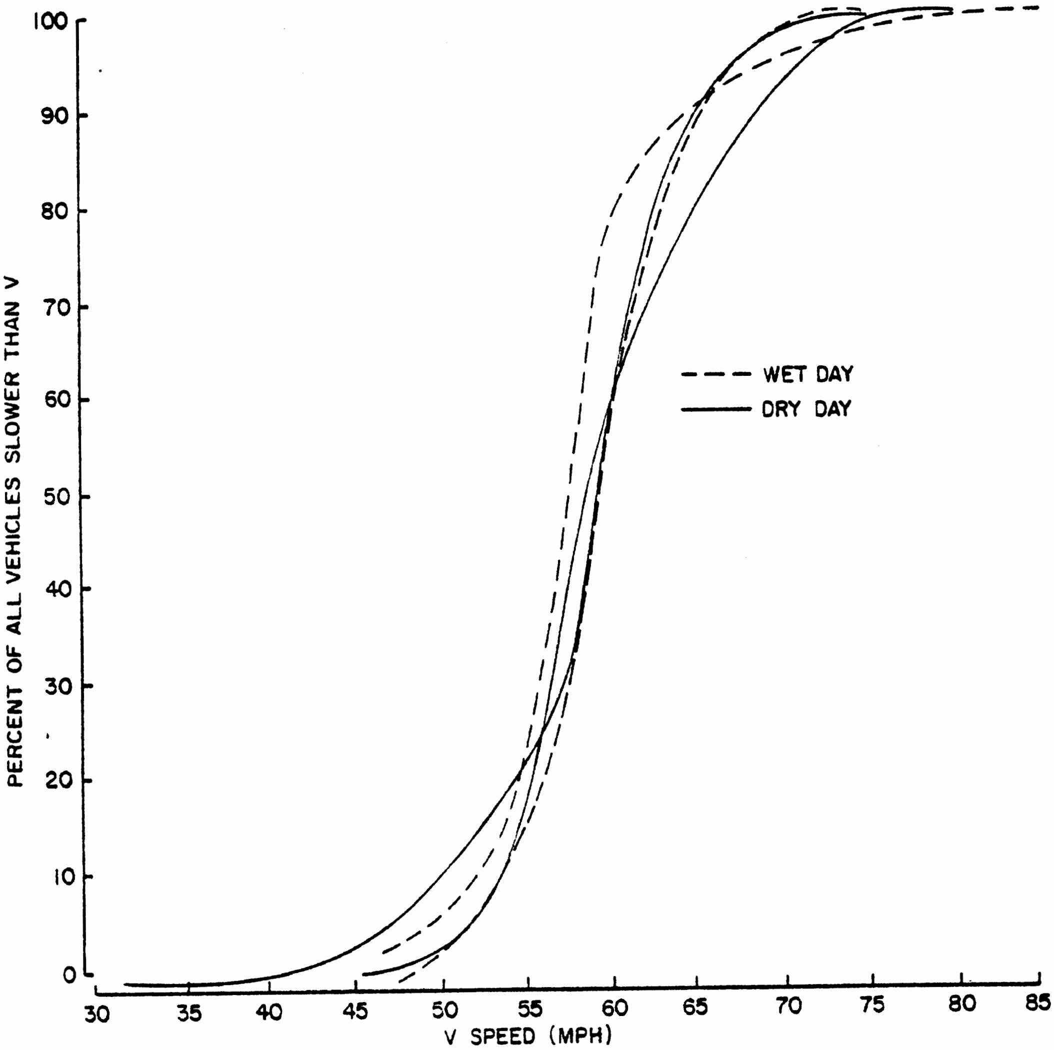


Figure E-2. Cumulative distribution of speeds at daylight, Site 1.

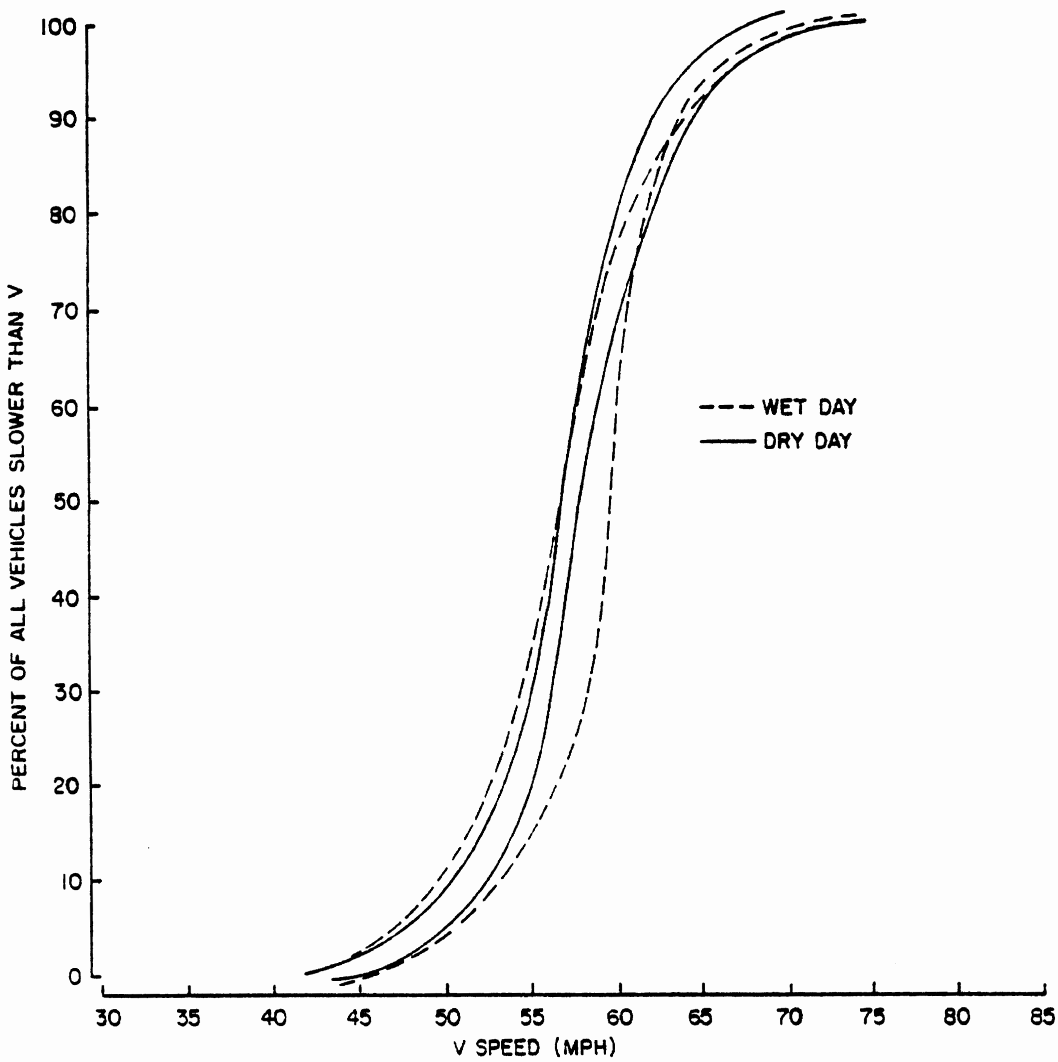


Figure E-3. Cumulative distribution of speeds at daylight, Site 2.

distributions are quite different from site to site, examinations of these summary statistics show good agreement between the dry and wet day distributions at each site. The differences between the total dry day and total wet day distributions at each site were also tested by the Kolmogorov-Smirnov test with the total dry day speed distributions as the known distributions. In 12 of the 15 distributions the null hypothesis of no difference was not rejected at the α=0.05 level. In the remaining three cases site by site examination shows that the maximum difference between speeds at any cumulative percentage at two of the sites was below 2.5 mph, at the third site these differences were below 3.0 mph and the total wet day sample sizes were quite large. This supports the conclusion that there are no practical differences between the distribution of speeds on dry and wet days at these sites.

Figures E-4 and E-5 show the speed distributions of two typical sites in the rural arterial category which clearly indicate the extent of the differences in the distributions for wet and dry days.

Table E-11 summarizes the speed distributions at four sites in the rural collector category for wet and dry days. Again there is little difference between the speed distributions under wet and dry conditions at a particular site. Kolmogorov-Smirnov tests showed no difference between the total wet day speed observations and the total dry day speed distributions at each site at the α=0.05 level.

Table E-10. Speed (mph) summary at 15 rural arterial sites by wet/dry conditions.

	Sample Size	Mean	Standard Deviation	Percentile Speeds		
				15%	50%	85%
SITE 8						
Dry Day 1	245	50.3	14.3	35.0	50.5	63.0
Dry Day 2	266	51.6	13.8	35.0	53.8	64.0
Dry Days	511	51.0	14.0	35.0	52.2	63.5
Wet Day 1	266	52.2	13.9	35.0	54.8	64.1
Wet Days	266	52.2	13.9	35.0	54.8	64.1
SITE 9*						
Dry Day 1	776	64.8	8.0	56.8	62.8	69.9
Dry Day 2	813	66.3	8.6	57.4	63.5	72.2
Dry Day 3	868	65.3	8.5	56.3	63.3	70.8
Dry Days	2457	65.5	8.4	56.7	63.2	70.7
Wet Day 1	920	64.5	8.3	56.0	63.2	69.9
Wet Day 2	99	61.2	7.6	53.1	62.7	68.4
Wet Day 3	414	64.0	6.6	56.8	63.0	67.3
Wet Days	1433	64.1	7.9	55.7	63.1	68.6
SITE 10						
Dry Day 1	866	62.0	7.6	54.2	61.8	65.7
Dry Day 2	496	61.6	8.7	52.1	60.0	66.2
Dry Day 3	521	60.4	7.4	52.4	58.9	63.6
Dry Days	1883	61.4	7.9	52.9	59.8	65.7
Wet Day 1	510	62.0	8.1	53.2	60.7	66.1
Wet Day 2	463	61.8	7.2	54.8	60.2	65.1
Wet Day 3	90	60.0	8.9	51.5	59.8	63.7
Wet Days	1063	61.7	7.8	54.5	60.4	65.2

*Hypothesis of no difference rejected by K-S test at α=0.05

Table E-10. Speed (mph) summary at 15 rural arterial sites by wet/dry conditions. (cont.)

	Sample Size	Mean	Standard Deviation	Percentile Speeds		
				15%	50%	85%
SITE 11						
Dry Day 1	228	61.6	11.9	48.2	60.7	70.3
Dry Day 2	239	57.9	12.9	35.0	57.8	68.8
Dry Days	467	59.7	12.5	42.3	59.2	69.2
Wet Day	305	61.2	14.6	35.0	62.0	72.9
Wet Days	305	61.2	14.6	35.0	62.0	72.9
SITE 12*						
Dry Day 1	801	62.8	6.7	54.2	62.1	65.0
Dry Day 2	408	61.7	7.6	52.7	60.7	65.5
Dry Day 3	308	60.6	6.8	52.6	59.1	63.8
Dry Days	1589	62.0	7.0	54.0	60.3	65.3
Wet Day 1	383	62.1	8.0	54.0	61.9	65.7
Wet Day 2	338	63.0	7.2	53.5	61.7	67.0
Wet Day 3	417	64.6	6.7	56.8	62.7	68.2
Wet Days	1138	63.2	7.4	54.9	62.1	67.0
SITE 13						
Dry Day 1	884	61.5	7.9	54.2	61.0	65.4
Dry Day 2	821	61.9	7.5	54.8	60.5	64.2
Dry Day 3	937	60.9	8.5	53.1	60.2	65.1
Dry Days	2642	61.7	7.7	54.2	60.2	65.1
Wet Day 1	723	61.6	7.5	54.5	60.2	64.8
Wet Day 2	1021	61.1	6.5	54.6	59.7	63.5
Wet Day 3	515	61.2	7.0	54.6	59.7	63.5
Wet Days	2259	61.2	7.0	54.6	59.9	63.9

*Hypothesis of no difference rejected by K-S test at α=0.05

Table E-10. Speed (mph) summary at 15 rural arterial sites by wet/dry conditions. (cont.)

	Sample Size	Mean	Standard Deviation	Percentile Speeds 15%	50%	85%
SITE 14						
Dry Day 1	226	62.5	7.9	52.8	60.9	67.1
Dry Day 2	191	57.3	11.4	42.6	57.6	64.5
Dry Day 3	197	60.7	8.8	51.9	60.8	69.5
Dry Days	614	60.3	9.6	50.2	58.7	67.1
Wet Day 1	129	60.4	9.9	51.2	60.0	64.7
Wet Day 2	234	63.1	7.2	53.7	62.2	66.8
Wet Days	363	62.1	8.4	53.0	60.8	65.3
SITE 15						
Dry Day 1	532	55.7	14.7	35.0	59.3	65.8
Dry Day 2	424	54.2	14.6	35.0	58.6	65.0
Dry Day 3	408	52.0	14.9	35.0	56.0	65.0
Dry Days	1364	54.1	14.8	35.0	58.4	65.2
Wet Day 1	395	53.5	14.3	35.0	56.6	64.2
Wet Day 2	372	56.4	14.8	35.0	60.0	66.7
Wet Day 3	396	53.3	15.3	35.0	55.8	63.4
Wet Days	1136	54.4	14.9	35.0	58.7	65.6
SITE 16						
Dry Day 1	2835	62.6	6.6	55.9	61.3	64.2
Dry Day 2	2381	63.2	7.1	56.5	62.7	66.8
Dry Day 3	2905	63.3	7.0	56.6	61.4	64.8
Dry Days	8121	62.7	6.9	56.5	62.4	65.7
Wet Day 1	1985	62.6	6.6	56.1	61.7	64.8
Wet Day 2	2391	63.4	6.0	58.0	62.3	65.1
Wet Day 3	2086	63.5	5.8	54.8	60.0	63.0
Wet Days	6462	63.2	6.1	57.0	61.4	65.0

Table E-10. Speed (mph) summary at 15 rural arterial sites by wet/dry conditions. (cont.)

	Sample Size	Mean	Standard Deviation	Percentile Speeds		
				15%	50%	85%
SITE 17						
Dry Day 1	792	60.8	8.7	53.9	59.9	65.2
Dry Day 2	780	59.8	9.5	51.0	59.2	65.2
Dry Days	1572	60.3	9.1	51.8	59.4	65.2
Wet Day 1	257	60.3	9.8	50.5	59.4	66.1
Wet Day 2	470	61.3	8.8	50.8	57.1	63.2
Wet Days	727	61.0	9.1	50.7	57.9	64.2
SITE 18*						
Dry Day 1	392	61.3	7.1	55.9	62.0	64.5
Dry Days	392	61.3	7.1	55.9	62.0	64.5
Wet Day 1	294	62.6	7.5	53.2	61.5	67.1
Wet Day 2	294	60.9	7.5	50.4	59.0	65.0
Wet Day 3	305	61.8	6.9	52.8	59.3	65.0
Wet Days	893	61.8	7.3	53.1	59.4	65.4
SITE 19						
Dry Day 1	615	61.7	8.2	54.7	60.2	65.8
Dry Days	615	61.7	8.2	54.7	60.2	65.8
Wet Day 1	337	60.0	8.4	54.7	60.2	65.8
Wet Day 2	244	61.7	8.7	51.4	60.8	67.0
Wet Day 3	184	60.7	9.9	49.3	60.5	66.9
Wet Days	765	60.7	8.9	52.3	60.5	66.4

*Hypothesis of no difference rejected by K-S test at α=0.05

Table E-10. Speed (mph) summary at 15 rural arterial sites by wet/dry conditions. (cont.)

	Sample Size	Mean	Standard Deviation	Percentile Speeds		
				15%	50%	85%
SITE 20						
Dry Day 1	1309	64.1	7.3	55.3	62.7	68.4
Dry Day 2	974	64.5	8.7	56.1	63.4	69.2
Dry Day 3	1040	63.7	8.7	54.6	63.0	69.0
Dry Days	3323	64.1	8.2	54.8	62.9	68.7
Wet Day 1	916	64.2	8.1	54.7	63.5	69.1
Wet Day 2	943	64.4	7.7	55.4	63.0	68.7
Wet Day 3	1009	63.8	7.8	53.5	63.1	68.8
Wet Days	2868	64.1	7.9	54.9	63.2	68.7
SITE 21						
Dry Day 1	1727	60.3	6.5	51.7	58.1	63.3
Dry Day 2	1359	60.4	6.9	51.4	58.2	63.7
Dry Day 3	1705	59.3	7.2	51.5	57.4	62.9
Dry Days	4791	60.0	6.9	51.6	57.9	63.3
Wet Day 1	1255	60.3	6.7	52.0	58.3	63.9
Wet Day 2	1460	59.1	6.0	49.7	55.5	59.9
Wet Day 3	1194	60.9	6.6	52.5	59.0	64.0
Wet Days	3909	60.0	6.5	52.1	58.2	63.1
SITE 22						
Dry Day 1	914	65.6	7.5	57.2	63.1	71.0
Dry Day 2	1013	66.0	8.4	56.7	64.2	71.2
Dry Day 3	1080	66.2	8.7	56.6	63.2	71.1
Dry Days	3007	66.0	8.3	57.0	63.3	71.1
Wet Day 1	1095	65.1	8.3	55.0	63.0	70.9
Wet Day 2	680	66.1	8.0	56.9	63.3	71.0
Wet Day 3	820	65.7	7.4	56.7	63.7	70.5
Wet Days	2595	65.6	8.0	56.0	63.4	70.8

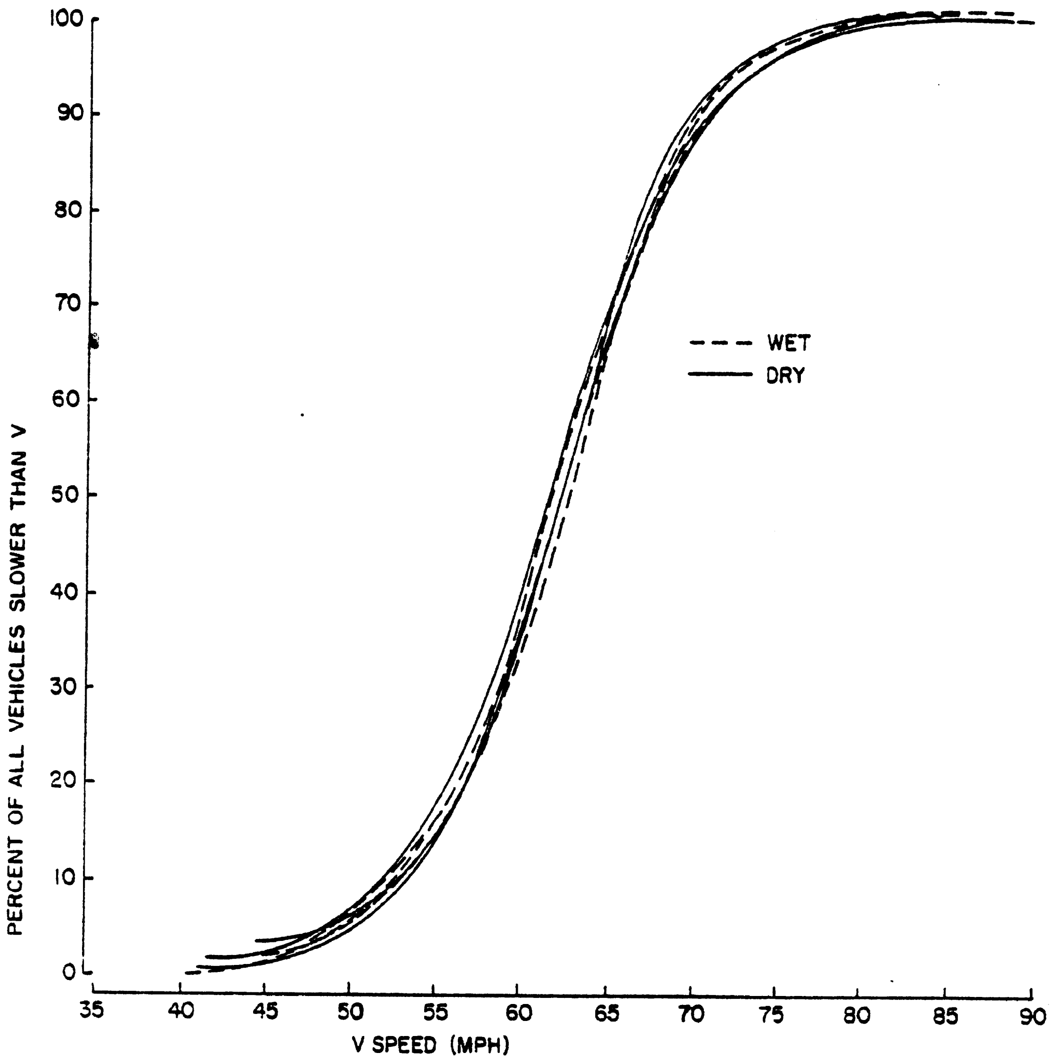


Figure E-4. Cumulative distribution of speeds at daylight, Site 20.

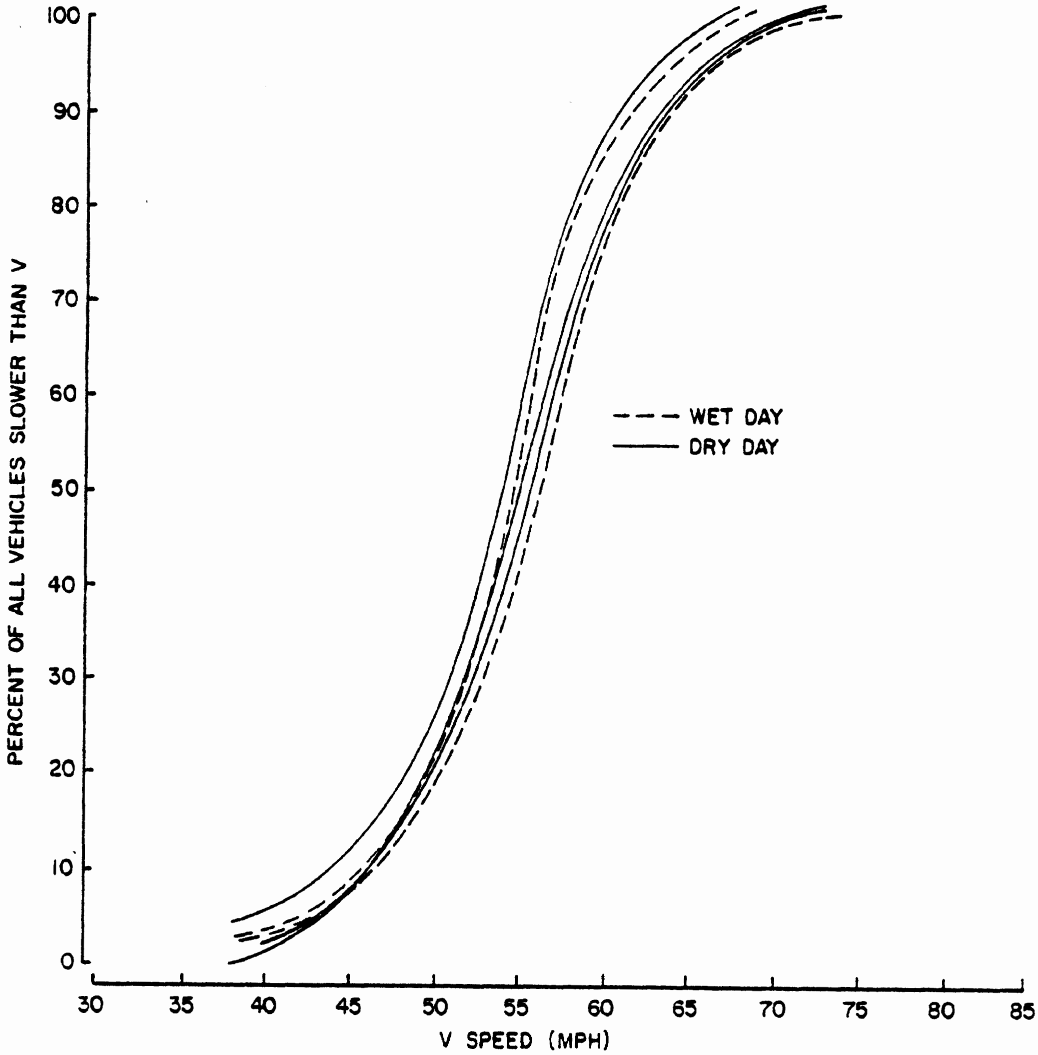


Figure E-5. Cumulative distribution of speeds at daylight, Site 21.

Table E-11. Speed (mph) summary at 4 rural collector sites by wet/dry conditions.

	Sample Size	Mean	Standard Deviation	Percentile Speeds 15%	50%	85%
SITE 23						
Dry Day 1	32	45.3	11.9	35.0	40.2	53.5
Dry Days	32	45.3	11.9	35.0	40.2	53.5
Wet Day 1	78	48.3	12.0	35.0	45.6	59.8
Wet Day 2	67	53.9	12.0	35.0	52.8	63.3
Wet Days	145	50.9	12.3	35.0	48.7	61.4
SITE 24						
Dry Day 1	1398	57.4	7.1	49.5	55.5	61.0
Dry Day 2	1385	57.1	6.8	48.5	54.9	60.4
Dry Day 3	1081	56.8	8.4	47.0	55.7	61.3
Dry Days	3864	57.1	7.4	48.4	55.3	61.0
Wet Day 1	1274	59.0	6.7	51.0	57.0	61.6
Wet Day 2	904	59.5	6.3	52.3	56.8	62.1
Wet Day 3	757	56.0	10.5	38.3	56.2	62.2
Wet Days	2935	58.4	7.9	48.1	56.7	61.7
SITE 25						
Dry Day 1	197	58.9	9.5	50.0	57.6	63.8
Dry Day 2	248	54.8	13.2	35.0	56.7	63.5
Dry Day 3	156	60.5	9.8	50.3	59.4	66.2
Dry Days	601	57.6	11.5	44.9	57.6	64.0
Wet Day 1	26	52.5	13.4	35.0	56.2	62.7
Wet Day 2	169	59.5	9.4	50.2	59.0	64.5
Wet Days	195	58.5	10.3	48.1	58.7	63.9

Table E-11. Speed (mph) summary at 4 rural collector sites by wet/dry conditions.

	Sample Size	Mean	Standard Deviation	Percentile Speeds		
				15%	50%	85%
SITE 26						
Dry Day 1	243	63.8	10.1	52.7	61.3	69.5
Dry Day 2	381	60.9	9.2	49.5	60.0	66.2
Dry Days	624	62.0	9.7	50.7	60.5	67.5
Wet Day 1	268	61.6	7.9	53.0	59.7	65.5
Wet Day 2	570	61.4	9.3	50.9	59.1	67.2
Wet Day 3	261	61.9	8.4	52.2	60.0	66.5
Wet Days	1099	61.6	8.7	51.1	59.3	66.8

Figures E-6 and E-7 illustrate this with sets of cumulative speed distributions for two sites in this road category.

Conclusions

Statistical examination of speed distributions on three classes of rural highway facilities (Interstate, arterial, and major collector) shows that the assumptions that speeds are normally distributed is a reasonable one. Specific distributions for the three categories of rural highway were not obtained because the categories are much too broad for their speeds to be captured by a single distribution.

Comparison of speed distributions at a sample of sites collected during daylight hours on wet and dry days showed no discernable differences between the speed distributions at a site on wet and dry days. Eighteen of the 26 sites showed no significant difference between the speed observations made on wet days when compared to the dry day speed distributions at the same site. In the remaining eight cases the wet day sample sizes were large, requiring almost a perfect fit for the null hypothesis of no difference not to be rejected by the Kolmogorov-Smirnov test. The maximum differences in speed at any given cumulative percentage in six of these eight cases were less than 2.5 mph, implying no practical difference between the speed distribution. Thus it can be concluded that there is no practical difference in the speeds drivers select when driving on the same roads under wet or dry conditions during the day.

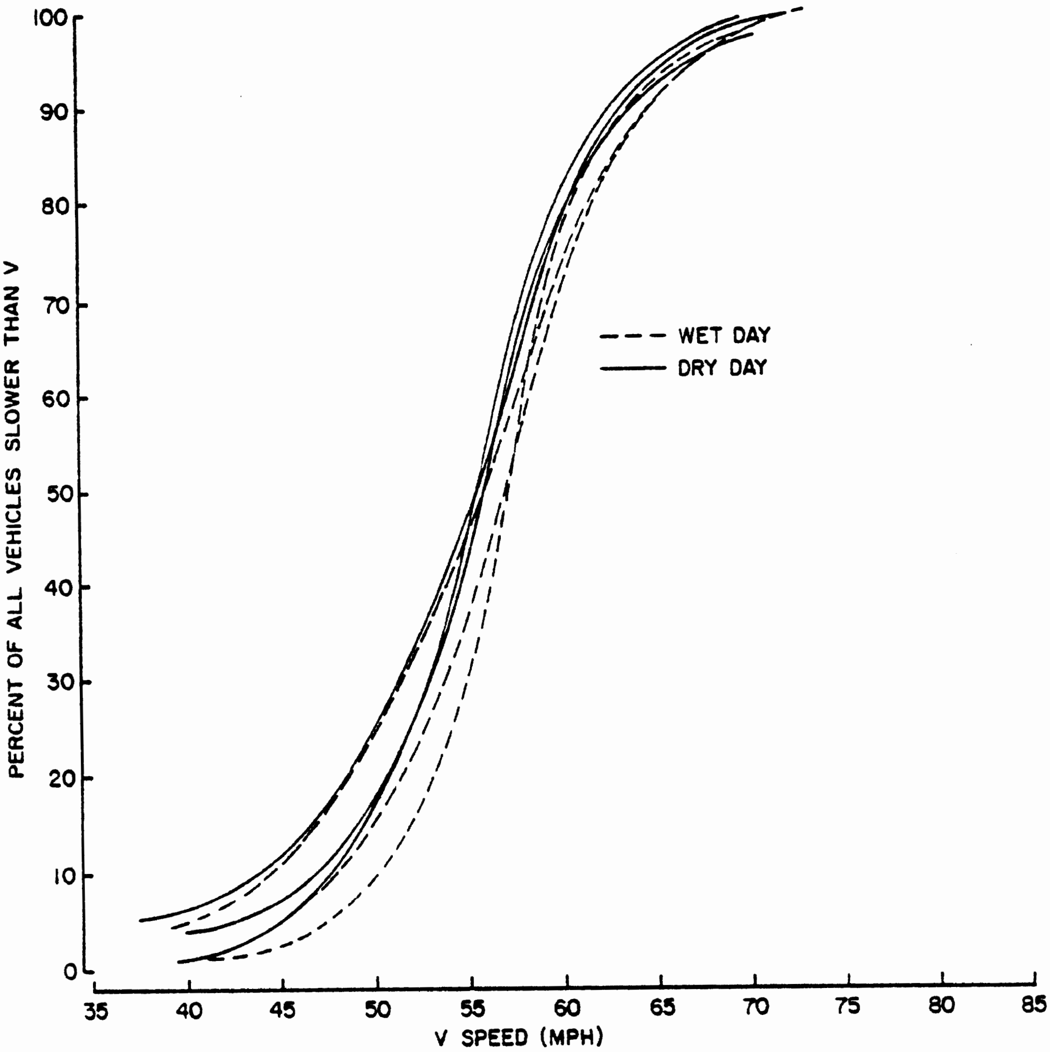


Figure E-6. Cumulative distribution of speeds at daylight, Site 24.

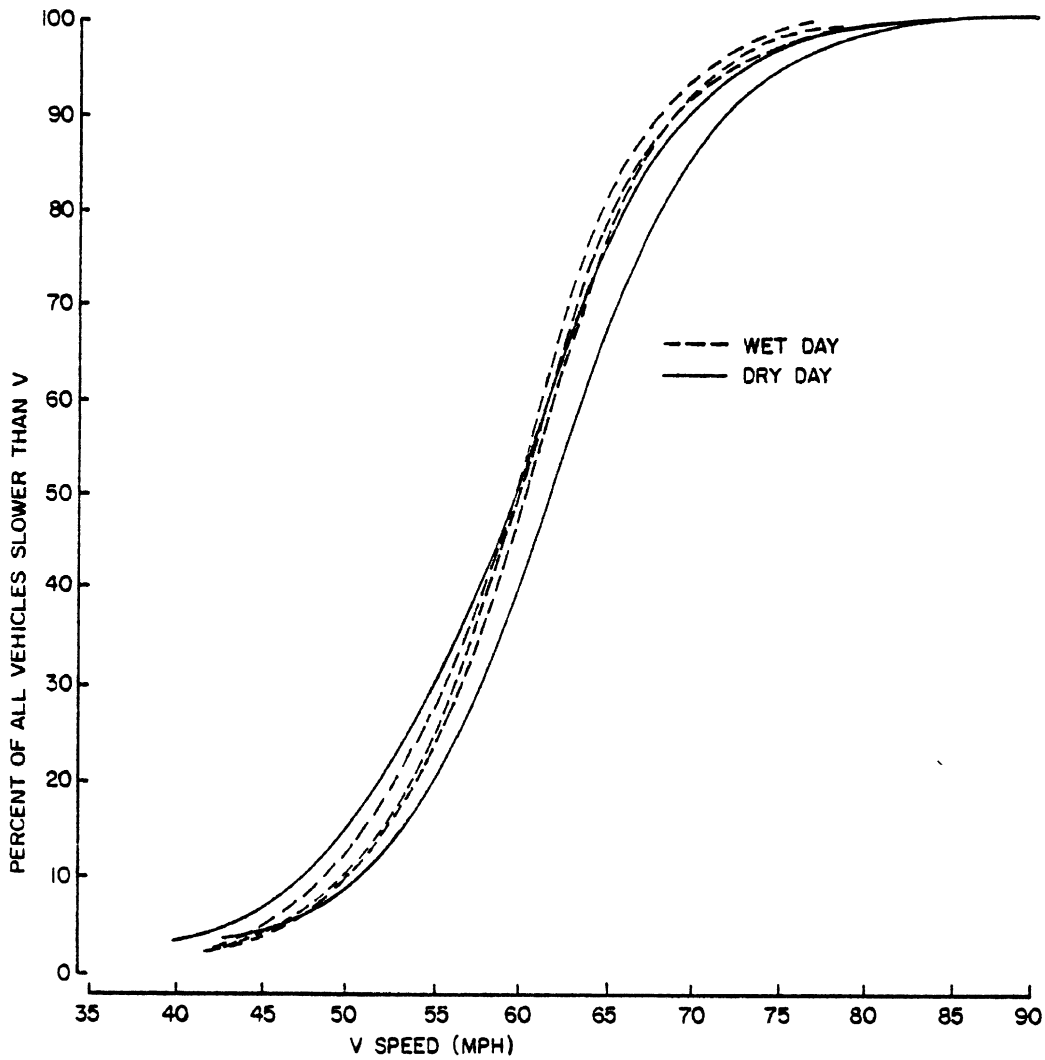


Figure E-7. Cumulative distribution of speeds at daylight, Site 26.

GEOMETRIC DESIGN CONSIDERATIONS

Sight Distance and Curve Parameters on Crest Vertical Curves

This discussion of sight distance on crest vertical curves is divided into several sections. First, the basic elements of sight distance for crest vertical curves are defined and assumptions outlined. In the second section the geometrical relationship for the sight distance from the observer's eye to the tangent point of the line of sight is derived. In the third section the geometrical relationship for the sight distance from the tangent point to the top of the object is derived. The sum of these two components, the total sight distance S (used here rather than SSD) is discussed in section four. In the fifth section, using these derived results, the variation of the sight distance for a moving observer and object for two cases, $S < L$ and $S > L$, is developed. It is shown that all sight distance cases can be obtained from the general sight distance formula (Eq.-16) developed in section four. Relationships are then developed showing the variation of sight distance S with curve length L and change in grades, a (A/100), for current policy eye and object heights.

Basic Elements of Sight Distance for Crest Vertical Curves

Figure E-8 shows the basic sight distance elements for crest vertical curves. It can be shown that assuming symmetry in the sight distance elements is not important in the derivation. The sight distance S is divided into two components, S_e and S_o. S_e is the distance from the eye of

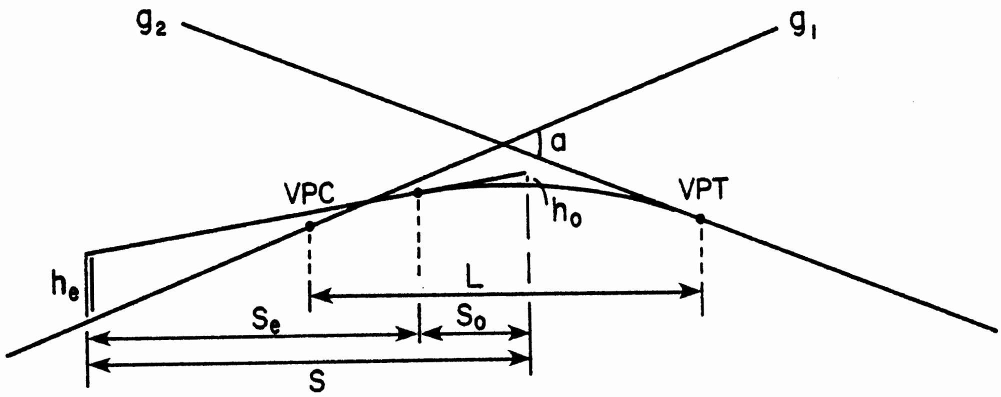


Figure E-8. Basic elements of sight distance (S) on crest vertical curves with curve length (L), grades g_1 and g_2, change of grade ($a=g_1-g_2$), eye height (h_e) and object height (h_o).

the observer to the tangent point of the line of sight to the curve, and S_o is the distance from the tangent point to the top of the object; hence, total sight distance $S = S_e + S_o$. The curve length, L, is measured horizontally between the VPC and VPT. All sight distances are measured as being in the horizontal plane. The difference in grades, a, is here defined as $0.01G_1-0.01G_2$, where G_1 is the percent of grade on the approach tangent and G_2 that on the departing tangent ($a=0.01A$, the difference in percent, $g_1=0.01G_1$ and $g_2=0.01G_2$).

Sight Distance Component From the Observer to the Tangent Point. In Figure E-9 the sight distance component from the observer's eye to the tangent of the curve point, S_e, is shown. For the crest vertical curve a parabola is used in U.S. practice with the general formula

$$y = Ax^2 + Bx + C \tag{E-1}$$

For the case shown in Figure E-9 where the parabola passes through the VPC (0,0) and the VPT (L,0) and has tangents, y_t,

$$y_t = g_1 x \qquad \text{through } (0,0)$$
$$y_t = -g_2 x + g_2 L \qquad \text{through } (L,0)$$

the parabola has a particular solution

$$y = g_1 x + (a/2L)x^2 \tag{E-2}$$

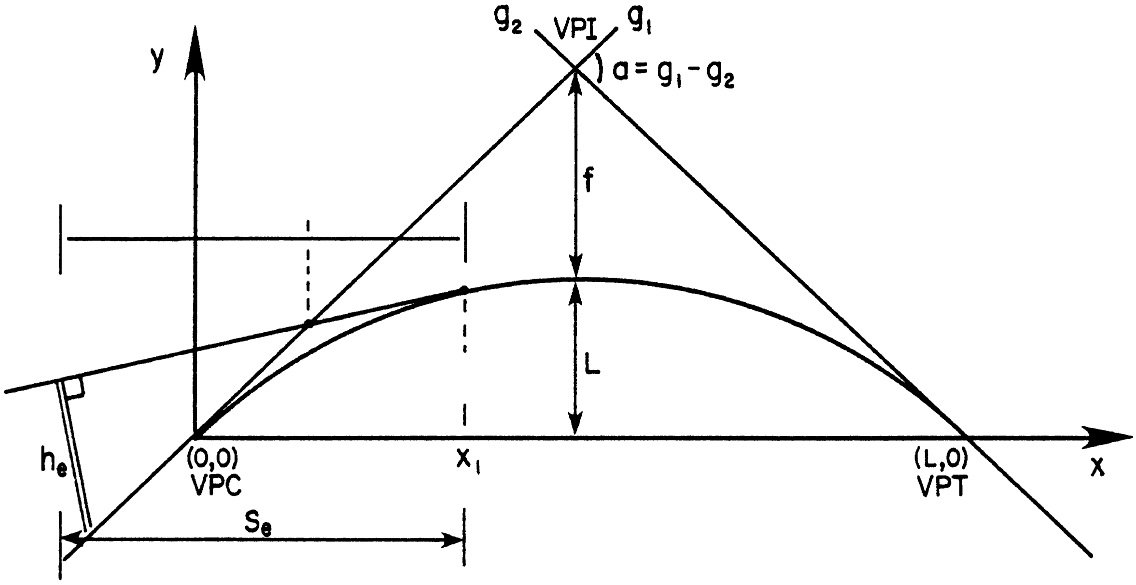


Figure E-9. Sight distance component, S_e, from the observer to the tangent point.

The ordinate at the maximum point can be determined by taking the derivative of y, setting it equal to zero, and if the grades are given in percent then Eq. E-3 can be obtained:

$$c = AL/800 = aL/8 \tag{E-3}$$

where c is the vertical offset at the VPI in feet.

Next the geometrical relationship for the sight distance component, S_e, from the observer to the tangent point of the line sight, is derived for the case when the observer is on the approach tangent.

In Figure E-10 a portion of Figure E-9 is shown. For the derivation of the expression of sight distance, S_e, from the observer to the tangent point at x_1, the distance is divided into two component, S_1 and S_2. S_1 is the distance from the observer to the VPC and S_2 is the distance from the VPC to the tangent point of the line of sight at x_1. The tangent through (x_1, y_1) can be obtained using Eq. E-2, the slope through any point of the parabola and the general formula of a line through a point for a given slope as follows:

$$y - y_1 = s\,(x - x_1)$$

Using Eqs. E-1 and E-2, we obtain for the particular tangent

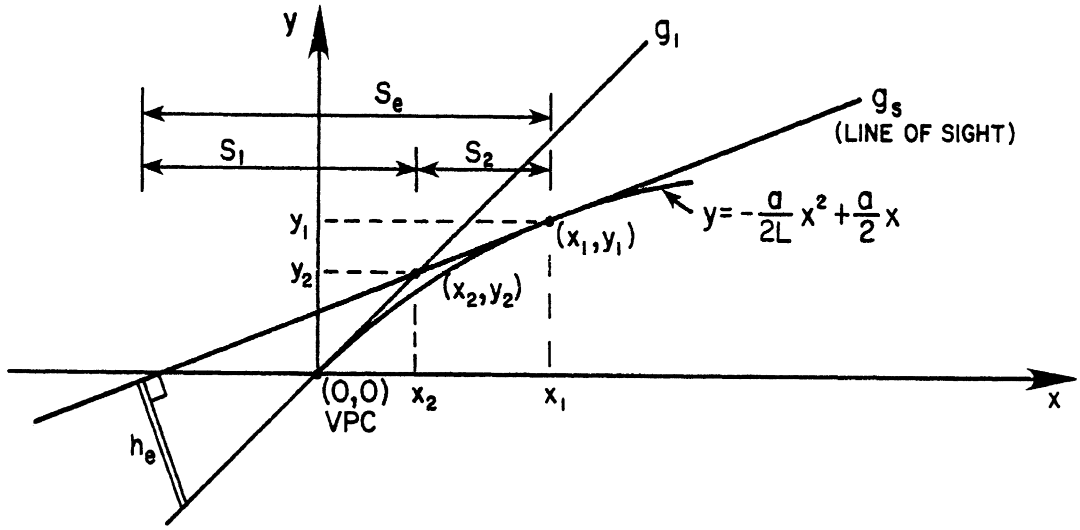


Figure E-10. Sight distance S_e from the observer to the tangent point (x_e, y).

$$y = -[(ax_1/L) - (a/2)]x + [(ax_1^2)/(2L)] \quad \text{(E-4)}$$

The two tangents $y = g_1x$ through (0,0) and Eq. E-4 have a common point (x_2,y_2):

$$x_2 = x_1/2$$

$$y_2 = ax_1/4$$

Now S_2, see Figure E-10, can be obtained as the distance T from (x_1,y_1) to (x_2,y_2) which is equal to the distance from (0,0) to (x_2,y_2):

$$S_2 = T = (x_2^2 + y_2^2)^{1/2} \quad \text{(E-5)}$$

$$S_2 = T = (x_1/2)(1 + a^2/4)^{1/2} \quad \text{(E-6)}$$

for small values of a the square root is about 1, and hence

$$S_2 = x_1/2.$$

From Figure E-11 S_1 can be obtained by

$$S_1 = h_e/(g_1 - g_s)$$

where:

g_1 = $a/2$;

g_s = $a/2 - ax_1/L$; and

h_e = observer eye height.

Inserting this in the above expressions, S_1 becomes

$$S_1 = h_eL/(ax_1) \quad \text{(E-7)}$$

Now S_e can be obtained from Eqs. E-6 and E-7

$$S_e = S_1 + S_2$$

$$S_e = (h_e L)/(a x_1) + x_1/2 \tag{E-8}$$

The location of the minimum of the sight distance component S_e can be obtained by taking the derivative and setting it equal to zero to obtain the location x_1 of the tangent point and inserting this in Eq. E-8:

$$dS_e/dx_1 = -(h_e L)/(a x_1^{\,2}) + 1/2$$

$$dS_e/dx_1 = 0 \text{ then } x_1 = (2h_e L/a)^{1/2}$$

and the minimum sight distance $S_{e,min}$ is

$$S_{e,min} = (2h_e L/a)^{1/2} \tag{E-9}$$

When the second derivative of S_e is taken it can be verified that this value is a minimum.

The sight distance component S_e from the observer to the tangent point of the line of sight as given in Eq. E-8 describes the variation of S_e when the observer is approaching the crest vertical curve. This S_e component reaches its minimum, Eq. E-9, when the observer is at the VPC and remains constant as long as x, the tangent point of the line of sight is between

$$X_1 = (2h_e L/a)^{1/2} < x < L$$

where X_1 is the critical position of the tangent point when the observer is at the VPC.

Sight Distance Component From the Tangent Point to the Object. In Figure E-11 the sight distance component, S_o, from the tangent point to the object is shown for the case where the object is beyond the VPT. The derivation is made as above. Compare Figures E-9 and E-11. S_{10} is the distance from the tangent point at x_1 to the common point with the tangent which equals g_2 at (x_3), g_2 passes through (L,0) with slope g_2 = -a/2 and has the formula

$$y = (a/2)(L - x)$$

The line of sight has the same formula as Eq. E-4. These two tangents can now be shown to have a common point with coordinates

$$x_3 = (L + x_1)/2 \qquad \text{(E-10)}$$

$$y_3 = a(L - x_1)/4$$

Now S_{10} is equal to the distance from $(x_3;y_3)$ to (L,0), i.e.,

$$S_{10} = [y_3{}^2 + (L - x_3)^2]^{1/2}$$

$$S_{10} = (L - x_1)\ (A^2 + 1)^{1/2}\ /\ 2$$

The square root for reasonable values of a is approximately unity and hence

$$S_{10} = (L-x_1)/2 \qquad \text{(E-11)}$$

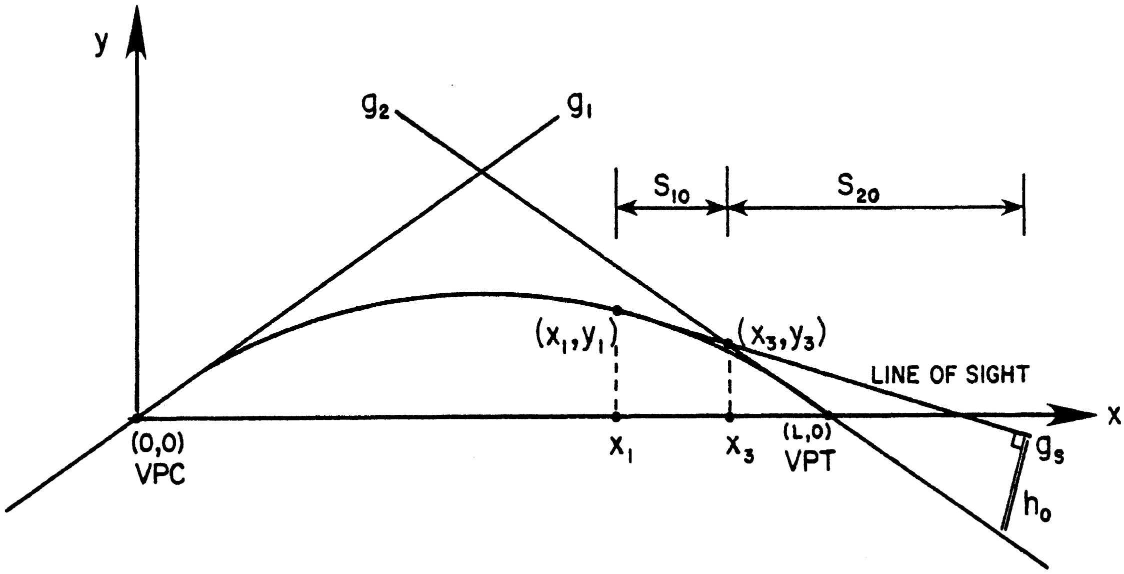


Figure E-11. Sight distance component (S_o) from the tangent point to the object.

In Figure E-11 the sight distance component S_{20} is the distance from the tangent intersection point at x_3 to the object. With g_s and g_2 we obtain

$$S_{20} = h_o/(g_s - g_2) = h_oL/[a(L - x_1)] \qquad \text{(E-12)}$$

where h_o= object height.

Then the sight distance S_o from the tangent point to the object becomes

$$S_o = S_{10} + S_{20} = (L-x_1)/2 + h_oL/[a(L-x_1)] \qquad \text{(E-13)}$$

The minimum of the sight distance component S_o can be obtained, as earlier for S_e, and it is

$$S_{o,min} = (2h_oL/a)^{1/2} \qquad \text{(E-14)}$$

The sight distance component S_o as given in Eq. E-13 describes the variation of the distance from the tangent point of the line of sight to the object. This S_o component reaches its minimum, Eq. E-14, when the object is in the curve, and remains constant as long as x, the tangent point of the line of sight, is between the VPC (0,0) and $X_2 = L - \{2h_oL/a\}^{1/2}$. X_2 is the the critical position of the tangent point when the the object is at the VPT (L,0).

Total Sight Distance. For the total sight distance S from the observer to the object two cases, S<L and S>L, are developed in this section. Using Eqs. E-9 and E-14 the formulas for sight distance and vertical curve length can be obtained for

S < L:

$$S = S_e + S_o = (2h_e L/a)^{1/2} + (2h_o L/a)^{1/2} \qquad \text{(E-15)}$$

$$L = aS^2/[(2h_e)^{1/2}+(2h_o)^{1/2}]^2$$

S > L:

The general expression for the total sight distance can be obtained from Eqs. E-8 and E-13

$$S = S_e + S_o \qquad \text{(E-16)}$$
$$= h_e L/(ax_1) + x_1/2 + (L - x_1)/2 + h_o L/\{a(L - x_1)\}$$

and hence when combining terms and inserting A = 100a the general sight distance formula can be simplified

$$S = L/2 + 100\, h_e L/(Ax) + 100\, h_o L/[A(L-x)] \qquad \text{(E-16a)}$$

The minimum sight distance when neither object nor observer is on the vertical curve, but beyond and before it respectively, can be obtained taking the derivative of Eq. E-16 with respect to x_1 and setting it equal to zero to determine first the location of the tangent point, x_1, and then solving to obtain S_{min}. Hence:

$$dS/dx_1 = - h_e L/(ax_1{}^2) + h_o L/[a(L - x_1)^2]$$

Set $dS/dx_1 = 0$, then $x_1 = L[(h_o h_e)^{1/2} - h_e]/(h_o - h_e)$.

Inserting x_1 into Eq. E-16 the formula for S_{min} can be obtained for S>L

$$S_{min} = L/2 + [(h_o)^{1/2} + (h_e)^{1/2}]^2/a \qquad \text{(E-17)}$$

Solving for L we obtain

$$L = 2S - (2/a)[(h_o)^{1/2} + (h_e)^{1/2}]^2 \qquad \text{(E-18)}$$

Equation E-16 can be viewed as the "general sight distance formula" for crest vertical curves, covering both cases. For $S<L$, Eq. E-15 can be obtained from Eq. E-16 by minimizing the separate sight distance components S_e and S_o. Equation E-17 can be obtained from Eq. E-16 through minimization of the total sight distance S.

<u>Variation of Sight Distances for Moving Observer and Object</u>. In this section we discuss the variation in sight distance for a moving observer and object for both cases, $S < L$ and $S > L$. Sight distance graphs showing the variation of S as a function of curve length L and change in grade a for typical eye height h_e and object height h_o values are developed.

o <u>$S < L$</u>

For this situation the variation for an approaching observer can be treated in three cases:

1. Observer on tangent, object on the vertical curve. The sight distance component S_e varies according to Eq. E-8 and reaches its minimum when the tangent point of the line of sight is at X1. The sight distance component S_o is constant for $0 < x < X_1$ (Eq. E-14). This means that the total sight distance S varies when x is between zero and X_1,

i.e., for $0 < x < X_1$, where $X_1 = (2h_eL/a)^{1/2}$, the sight distance S is:

$$S = h_eL/(ax) + x/2 + (2h_oL/a)^{1/2} \tag{E-19}$$

2. Observer and object both on the vertical curve. Both sight distance components S_e and S_o are constant and at their minimum values as long as $X_1 < x < X_2$. The boundary values reflect the transition when the observer is at the starting point of the vertical curve. Then $x = X_1$, and when the object is at the ending point of the vertical curve, $x = X_2$. The total sight distance S is constant and behaves according to Eq. E-15, i.e., for $X_1 < x < X_2$, where X_1 is as under 1 and $X_2 = L - (2h_oL/a)^{1/2}$, the sight distance S is:

$$S = (2h_eL/a)^{1/2} + (2h_oL/a)^{1/2} \tag{E-20}$$

3. Observer on the curve, object beyond the VPT. The sight distance component S_e is constant for $X_2 < x < L$, Eq. E-9. The sight distance component S_o varies according to Eq. E-13 and has its minimum when x, the tangent point of the line of sight, is equal to X_2. This means that the total sight distance S varies when x is between X_2 and L, i.e., for $X_2 < x < L$, where $X_2 = L - (2h_oL/a)^{1/2}$, the sight distance S is:

$$S = (2h_eL/a)^{1/2} + (L-x)/2 + h_oL/[a(L - x)] \tag{E-21}$$

o $\underline{S > L}$

When the sight distance S is greater than L, two cases must be considered separately, $S_o < L$ and $S_o > L$. In the first case, the observer is on the tangent grade and the object is on the vertical curve. When $S_o > L$, the object is beyond the VPT.

When $S_o < L$ one follows the three-step procedure as above when determining the variation in the sight distance. In the second case, $S_o > L$, one has only one step because the vertical curve does not affect the relationship.

o $\underline{S_o < L}$

In this case the sight distance S is greater than the curve length L, but the sight distance component to the object S_o is shorter than the curve length L. The variation of the sight distance when the observer is approaching the vertical curve can be described by the following:

1. Observer on the grade; object on the vertical curve.

For $0 < x < X_2$ (Note: $X_1 > X_2$), where $X_2 = L - (2h_oL/a)^{1/2}$ as before, the sight distance S is:

$$S = h_eL/(ax) + x/2 + (2h_oL/a)^{1/2} \qquad \text{(E-22)}$$

Between the boundary values, because $X_1 > X_2$, the sight distance does not reach its absolute minimum, but reaches a local minimum at X_2.

2. Observer on the grade; object beyond the VPT. For $X_2 < x < X_1$ (note: $X_1 > X_2$), where X_1 and X_2 are as before, the sight distance S is:

$$S = h_eL/(ax) + x/2 + (L-x)/2 + h_oL/[a(L-x)] \qquad \text{(E-23)}$$

The sight distance has its absolute minimum when

$$x = L[(h_eh_o)^{1/2} - h_e]/(h_o - h_e) \qquad \text{(E-24)}$$

3. Observer on the vertical curve; object beyond the end of the curve. For $X_1 < x < L$ (note: $X_1 > X_2$), where X_1 is as before, the sight distance S is:

$$S = (2h_eL/a)^{1/2} + (L - x)/2 + h_oL/[a(L - x)] \qquad \text{(E-25)}$$

Because $X_1 > X_2$, the sight distance does not reach its absolute minimum between the boundary values, but reaches a local minimum at X_1.

When $S > L$ and $S_o > L$, we have the case with the observer approaching and object beyond the curve, with boundaries $0 < x < L$, i.e., for $0 < x < L$, the sight distance S is, Eq. E-16,

$$S = h_eL/(Ax) + x/2 + (L - x)/2 + h_oL/[A(L - x)]$$

The sight distance has its minimum, as determined earlier when

$$x = L[(h_eh_o)^{1/2} - h_e]/(h_o- h_e) \qquad \text{(E-25a)}$$

Sight Distance Graphs

These design aids were generated using the approach discussed above for some example cases. Figures 7 to 11 in the text are examples of these graphs with respect to the location of the observer.

A computer program was written in Standard PASCAL to generate sight distance graphs in tabular form. Graphs were plotted using the tabular output of the original programs with the statistical program package MIDAS of the Michigan Terminal System (MTS) of the University of Michigan. Figure E-12 shows an example of sight distance graph plot with respect to the the point of tangency of the line of sight for the same case as Figure 7.

The computer program can be used to:

1. Calculate sight distance graphs for given S, h_e, h_o, and a.

2. Calculate sight distance graphs for given L, h_e, h_o, and and a.

3. Calculate minimum required L values for given S, h_e, h_o, and a. The L values will be rounded up to the nearest hundred feet.

4. Calculate minimum required S for given L, h_e, h_o, and a.

5. Carry out sensitivity analysis with respect to any of the variables S, L, h_e, h_o, or a.

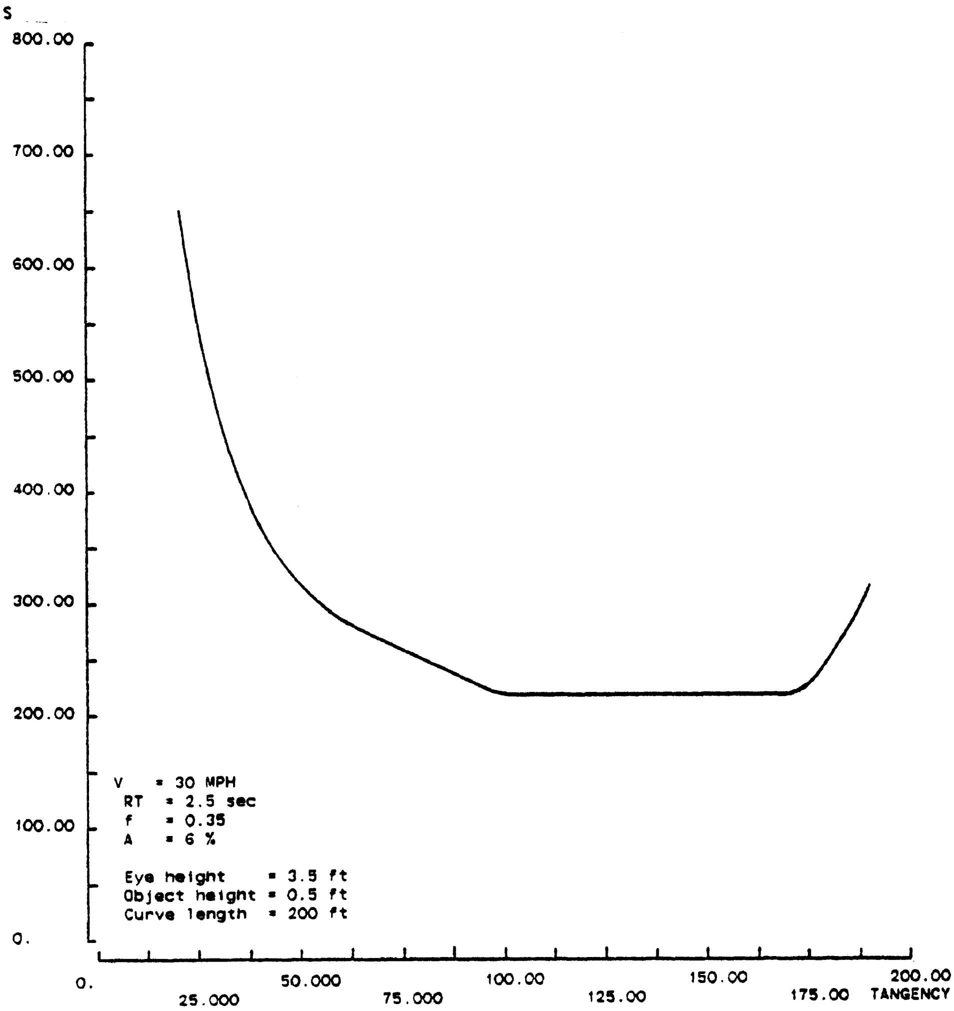


Figure E-12. Sight distance graph with respect to the point of tangency of line of sight.

Radius of Vertical Curve

From elementary calculus:

$$R(x) = \{[1 + (dy/dx)^2]/(d^2y/dx^2)\}^{3/2}$$

where R(x) is the radius of the vertical curve at point x, and y is the elevation of the vertical curve.

To a satisfactory approximation:

$$R(x) = (L/a)\{1 + (a^2x^2)/L^2 + 2ag_1x\}^{3/2}$$

The term is very near 1.0 for all values of x and hence, one can approximate the radius by L/A.

Sight Distance and Curve Parameters on Horizontal Curves

In this section the geometrical relationships among the factors affecting the variation of sight distance from the observer to the object for horizontal curves for four cases are studied. Later in this section the derivations are extended to cover nine cases. The four cases on which the derivation is based are:

Case 1: Observer and object in the horizontal curve.

Case 2: Observer before the PC and object in the horizontal curve.

Case 3: Observer in the curve and object beyond the PT of the curve.

Case 4: Observer before the PC and object beyond the PT.

In the following section the geometrical relationships are derived for the above cases for a single obstacle to the line of sight inside the curve. The derivation is divided into three parts as follows:

1. Derivation of the sight distance component S_e from the observer to the obstacle.
2. Derivation of the sight distance component S_o from the obstacle to the object in the roadway.
3. Total sight distance S.

It should be pointed out that the formulas derived are also valid for any other type of obstacle restricting the sight distance, such as guardrail, slope cut, fencing, etc., because the minimum value obtained with the formulas remains constant as long as the line of sight from the observer to the object has a tangent with respect to, for example, the guardrail or slope.

In this section the general sight distance formula for horizontal curves is derived. It is shown that this formula covers all cases of available sight distance with respect to locations of observer, obstacle, and object. With the general formula the available sight distance can be calculated for any combination of the locations of observer and obstacle in a horizontal curve. In the last part of this section a procedure is described to calculate sight distances in horizontal curves using the "General Sight Distance Formula For Horizontal Curves." The formulas give as sight distances the distances along the line of sight. The notation used in the derivations is summarized in Table E-12.

Derivation of Formulas for Variation of Sight Distance in Horizontal Curves

Case 1: Observer and Object in the Horizontal Curve. Case 1 is shown in Figure E-13. In this figure the notation used in the derivation is also shown. The obstacle is located inside the curve with radius R at a distance m from the centerline (CL) of the lane. Observer and object are both assumed to be located on the CL. The small effect of

Table E-12. Summary of notation used in derivation of formulas for horizontal curves.

R	=	radius of the curve
I_1	=	central angle of the line of sight corresponding to the component of the line of sight from the observer to the obstacle
I_2	=	central angle of the line sight corresponding to the component of the line of sight from (K), see figures, to the obstacle. Note that the line K-O is perpendicular to the line of sight, see Fig. E-
I_{ob}	=	the central angle of the line of sight corresponding to the distance from the observer to the PT, end of the curve, all cases
L	=	curve length
L_{ob}	=	the distance along the curve from the observer to the PT, when the observer is in the curve
L_{01}	=	the distance from the observer to the station where the obstacle is located, for observer in the curve
L_{02}	=	the distance from PC to the station where the obstacle is located for observer at the PC or before the PC
L_{21}	=	the distance from the observer to PC for observer before the PC
L_{22}	=	the distance from PT to the object, for object beyond the PT.
m	=	lateral clearance at the obstacle measured from the centerline of the lane
S_e	=	S_1+S_2; sight distance component from the observer to the obstacle
S_o	=	sight distance component from the obstacle to the object
S	=	S_e+S_o; total sight distance

other positions of the observer and object is discussed in another section of this report.

Derivation of S_e, the sight distance component from the observer to the obstacle. From Figure E-14 we obtain

$$S_1 = R \sin(I_1 - I_2) \tag{E-26}$$

$$S_2 = R_1 \sin(I_2) = (R - m) \sin(I_2) \tag{E-27}$$

and hence

$$S_e = S_1 + S_2 = R \sin(I_1 - I_2) + (R - m) \sin(I_2) \tag{E-28}$$

where:

$$I_1 = 180L_{01}/(\pi R) \tag{E-29}$$

in which L_{01} is the distance from the observer to the station of the obstacle along the CL as shown.

I_2 can be obtained from Figure E-13 using the trigonometric relationship for h, the shortest distance from the center point of the curve perpendicular to the line of sight.

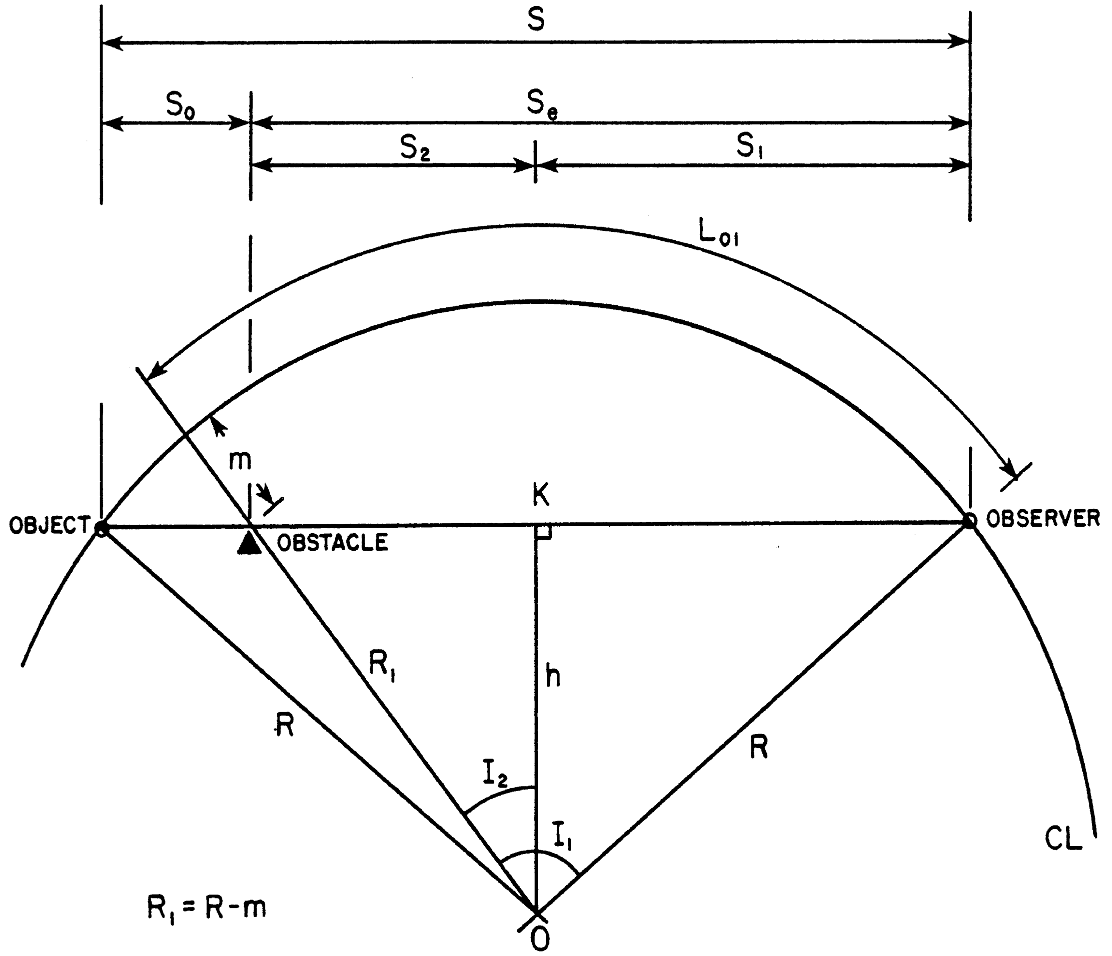


Figure E-13. Observer and object in the horizontal curve (Case 1).

We have then

$$h = R \cos(I_1 - I_2) \quad \text{(E-30)}$$

and also

$$h = (R - m) \cos(I_2) \quad \text{(E-31)}$$

and hence when setting these expressions for h: $h = R \cos(I_1-I_2) = (R - m) \cos(I_2)$ and solving for I_2, we obtain

$$\tan(I_2) = \{R[1-\cos(I_1)]-m\}/[R \sin(I_1)] \quad \text{(E-32)}$$

$$I_2 = \tan^{-1}\{[R(1-\cos(I_1))-m]/[R \sin(I_1)]\} \quad \text{(E-33)}$$

<u>Derivation of S_o</u>, the sight distance component from the obstacle to the object. Using the results obtained above and Figure E-13, we determine

$$S_o = S_1 - S_2 = R \sin(I_1-I_2) - (R - m) \sin(I_2) \quad \text{(E-34)}$$

<u>Derivation of S</u>, the total sight distance for the case when both observer and object are in the horizontal curve. Using the results obtained above and Figure E-14, we obtain

$$S = S_e + S_o = 2S_1 = 2R \sin(I_1 - I_2) \quad \text{(E-35)}$$

where I_1 and I_2 are as derived in Eqs. E-29 and E-33.

<u>Case 2: Observer Before the PC and Object in the Horizontal Curve</u>. Case 2 and the notation used are presented in Figure E-14. The derivation follows the three steps described previously.

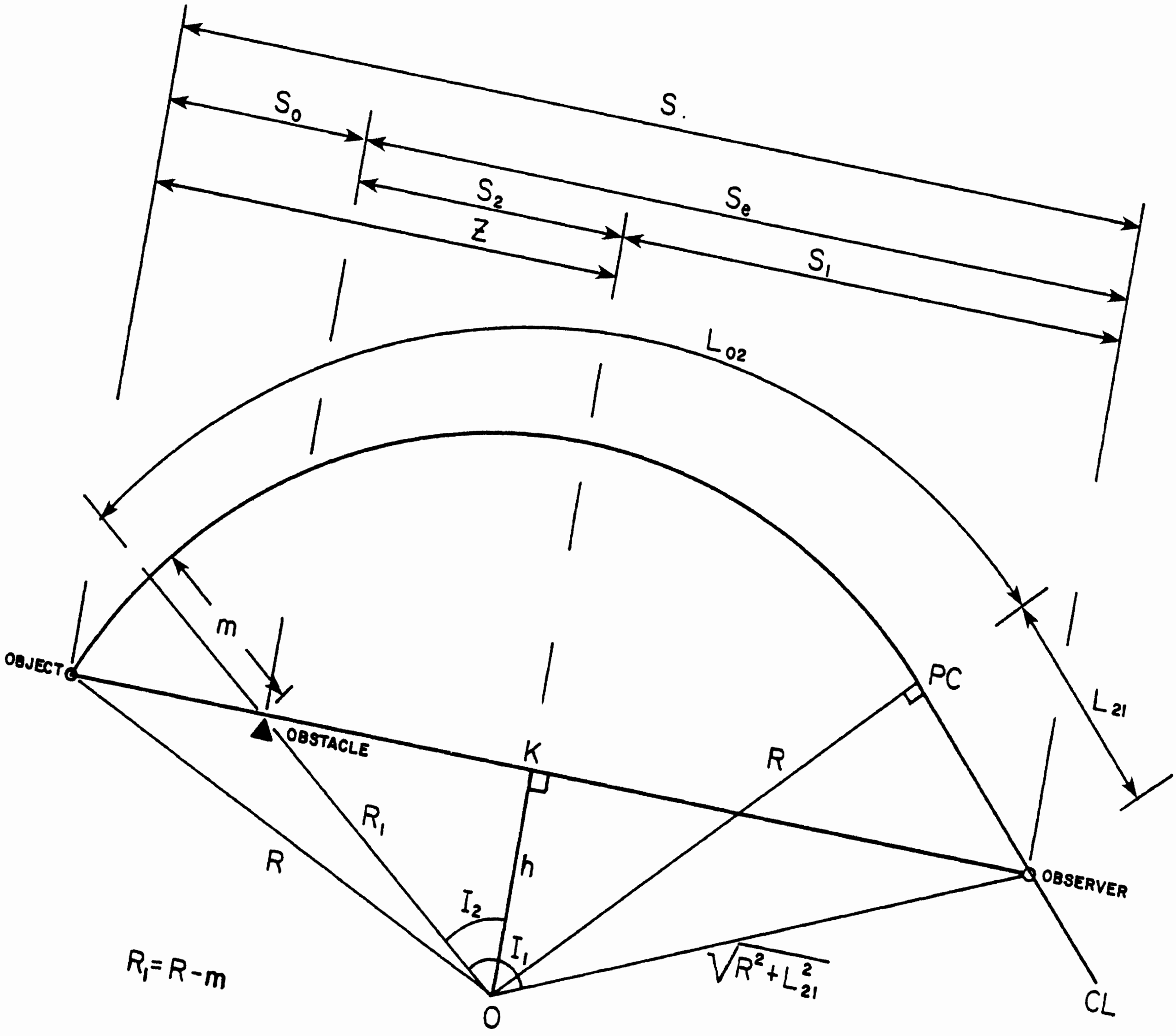


Figure E-14. Observer before the PC and object in the horizontal curve (Case 2).

Sight distance component S_e from the observer to the obstacle is derived from Figure E-14. Thus:

$$S_1 = \sin(I_1 - I_2)\ (R^2 + L_{21}{}^2)^{1/2}$$

where L_{21} is the distance from the observer to PC.

$$S_2 = R_1 \sin(I_2) = (R - m) \sin(I_2) \qquad \text{(E-36)}$$

Hence

$$S_e = S_1 + S_2 = \sin(I_1 - I_2)(R^2 + L_{21}{}^2)^{1/2} + (R - m) \sin(I_2) \qquad \text{(E-37)}$$

From trigonometry we obtain I_1:

$$I_1 = 180(L_{02})/(\pi\ R) + \tan^{-1}(L_{21}/R) \qquad \text{(E-38)}$$

I_2 can be obtained from Figure E-15 using h:

$$\begin{aligned} h &= S_1/(\tan(I_1-I_2)) \\ &= (R^2 + L_{21}{}^2)^{1/2} \sin(I_1 - I_2)/[\tan(I_1 - I_2)] \\ &= (R^2 + L_{21}{}^2)^{1/2} \cos(I_1 - I_2) \qquad \text{(E-39)} \end{aligned}$$

and

$$h = S_2/\tan(I_2) = (R - m) \sin(I_2)/\tan(I_2) = (R - m) \cos(I_2) \qquad \text{(E-40)}$$

Hence by setting these expressions for h:

$$h = (R^2 + L_{21}{}^2)^{1/2} \cos(I_1-I_2) = (R - m) \cos(I_2) \qquad \text{(E-41)}$$

Solving this equation for I_2, we obtain

$$I_2 = \tan^{-1}\{[R - m - \{R^2 + L_{21}{}^2)^{1/2} \cos(I_1)]/[(R^2 + L_{21}{}^2)^{1/2} \sin(I_1)]\} \tag{E-42}$$

<u>Sight distance component S_o</u> from the obstacle to the object is desired using the results obtained in Eq. E-27 and the Figure E-14. Thus, we obtain for S_o:

$$S_o = Z - S_2 \tag{E-43}$$

where

$$Z = (R^2 - h^2)^{1/2} = [R^2 - (R^2 + L_{21}{}^2) \cos^2(I_1 - I_2)]^{1/2} \tag{E-44}$$

and, hence, inserting the expressions for Z and S_2:

$$S_o = [R^2 - (R^2 + L_{21}{}^2) \cos^2(I_1 - I_2)]^{1/2} - (R - m) \sin(I_2)$$

$$S_o = [(R^2 + L_{21}{}^2) \sin^2(I_1 - I_2) - L_{21}{}^2]^{1/2} - (R - m) \sin(I_2) \tag{E-45}$$

<u>The total sight distance S</u> for the case when observer has not yet passed the PC and the object is in the curve is derived using the results obtained in Eqs. E-37 and E-45 and Figure E-14. Thus, we obtain for total sight distance S:

$$S = S_e + S_o$$

$$= (R^2 + L_{21}{}^2)^{1/2} \sin(I_1 - I_2) + (R - m) \sin(I_2)$$

$$+ [(R^2 + L_{21}^2) \sin^2(I_1 - I_2) - L_{21}^2]^{1/2} - (R - m) \sin(I_2)$$

and hence

$$S = (R^2 + L_{21}^2)^{1/2} \sin(I_1 - I_2) + [(R^2 + L_{21}^2) \sin^2(I_1 - I_2) - L_{21}^2]^{1/2} \quad \text{(E-46)}$$

where I_1 and I_2 are as given in Eqs. E-38 and E-42. When L_{22}=0, Eq. E-46 reduces to the sight distance formula derived in Case 1.

<u>Case 3: Observer in the Curve and Object Beyond the PT of the Curve</u>. Case 3 and the notation used are presented in Figure E-15. The derivation is similar to that for Case 2 with only the locations of the observer and the object being interchanged. The results are as follows.

<u>Sight distance component S_e</u>, from the observer to the obstacle:

$$S_e = S_1 + S_2 = R \sin(I_1 - I_2) + (R - m) \sin(I_2) \quad \text{(E-47)}$$

where

$$I_1 = 180(L_{01})/(\pi R) \quad \text{(E-48)}$$

and

$$I_2 = \tan^{-1}\{[R(1 - \cos(I_1)) - m]/[R \sin(I_1)]\} \quad \text{(E-49)}$$

<u>Sight distance component S_o</u>, from the obstacle to the object:

$$S_o = Z - S_2$$

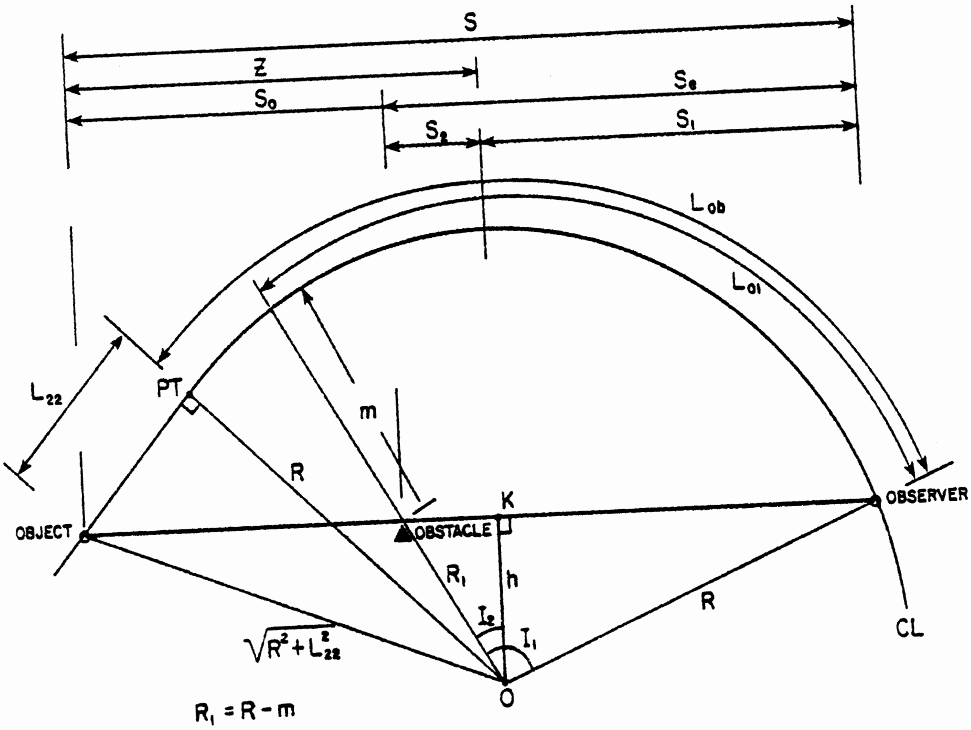


Figure E-15. Observer in the curve and object beyond the PT (Case 3).

$$= \frac{[R^2 \sin^2(I_1 - I_2) + L_{22}{}^2]^{1/2} - (R - m)}{\sin(I_2)} \qquad \text{(E-50)}$$

Total sight distance S, when the observer is in the curve and the object beyond the PT, is obtained using the results obtained in Eqs. E-47 and E-50. Thus, we get

$$S = S_e + S_o$$

$$= R \sin(I_1 - I_2) + [R^2 \sin^2(I_1 - I_2) + L_{22}{}^2]^{1/2} \qquad \text{(E-51)}$$

where I_1 and I_2 are as given in Eqs. E-48 and E-49.

When $L_{22} = 0$, Eq. E-51 reduces to the sight distance formula case 1.

Case 4: Observer Before the PC and Object Beyond the PT. In this case, both the observer and the object are off of the horizontal curve. Case 4 and the notation used are shown in Figure E-16. The derivation is similar to those described previously.

Sight distance component S_e, from the observer to the obstacle is obtained from Figure E-16. Thus:

$$S_e = S_1 + S_2 \qquad \text{(E-52)}$$

where

$$S_1 = (R^2 + L_{21}{}^2)^{1/2} \sin(I_1 - I_2) \qquad \text{(E-53)}$$

$$S_2 = R_1 \sin(I_2) = (R - m) \sin(I_2) \qquad \text{(E-54)}$$

L_{21} is the distance from the observer to the PC and hence we obtain

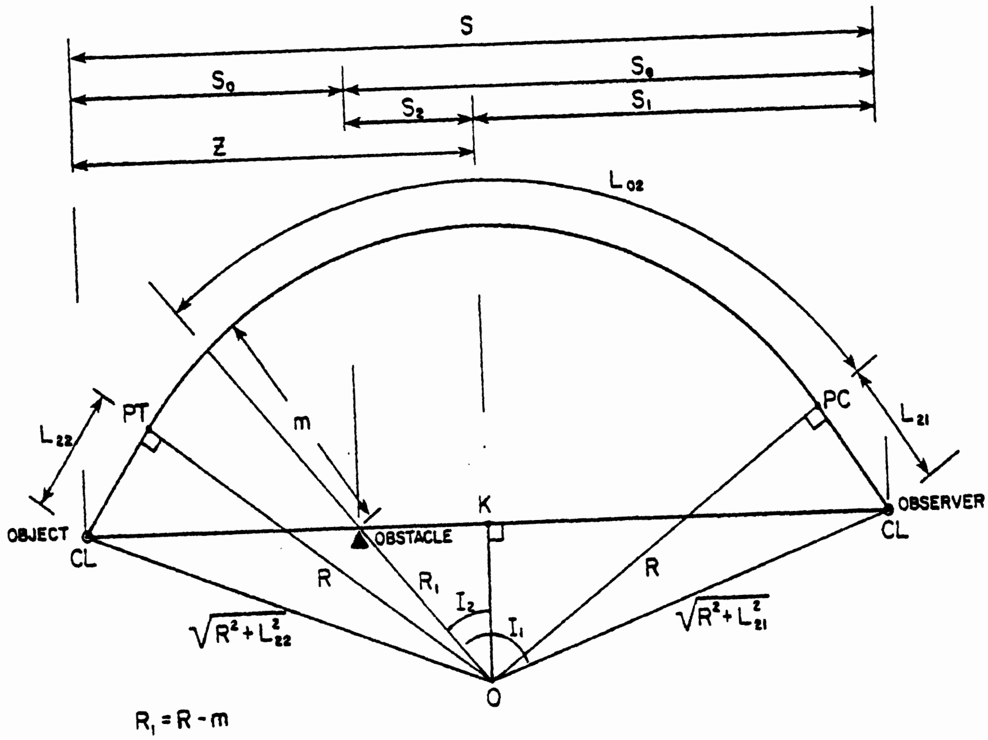


Figure E-16. Observer before the PC and object beyond the PT (Case 4).

$$S_e = (R^2 + L_{21}{}^2)^{1/2} \sin(I_1 - I_2) + (R - m) \sin I_2 \quad \text{(E-55)}$$

where:

$$I_1 = 180(L_{02})/(\pi R) + \tan^{-1}(L_{21}/R) \quad \text{(E-56)}$$

$$I_2 = \tan^{-1}\{[R - m - (R^2 + L_{21}{}^2)^{1/2} \cos(I_1)]/[(R^2 + L_{21}{}^2)^{1/2} \sin(I_1)]\} \quad \text{(E-57)}$$

L_{02} is the distance from the PC to the station where the obstacle is located.

<u>Sight distance component S_o</u>, from the obstacle to the object, is obtained from Figure E-16. Thus:

$$
\begin{aligned}
S_o &= Z - S_2 \\
&= (R^2 + L_{22}{}^2 - h^2)^{1/2} - S_2 \\
&= [R^2 + L_{22}{}^2 - (R^2 + L_{21}{}^2) \cos^2(I_1 - I_2)]^{1/2} - S_2
\end{aligned}
$$

and hence

$$S_o = [L_{22}{}^2 - L_{21}{}^2 + (R^2+L_{21}{}^2) \sin^2(I_1 - I_2)]^{1/2} - (R - m) \sin(I_2) \quad \text{(E-58)}$$

where L_{21} is as defined earlier, L_{22} is the distance from the PT to the object, and I_1 and I_2 are as given in Eqs. E-56 and E-57.

<u>Total sight distance S</u>, for the case when both observer and object are off of the curve, is obtained using the results from in the two preceding parts of this section and Figure E-16. Thus, we obtain for total sight distance S:

$$S = S_e + S_o$$
$$= (R^2 + L_{21}{}^2)^{1/2} \sin(I_1 - I_2)$$
$$+ [L_{22}{}^2 - L_{21}{}^2 + (R^2 + L_{21}{}^2) \sin^2(I_1 - I_2)]^{1/2} \tag{E-59}$$

By setting $L_{22} = 0$ we obtain case 2; when setting $L_{21} = 0$, we obtain case 3; and setting both L_{21} and $L_{22} = 0$ gives the results for case 1.

Hence, the sight distance Eq. E-59 can be called the "General Sight Distance Formula for Horizontal Curves" covering all cases of available sight distance with respect to locations of observer, obstacle, and object.

Calculation Procedure for Sight Distances in Horizontal Curves

Inspection of Figures E-13 to E-16 reveals the geometric relationship of the locations of the observer, obstacle, and object. When the observer moves, the farthest point the observer still can see also changes. The magnitude of the available sight distance change clearly depends on the road geometry.

In this part a four-step calculation procedure for S is given for three cases and their three subcases are described. The three cases are:

1. Observer before the PC (or PT).
2. Observer at the PC (or PT).
3. Observer in the curve.

For these locations three positions of the object must be considered:

4. Object beyond the PT (or PC).
5. Object opposite the PT (or PC).
6. Object inside the curve.

The calculation procedure requires the following four steps:

1. Determine the angles I_1 and I_2.
2. Determine I_{ob} (see Figs. E-15 and E-16).
3. Determine the location of the object with respect to the PT.
4. Determine the total available sight distance S.

To calculate the sight distance S, the geometric characteristics of the horizontal curve ,R, L, m, and I, must be known or measured. Similarly for a desired stopping sight distance SSD the required lateral clearance m for given R and L can be calculated; or in design, the required curve radius R given m, S, and I.

<u>Step 1</u>--<u>Determine I_1 and I_2</u>

o Observer before the PC ($L_{21} > 0$):

$$I_1 = 180(L_{02})/(\pi R) + \tan^{-1}(L_{21}/R) \qquad \text{(E-60)}$$

$$I_2 = \tan^{-1}([R-m-\{R^2 + L_{21}{}^2)^{1/2} \cos(I_1)]/ \\ [(R^2 + L_{21}{}^2)^{1/2} \sin(I_1)]\} \qquad \text{(E-61)}$$

- Observer at the PC:

$$I_1 = 180(L_{02})/(\pi R) \quad \text{(E-62)}$$

$$I_2 = \tan^{-1}\{[R(1 - \cos(I_1)) - m]/[R \sin(I_1)]\} \quad \text{(E-63)}$$

- Observer in the curve (L_{21}=0):

$$I_1 = 180(L_{01})/(\pi R) \quad \text{(E-64)}$$

$$I_2 = \tan^{-1}\{[R(1 - \cos(I_1)) - m]/[R \sin(I_1)]\} \quad \text{(E-65)}$$

<u>Step 2</u>--<u>Calculate</u>(I_{ob})

- Observer before the PC (L_{21} > 0):

$$I_{ob} = 180L/(\pi R) + \tan^{-1}(L_{21}/R) \quad \text{(E-66)}$$

- Observer at the PC:

$$I_{ob} = 180L/(\pi R) \quad \text{(E-67)}$$

- Observer in the curve:

$$I_{ob} = 180(L_{ob})/(\pi R) \quad \text{(E-68)}$$

<u>Step 3--Test Location of Object with Respect to PT Using Values Obtained in Steps 1 and 2</u>

- Object is beyond the PT if and only if: $I_{ob} < 2(I_1-I_2)$
- Object is at PT if and only if: $I_{ob} = 2(I_1-I_2)$
- Object is in the curve if and only if: $I_{ob} > 2(I_1-I_2)$

<u>Step 4--Calculation of Total Sight distance S Using General Sight Distance Formula</u>

- Observer before the PC: Set L_{21} equal to the measured distance from the observer to the PC. If object is beyond the PT, then calculate (L_{22}):

$$L_{22} = (R \cos(I') - h) / \sin(I')$$

where $I' = I_{ob} - (I_1 - I_2)$ and $h = (R - m) \cos(I_2)$,
and I_{ob} is obtained from Eq. E-66 and I_1 and I_2 are obtained from Eqs. E-60 and E-61. Otherwise, set L_{22}=0 and obtain I_1 and I_2 from Eqs. E-60 and E-61.

- Observer at the PC: Set L_{21}=0. If object is beyond the PT, then calculate L_{22}:

$$L_{22} = (R \cos(I') - h) / \sin(I') \qquad \text{(E-69)}$$

where $I' = I_{ob} - (I_1 - I_2)$ and $h = (R - m) \cos(I_2)$,
and I_{ob} is obtained from Eq. E-67 and I_1 and I_2 are from Eqs. E-62 and E-63. Otherwise, set L_{22}=0 and obtain I_1 and I_2 from Eqs. E-62 and E-63.

- Observer in the curve: Set L_{21}=0. If object is beyond the PT, then calculate L_{22}:

$$L_{22} = (R \sin(I') - h) / \sin(I') \qquad \text{(E-70)}$$

where $I' = I_{ob} - (I_1 - I_2)$ and $h = (R-m) \cos(I_2)$, and I_{ob} is from Eq. E-68 and I_1 and I_2 are from Eqs. E-64 and E-65. Otherwise, set L_{22}=0 and obtain I_1 and I_2 from Eqs. E-64 and E-65.

Calculation of Critical Lateral Clearance Values

With the procedure described above the available sight distance can be calculated for any combination of locations of observer and obstacle in the curve. Sometimes, for example in design, it is only desired to obtain critical lateral clearance values, m. These are the lateral clearance values corresponding to a position of the line of sight requiring the greatest lateral clearance as shown in Figure E-18. Formulas are developed below for three cases:

1. $S < L, I^* < I$
2. $S = L, I^* = I$
3. $S > L, I^* > I$

to calculate those critical lateral clearance values m_{max} (for notation see Fig. E-17). Alternatively for cases 1 and 2, $S<=L$ and $I^*<=I$ (AASHTO (5 Figures on pp. 188 and 367)) can be used; and for case 3, $S > L$, $I^* > I$ (Fig. E-18) can be used to determine the critical m-values.

1. $S < L$ and $I^* < I$. Calculate m_{max} using the formulas below or use AASHTO (5, Figures on pp. 188 and 367):

$$m_{max} = R\ (1 - \cos(I^*/2))$$

$$S = (\pi\ R\ I^*)/180 \quad \text{for } I^* < I$$

$$R_1 = R - m_{max}$$

Then

$$m_{max} = R\ [1 - \cos(90S/(\pi\ R))] \tag{E-71}$$

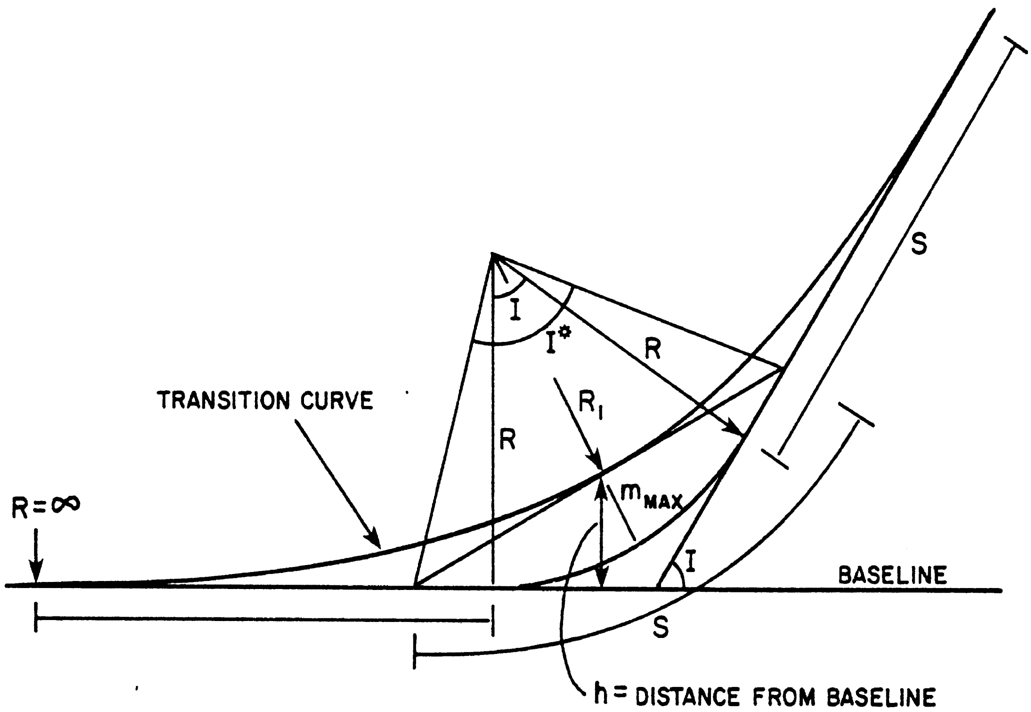


Figure E-17. Transition curve for lateral clearance; radius from R =inf. to R with lateral displacement at the critical point m_{max}, for I*>I.

Maximum lateral clearance m takes a constant value in a section of a and starts at b measured from the start of the curve, where

$$a = \pi R\ (I - I^{*})/180$$

$$b = \pi R\ I^{*}/360$$

The starting and end points of the section with maximum lateral clearance m are connected to points located at the required sight distance before and after the curve using transition curves.

Transition curve from R = infinity to $R = R_1$

Distance from the baseline at R_1: $h = R - R_1 \cos(I^{*}/2)$

2. <u>$S = L$ and $I^{*} = I$</u>. Calculate m_{max} using the formulas below or use AASHTO(5, Figures on pp. 188 and 367):

$$m_{max} = R\ (1 - \cos(I/2)) \qquad \text{(E-72)}$$

$$S = \pi R\ I/180 \quad \text{for } I^{*} = I$$

$$R_1 = R - m_{max}$$

The maximum value for m occurs at the middle point of the curve. This point is connected to points located at the sight distance before and after the curve using transition curves.

Transition curve from R = infinity to $R = R_1$

Distance from the base line at R_1 : $h = R - R_1 \cos(I/2)$

3. <u>$S > L$ and $I^* > I$</u>. Calculate m_{max} using the formulas below or use Figure E-18.

$$m_{max} = R \tan[(I^*-I)/2] \sin(I/2) + [R(1 - \cos(I/2)] \quad \text{(E-73)}$$

$$S = S_s + \pi R I/180 \quad \text{for } I^* > I$$

where S_s is the sight distance component on the tangent $R_1 = R - m_{max}$.

The maximum value for m occurs at the middle point of the curve. This point is connected to points located at the required sight distance before and after the curve using transition curves.

Transition curve from R = infinity to $R = R_1$

Distance from the base line at R_1 : $h = R - R_1 \cos(I/2)$

Example of the use of Figure E-18:

1. Determine required sight distance S. Check that $I^* > I$ and $S > L$.
2. Enter the diagram with the difference between the sight distance and the curve length, $S - L$ (in feet), and read for appropriate curve radius R or curvature D the $(I^* - I)/2$ value.
3. With this $(I^* - I)/2$ value, enter the right bottom part of the figure and using the central angle I of the curve read for the radius R the critical lateral clearance m (in ft), (or read as m-value the scale with m as a fraction of the radius R and multiply

this m-value by radius R to obtain the critical value of m in ft).

(Example: S - L = 300 ft, R=800 ft, I = 15, follow the dashed line and read m = 0.033 R or m = 26.5 ft.)

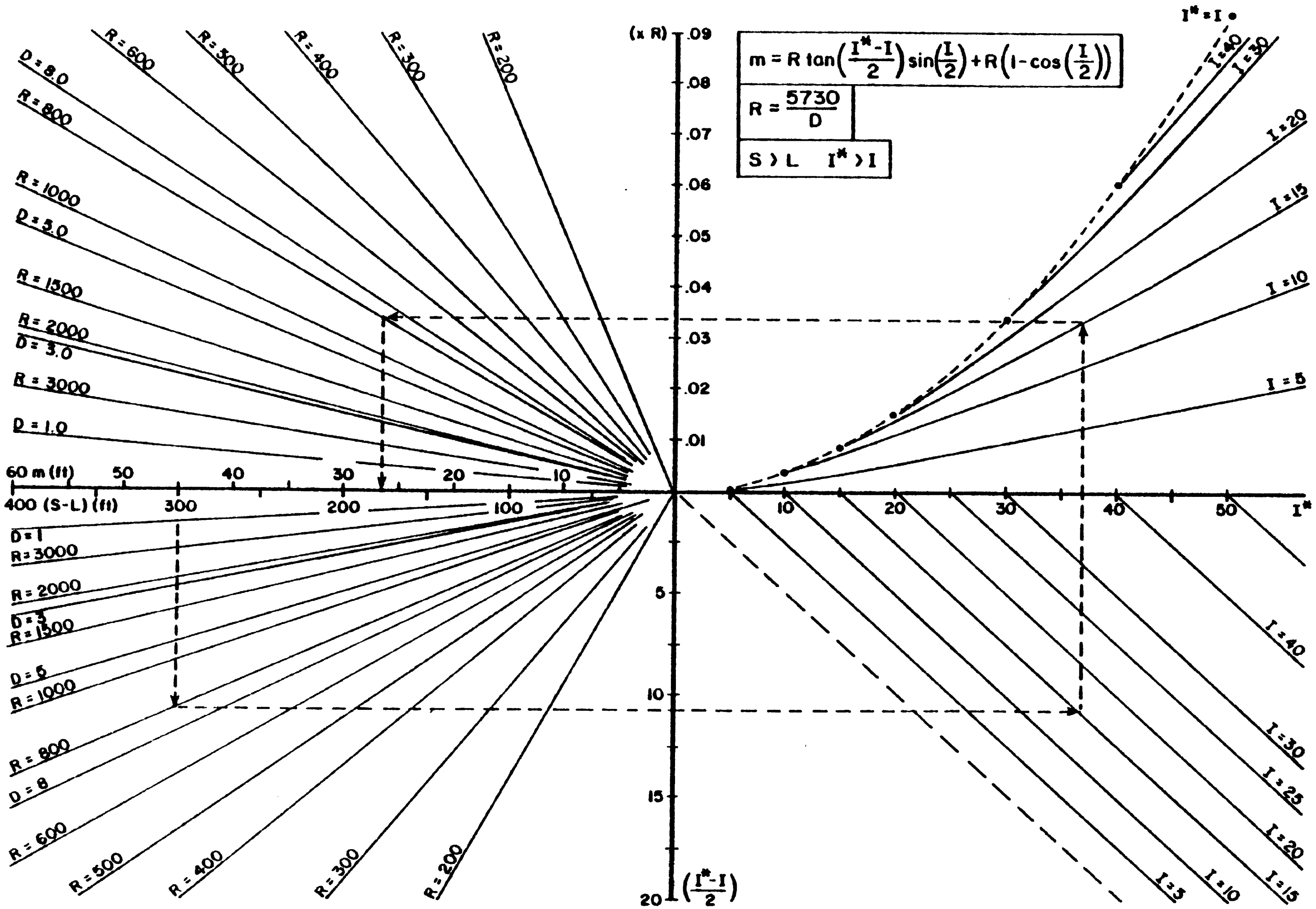


Figure E-18. Critical lateral clearance m as a function I, R, L and S.

Chord Approximation for SSD on Horizontal Curve

In this closer approximation to the true SSD the arc of the sight distance along the centerline of the horizontal curve is not used, but rather the two chords, each subtending S/2 as shown in Figure E-19(a) and (b), are used. (In Figure E-19(c) the two chords subtend L/2. The derivation is given in the following.

$S < L$. From Figure E-19(a), by the right triangle relationships

$$(c/2)^2 + M^2 = (S/2)^2 \tag{E-74}$$

and

$$(c/2)^2 + (R - M)^2 = R^2 \tag{E-75}$$

Solving for c/2 in Eq. E-74 and substituting in Eq. E-75

$$2RM - M^2 + M^2 = S^2/4$$

Simplifying

$$M = S^2/8R \tag{E-76}$$

$S = L$. From Figure E-19(b) and the case above

$$O = S^2/8R = L^2/8R \tag{E-77}$$

$S > L$. From Figure E-19(c)

$$S = L + 2T \tag{E-78}$$

By the right triangle relationships

$$(S/2)^2 = K^2 + m^2 \tag{E-79}$$

and

$$K^2 + (R-m)^2 = E^2 \tag{E-80}$$

and

$$T^2 + R^2 = E^2 \tag{E-81}$$

Eliminating K, E, and T from Eqs. E-78 through E-80, we obtain

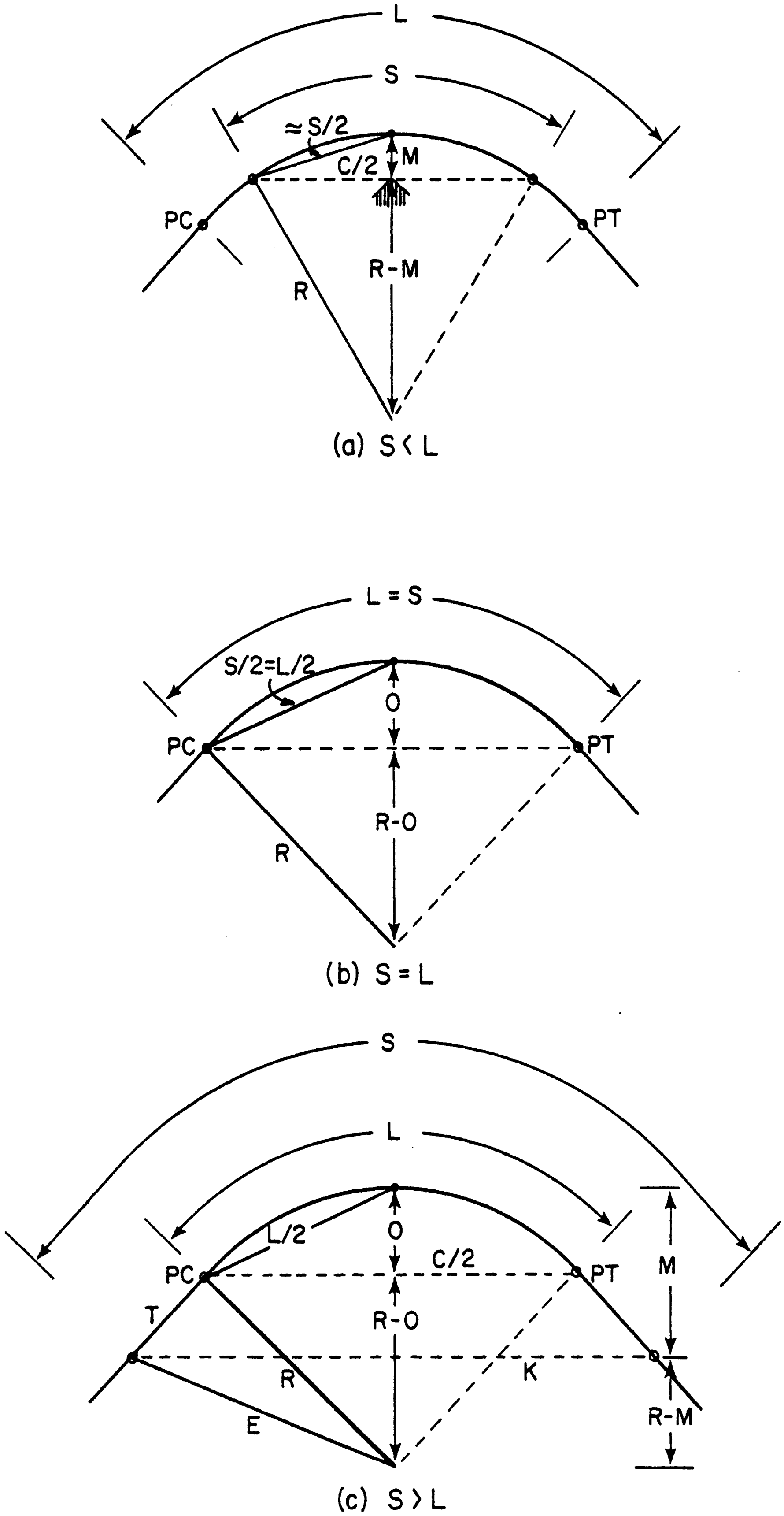


Figure E-19. Chord approximation for SSD on horizontal curves; (a) S<L, (b) S=L, (c) S>L.

$$m = L(2S-L)/(8R) \tag{E-82}$$

When S = L, Eq. E-77 can be obtained from Eq. E-76 or Eq. E-81.

For the case where S > L we can compare the result with the case where S < L by the ratio

$$m/M = [L(2S-L)/8R]/[S^2/(8R)] = (L/S)(2-L/S) \tag{E-83}$$

This can be seen independent of R. Tabulating this result we have

L/S	m/M
0.1	0.19
0.2	0.36
0.3	0.51
0.4	0.64
0.5	0.75
0.6	0.84
0.7	0.91
0.8	0.96
0.9	0.99
1.0	1.00

For example, consider a 2-deg (R about 2865 ft) curve 900 ft long with a required S = 1150 ft, then

$$M = 1150^2/[(8)(2865)] = 58 \text{ ft}$$

$$m/M = (900/1150)[2-(900/1150)] = 0.953$$

$$m = (0.953)(58) = 55 \text{ ft}$$

The elasticity of the offset, M, with respect to S for S < L is 2, indicating that a 1 percent increase in sight distance increases M by 2 percent. In the case where S > L the change is always greater than 1 percent and equals 2S(2S - L) percent.

Table E-13 shows that the chord approximation introduces error in SSD of 0.5 ft or greater, only for curves with a radius less than 400 ft.

Determination of Volume of Excavation

Referring to Figure E-20 one can write

$$dV = (Wy + Ry^2)\ dx$$

where:

dV = the differential volume of excavation;

W = the width of the road bed;

R = the side slope (horizontal:1);

V = $(aL^2/8)(W/3 + RaL/40)$; and

y = $ax^2/(2L)$.

Since L is proportional to SSD squared when SSD < L, for this common case V is proportional to more than SSD raised to the fourth power. When SSD > L, V is proportional to SSD to more than SSD squared.

Variance of SSD Required for Stopping

SSD is defined as

$$SSD = 1.47\ V(PRT) + V^2/(30f)$$

Under conditions of independence of the parenthesized variables and using a Taylor-series the mean and variance of SSD can be expressed by

$$E(SSD) = V\mu_{PRT} + V^2/(30\mu_f) + V^2\sigma_f{}^2/(30\mu_f{}^3)$$

$$VAR(SSD) = 2.16V^2\sigma_{PRT}{}^2 + V^4\sigma_f{}^2/(900\mu_f)$$

Table E-13. Comparison of exact and approximate formulas for required clearance.

R (ft)	S (ft)	$S^2/8R$	M (ft) R(1-cos(28.65 S/R))	Error (ft)
200	120	9.0	8.9	0.1
	200	25.0	24.5	0.5
	275	47.3	45.4	1.8
300	120	6.0	6.0	0.0
	200	16.7	16.5	0.2
	275	31.5	31.0	0.5
	375	58.6	56.7	1.9
400	200	12.5	12.4	0.1
	275	23.6	23.4	0.2
	375	43.9	43.5	0.4
800	275	11.8	11.8	0.0
	375	22.0	21.9	0.1
	525	43.1	42.7	0.4
1000	275	9.5	9.4	0.1
	375	17.6	17.5	0.1
	525	34.5	34.3	0.2
	625	48.8	48.4	0.4
1400	375	12.6	12.5	0.1
	525	24.6	24.5	0.1
	625	34.9	34.7	0.2
	750	50.2	49.9	0.3
1900	625	25.7	25.6	0.1
	750	37.0	36.9	0.1
	850	47.5	47.3	0.2
5700	625	8.6	8.6	0.0
	750	12.3	12.3	0.0
	850	15.8	15.8	0.0
	1100	26.5	26.5	0.0

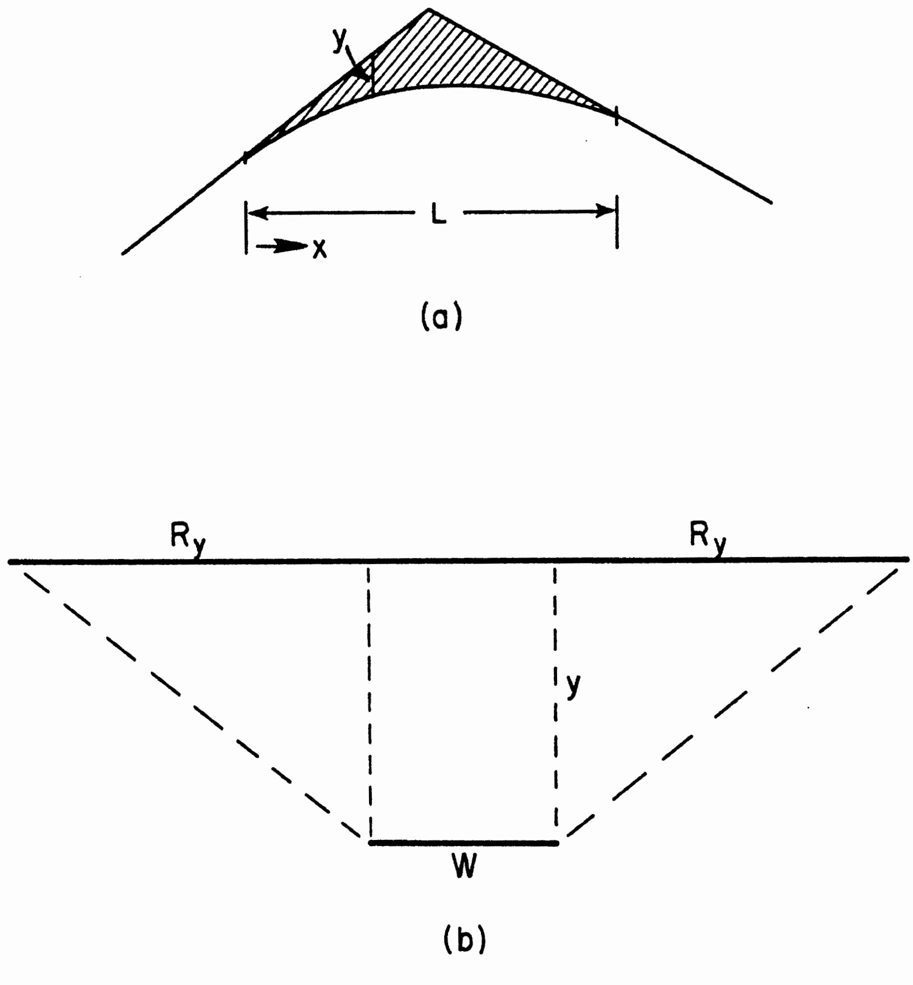


Figure E-20. Determination of volume of excavation.

where μ_{PRT} and μ_f are averages of the distributions of the individual variables, V is the initial speed in miles per hour, and $\sigma_{PRT}{}^2$ and $\sigma_f{}^2$ are the variance of the indicated variables.

The importance of the initial speed is clear.

APPENDIX F

VEHICLE ACCELERATION CHARACTERISTICS
INFLUENCING HIGHWAY DESIGN

INTRODUCTION

This appendix presents findings concerning the interrelationships between the acceleration characteristics of highway vehicles and highway design policies. This work provides an evaluation of acceleration characteristics thereby supplementing the investigation of deceleration characteristics included in Project 15-8 conducted under the auspices of the National Cooperative Highway Research Program (NCHRP).

The main thrust of Project 15-8 addresses stopping sight distance; and, in that regard, the influences of vehicle deceleration characteristics on braking distance are examined elsewhere in this report.

The discussion of acceleration characteristics may be conveniently separated from a discussion of deceleration characteristics, because acceleration and deceleration are achieved using entirely different mechanical devices, namely the engine and the brakes. Nevertheless, both of these devices are controlled by drivers, thereby adding elements of driver skill and "taste" to the performance of the driver-vehicle system--the system of interest to the highway engineer. In some situations, the performance of the driver-vehicle system is limited primarily by vehicle and highway characteristics, for example, climbing a steep grade with a heavy truck. In contrast, driver comfort (or taste) determines driver-vehicle system performance in acceleration maneuvers that do not challenge the performance limits imposed by the mechanical properties of vehicle components or the friction available at the tire-road interface. Furthermore, traffic conditions influence how a particular vehicle is operated. Within this discussion of the acceleration subject, an attempt is made to distinguish situations in which driver and/or vehicle factors contribute to the findings presented.

Part of the material presented pertains to vehicle or component performance. This performance can be predicted more accurately than the performance of the driver-vehicle system. Generally representative observations of the performance of the driver-vehicle system may be difficult to make, and they frequently involve uncertainties that require statistical evaluation. The approach taken here has been to apply (1) principles of physics and (2) data from component measurements to predict vehicle performance. Where possible, these predictions are compared (and sometimes "calibrated") using the results of full-scale vehicle tests or observations of vehicles in use.

The subjects discussed with respect to acceleration characteristics are:

1. The basic physical factors influencing acceleration performance.

2. Those aspects of the proposed AASHTO (American Association of State Highway and Transportation Officials) geometric design policy (F-1) involving acceleration (propulsion).

3. Comparisons between the numerical values used in the design policy and numerical values based on the acceleration characteristics of the current vehicle fleet.

4. Suggestions or insights to be considered in developing new design charts.

ACCELERATION

Basic Factors Influencing Acceleration Performance

The acceleration performance of pneumatic-tired vehicles depends on the difference between the power available from the engine and the power required to overcome resistance to motion. So called "natural" sources of retardation include rolling resistance in the tires, aerodynamic drag, and rolling resistance or inefficiency in the driveline (i.e., chassis friction). In addition to overcoming natural retardation, the engine supplies the power needed to increase velocity and/or climb hills; in other words, the power needed to increase the kinetic and/or potential energy of the vehicle.

The engines employed in highway vehicles may be roughly considered as nearly constant torque devices when operated at typical ranges of engine speed. As illustrated in the following simplified development, an interpretation of the implications of constant torque (or an upper bound on power) is fundamental to understanding the relationship between vehicle speed and acceleration capability.

To first approximation engine power is the product of propulsive force and speed, i.e.,

$$P_e = F_p V \tag{F-1}$$

where P_e is power, F_p is propulsive force, and V is forward velocity.

Second, force equals mass times acceleration

$$F = MA \tag{F-2}$$

where F is force, M is mass, and A is acceleration.

At the beginning of this development, ignore any natural retardation and grade influences such that $F = F_p$. (Natural retardation and grade influences will be considered later, after basic notions concerning power-to-weight ratio have been presented.) By combining Eqs. F-1 and F-2, the following result is obtained:

$$A = P_e/(MV) \tag{F-3}$$

As illustrated in Figure F-1, Eq. F-3 shows that acceleration (that is, the maximum upper bound on acceleration) decreases in a manner that is inversely proportional to the forward speed of a vehicle of a specified mass equipped with an engine with a given power capability.

For a particular vehicle, the power-to-mass ratio "scales" the acceleration-to-velocity relationship (see Fig. F-1). This power-to-mass scale factor (often referred to as a horsepower-to-weight ratio) provides a first order

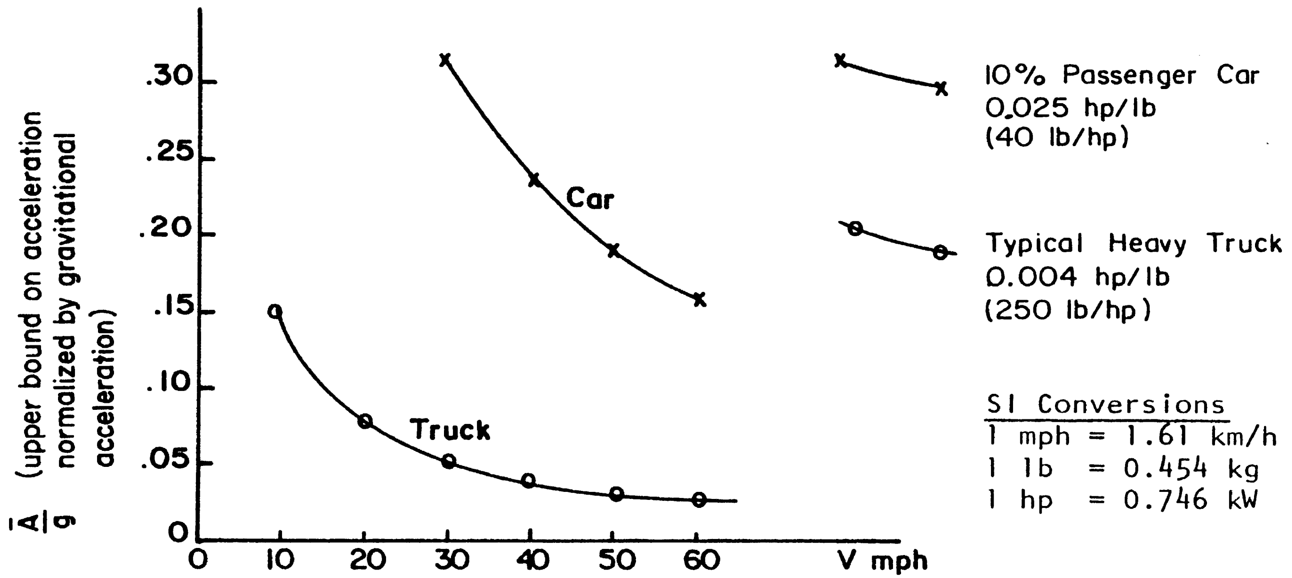


Figure F-1. The influence of velocity on acceleration as determined by power-to-weight ratio.

indication of the relative acceleration capabilities of various highway vehicles.

Clearly, the actual acceleration performance of a vehicle depends on its natural retardation. The net propulsive force, F_p, is opposed by rolling resistance, F_r, and aerodynamic drag, F_a, in straightline motion on a level roadway (see Fig. F-2.).

For zero acceleration (the condition for sustaining speed), the net force on the vehicle is zero, that is,

$$F_p = F_a + F_r \tag{F-4}$$

Figure F-3 presents curves showing how F_p, F_a, and F_r vary with velocity. The point at which the total drag ($F_d = F_a + F_r$) equals the net propulsive force, F_p, determines the maximum sustained speed, V_s (see Fig. F-3).

In Figure F-3 natural retardation has been broken down into 3 components:

1. F_{rt}, a tire rolling-resistance force that depends on vertical load but is independent of velocity.
2. F_{rc}, a chassis friction term that is conventionally represented as a linear function of velocity.
3. F_a, an aerodynamic drag force that depends on the velocity squared.

In addition to these components, upgrades produce another drag force that depends on vehicle weight, but is independent of velocity. Obviously, the maximum sustained

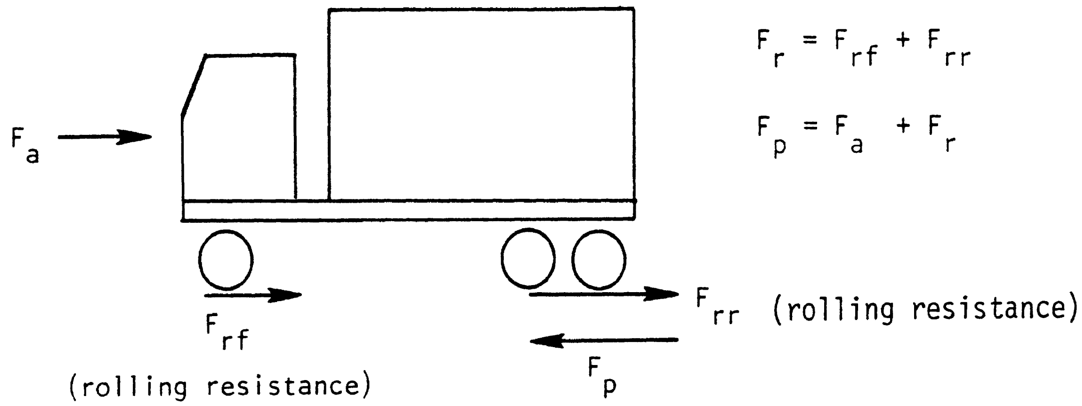


Figure F-2. Force balance for sustained speed.

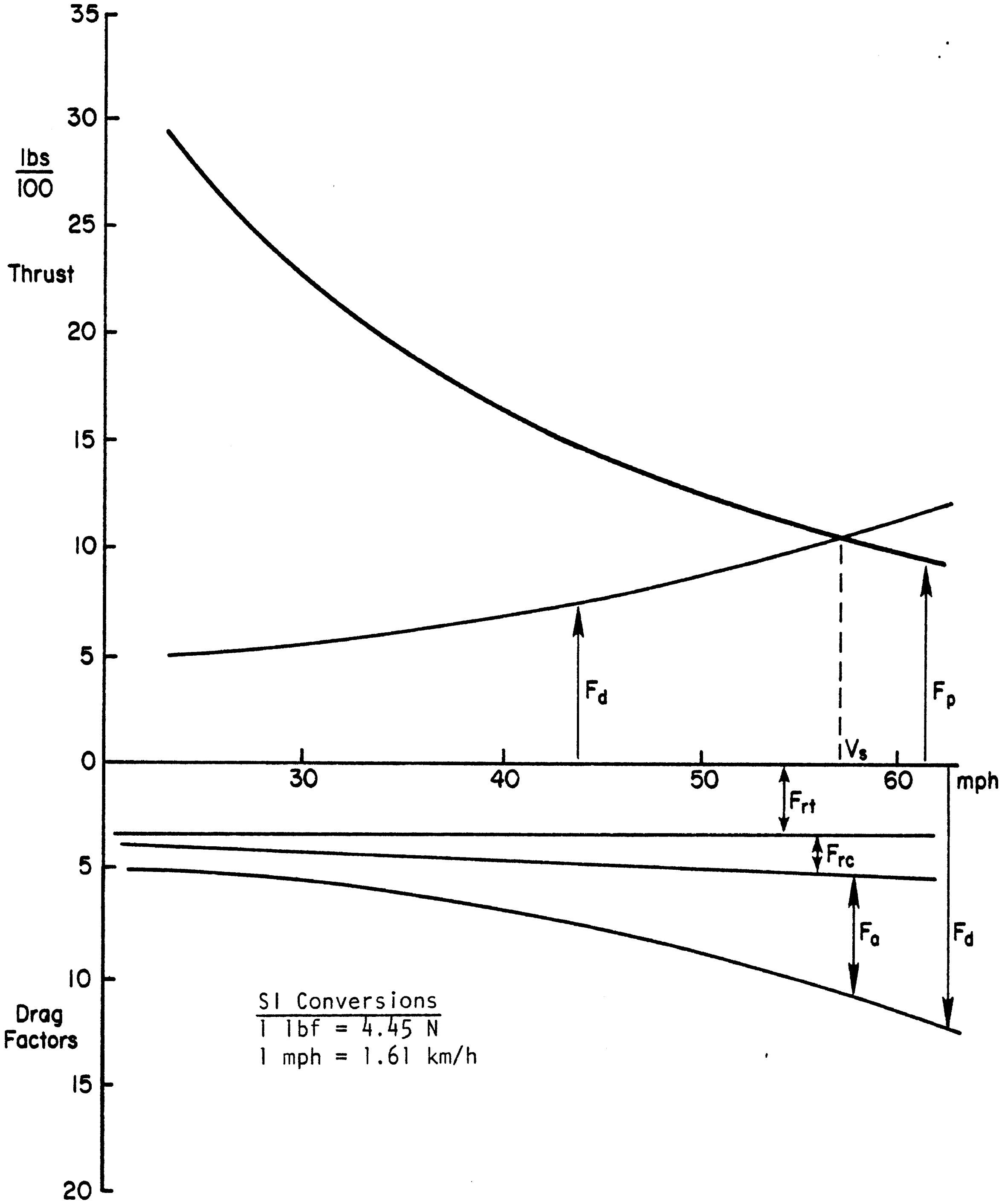


Figure F-3. Influence of velocity on force components.

speed on an upgrade is less than that on a level road; however, the amount of speed reduction caused by an upgrade varies (to first order) in a manner that is inversely proportional to the levels of velocity involved because of the relationship between propulsive force and velocity.

When the forces are balanced a vehicle sustains speed, neither accelerating nor decelerating; however, when the forces are unbalanced, the vehicle will accelerate by an amount depending on the inertias involved. There are two types of inertia to consider: (1) the mass of the vehicle, and (2) the rotational inertia of the drive system. In acceleration analyses, these two types of inertia are often combined into a single effective mass, m_e (or an equivalent weight, W_e). At highway speeds the contribution of the rotational inertia to m_e may be approximately 3 to 4 percent of the total; while at low speed the effective mass may be on the order of 1.5 times the actual mass. Because of the high gear ratios associated with low gears (that is, high effective mass), the low speed acceleration capability of a vehicle may be much less than that implied by the power-to-mass ratio.

For the purposes of the highway engineer, acceleration performance is often described through graphs or tables of velocity versus distance. The velocity versus distance performance of a vehicle can be derived from acceleration versus velocity information as follows:

1. Use Newton's laws of motion to find acceleration:

$$A = (dv/dt)=(F/m_e) \qquad \text{(F-5)}$$

where A is acceleration (or deceleration if $A < 0$), V is velocity, F is the net force which is a function of velocity, m_e is the effective mass, and d/dt represents the time rate of change of a variable

2. Solve Eq. F-5 to obtain velocity as a function of time. (This can be done theoretically if F is a simple function of velocity, or it can be done numerically.)

3. Since

$$V = d\ (d)/dt \qquad \text{(F-6)}$$

that is, velocity is the time rate of change of distance, d, integrate Eq. F-6 to obtain distance as a function of time.

4. Using time to find corresponding pairs of speed-distance points, construct curves of velocity versus distance.

In summary, once the acceleration versus velocity characteristic of a vehicle has been determined, knowledge of elementary calculus can be used to obtain velocity and/or distance information.

Although the acceleration capability of a vehicle changes with velocity, first order estimates of acceleration performance for small speed changes are often made using a "constant" acceleration analysis. In this type of analysis an average acceleration is used to approximate the portion of the acceleration function existing between two speeds. The equations resulting from this type of analysis are readily derived from simple integrations with respect to time. The results are:

1. With respect to elapsed time, T,

$$T = (V_f - V_i)/A \quad \text{(F-7)}$$

where V_f is the final velocity, V_i is the initial velocity, and A is a constant level of acceleration (or deceleration).

2. With respect to total distance, d,

$$d = V_i\ T + 1/2\ A\ T^2 \quad \text{(F-8)}$$

$$\text{or } d = (V_f^2 - V_i^2)/2A \quad \text{(F-9)}$$

(Equation F-9 can be interpreted and derived from a work/ energy balance, i.e., Work = Fd = mAd = $m(V_f^2-V_i^2)/2$.

In closing this general discussion two observations aid in providing a perspective with regard to (1) driver controlled acceleration performance and (2) braking performance. First, clearly the constant acceleration analysis applies to situations in which the driver chooses to use something less than the acceleration capability of the vehicle. Provided information on "normal" acceleration is available, the performance of the driver-vehicle system can be analyzed according to Eqs. F-7, F-8, and F-9. In particular, the acceleration used from a standing start is usually chosen by the driver. In this case, the relationship between distance traveled and time elapsed is given by the following version of Eq. F-8: $d = 1/2\ AT^2$ in which A is the driver's acceleration characteristic for the type of vehicle involved.

Second, the foundation brake used in motor vehicles is, to first order, a constant torque (or brake force) device when a brake line pressure is applied. Hence, a constant acceleration analysis is often used in estimating braking performance. A version of Eq. F-9, commonly employed in estimating stopping distance, is as follows:

$$d = V_i^2/(2D) \qquad \text{(F-10)}$$

where d is the stopping distance, V_i is the initial velocity, and D is deceleration.

Those Aspects of Geometric Design Policy Influenced by Acceleration Characteristics

The 1981 AASHTO policy draft report (F-1) has been reviewed to identify road designer needs for acceleration and deceleration data. (This work was performed by Prof. D.E. Cleveland of the Civil Engineering Department at the University of Michigan (F-2)). The "standard" applications, in which acceleration characteristics are used, include (1) enhancing the uniformity of vehicle operating speeds on grades, (2) determining the length of acceleration lanes for entrance terminals, (3) providing adequate sight distance for accelerating across intersections, and (4) providing adequate sight distance for passing on two-lane highways.

Different types of acceleration data are employed in these applications. For studying vehicle-operating characteristics on grades, design curves relating (1) speed, (2) magnitude of grade, and (3) length of grade are presented for recreational vehicles and trucks (see Figs. III-26, III-27A and B, III-30, and III-31 from Ref. F-1). The results for significant upgrades apply to situations in which the vehicles are actually decelerating because of a lack of power. Nevertheless, we have chosen to include these cases under the heading of acceleration since the driver is using the engine in an attempt to increase or, at least, maintain speed. The acceleration characteristics for passenger cars are not required in this application because cars are believed to have enough power to readily negotiate grades as steep as 7 or 8 percent (see Ref. F-1, page III-96).

With regard to the length of acceleration lanes, acceleration characteristics for passenger cars are used (F-2) (see Fig. II-13 page II-17 of Ref. F-1). These characteristics are intended to represent the normal acceleration performance for a low horsepower passenger car. In practice this "normal" acceleration characteristic is related more to driver preferences than to vehicle capability. Even so, in certain situations, for example, at tight interchanges with grade separation, heavy trucks may not be able to accelerate to within 5 mph (8 km/h) of typical running speeds in a distance determined by normal passenger car acceleration levels.

In the case of accelerating across an intersection, the design policy provides information on three design vehicles (Fig. IX-15, page IX-48), a passenger car, a straight truck, and a tractor-semitrailer vehicle. The information is given in the form of curves of accelerating time versus distance traveled for each type of vehicle. Unpublished data have been used in Ref. F-1 to determine the assumed relationships for straight trucks and tractor-semitrailers.

Finally with regard to passing maneuvers, average acceleration levels are given for various speed ranges. (See Ref. F-1, Table III-4, page III-15.) These values of acceleration are based on observations of traffic and, apparently, they represent the performance of car-driver systems.

Characteristics of the Current Vehicle Fleet Applicable to the Design Policy

The characteristics of the vehicle fleet are continually changing. An emphasis on fuel economy has brought about lighter and less powerful passenger cars. In heavy trucks, the trend has been towards heavier vehicles with more powerful engines. Vehicles now have more efficient aerodynamic shapes, tires with less rolling resistance, and more efficient engines and drivelines than they had 5 to 10 years ago. For trucks these changes in retardation are approximately equivalent to a 1 percent change in grade (F-3). That is, for heavy trucks, 3 percent downgrades are now effectively 4 percent, and 3 percent upgrades are now effectively 2 percent--clearly an acceleration advantage and, as it has turned out, a braking problem. The purpose of this section is to compare the acceleration characteristics of the current vehicle fleet with those used in the design policy.

Heavy Truck Acceleration; Climbing Lane Criteria. In this study emphasis has been placed on the acceleration characteristics of the heavy truck. At first an attempt was made to acquire relevant data from various manufacturers. Although cooperation was obtained, the information received did not represent a comprehensive assessment of the nation's truck fleet. In order to develop a uniform method for assessing truck performance, a review was made of the methods available for predicting the acceleration performance of heavy trucks. On the basis of the findings of the review, we have concluded that (1) suitable models of the acceleration performance of trucks are available, and (2) a large body of pertinent information on heavy trucks is contained in the 1977 Truck Inventory and Use (TIU) Survey conducted by the Department of Commerce (F-4).

The draft design policy presents a set of curves showing a decreasing trend in weight-to-horsepower ratio for the vehicle fleet from 1949 to 1973. Data from the TIU survey have been superimposed on the information presented in the design policy (see Fig. F-4). The "1977" curve in Figure F-4 was obtained from computerized files of the TIU information gathered from over 96,000 trucks. The weight used in the 1977 data is the maximum weight carried during 1977 as reported by the vehicle operator. In this regard the weight-to-power estimates represent the average of the performance capability of the fleet in a heavily loaded condition (that is, the weight-to-power properties of vehicles when they are operating empty or partially loaded are not included here). Nevertheless, the 1977 curve falls well below the other curves, thereby continuing the trend towards lower weight-to-power ratio (that is, higher power-to-weight ratio and greater acceleration capability).

With respect to the "300-lb/hp" vehicle used in the design policy, the results from the TIU survey indicate that the average loaded truck in the 60,000- to 80,000-pound weight class has an engine with an average horsepower of 282 with an estimated standard deviation of 51 hp. These figures correspond to an average weight-to-horsepower ratio of 248 lb/hp, with a 303-lb/hp vehicle being one standard deviation less powerful than average. Hence, according to the TIU data, a 300-lb/hp design vehicle that is almost one standard deviation below average might be characterized as "substantially below average" rather than as "typical" as qualitatively referred to in the draft design policy. Nonetheless, one could argue that a 300-lb/hp vehicle represents a reasonable vehicle to use in designing highways and establishing the need for climbing lanes.

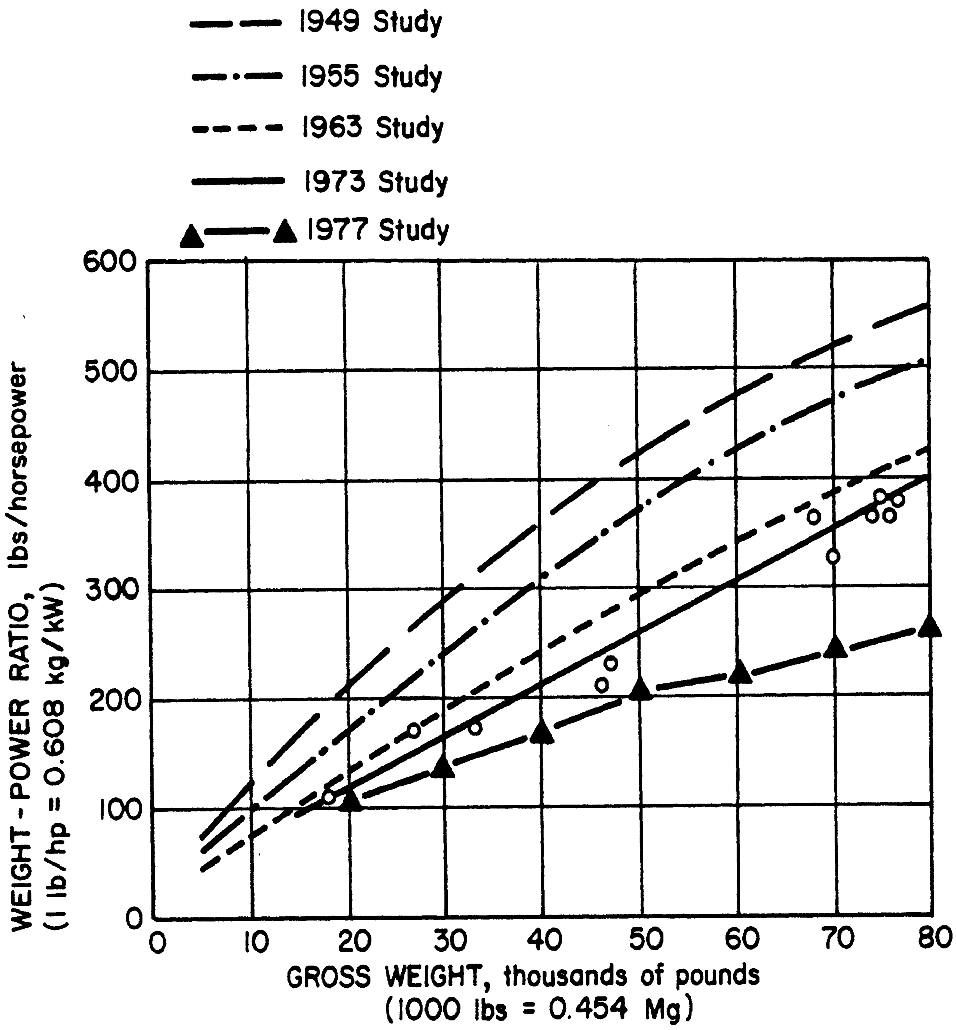


Figure F-4. Trend in weight-power ratios from 1949 to 1977 (F-5).

The relative importance of the power-to-weight ratio (the inverse of the weight-to-horsepower ratio) may be understood by comparing the upper bound on propulsive thrust to the influences of driveline efficiency, rolling resistance, and aerodynamic drag on net thrust; for example, see Figure F-5 representing a typical heavy truck similar to the one analyzed in Ref. F-5 and subsequently used to develop Figure III-31, page 107, of the draft design policy. In this case, we have employed retardation (drag) factors that are derived from (1) our literature review and (2) contacts with manufacturers. Table F-1 summarizes the equations, relationships, and coefficient values employed in this analysis.

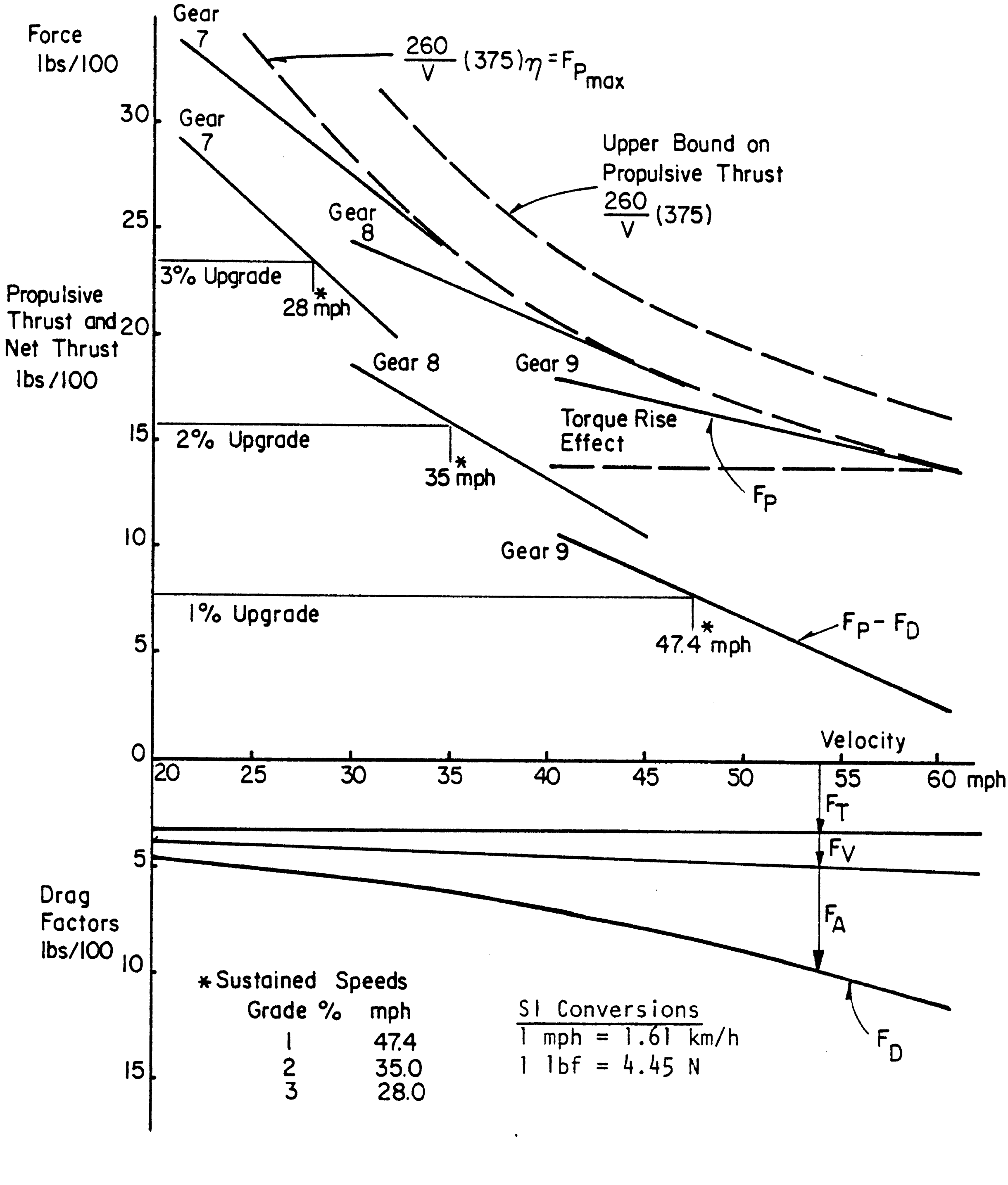


Figure F-5. Force analysis of a design heavy truck operating in 7th, 8th, or 9th gear.

Table F-1. Analysis factors for a 300-lb/hp truck.

V (forward velocity) = independent variable, mph.

GVW (gross weight) = 78000 lb.

NHP (net engine horsepower) = 260 hp at 0 to 500 ft.

C_e (elevation correction factor) = 1-4 (10^{-5}) E, in which E = elevation, e.g., C_e = 0.6 at 10,000 ft).

η (driveline efficiency) = 0.86 for tandem drive axles (note: for a single drive axle η = 0.9).

F_r (rolling resistance, radial tires) = (GVW/1000) (4.1 + 0.041V) C_r, in which C_r is a factor defining the quality of the road surface

Typical values of C_r are 1.0 for a smooth concrete road, 1.2 for worn concrete road or a cold black top road, and 1.5 for a hot black top road. Note: (1) For bias tires, F_r = (GVW/1000) (6.6 + 0.046V) C_r. (2) The source of the velocity dependent term in the rolling resistance equations may not be dependent on tire properties but rather on friction in rotating parts.

F_a (aerodynamic drag) = C_a(A)(0.0024) V^2 C_p, in which C_a is a drag factor depending on vehicle shape. (Typical values of C_a are 0.9 for highway tractors without aerodynamic aids and 0.7 for tractors with aerodynamic shields.)

A (frontal area) = 102 ft^2

C_p (elevation factor) = 1.0 at sea level, 0.86 at 5,000 ft, and 0.74 at 10,000 ft.

GR (overall gear ratio, including a rear axle ratio of 4.11)

Gear Number	Ratio
1	48.62
2	32.47
3	23.80
4	17.76
5	13.15
6	10.15
7	7.74
8	5.54
9	4.11

Tire factor (rpm/mph) = 8.4 for a 10 x 20 truck tire (504 rev./mile)

Engine Power and Torque Characteristics (see Fig. F-6 from Ref. F-5). These data are characterized by a torque at 1,400 rpm that is approximately 1.3 times the torque at 2,100 rpm, the rated speed at which maximum power (260 hp) is delivered. In other words, the torque increases by approximately 30 percent going from rated speed down to the speed at which shifting is expected to occur.

W_e (total equivalent) = $W + g/R_t^2\ (I_e\ G_r^2 + I_t)$, in which

I_e = 2.58 ft lb sec^2 for a typical engine

I_t = 170 ft lb sec^2 for 18 10x20 tires

g = 32.2 ft/sec^2

R_t = 1.667 ft for a 10x20 tire

W = gross vehicle weight

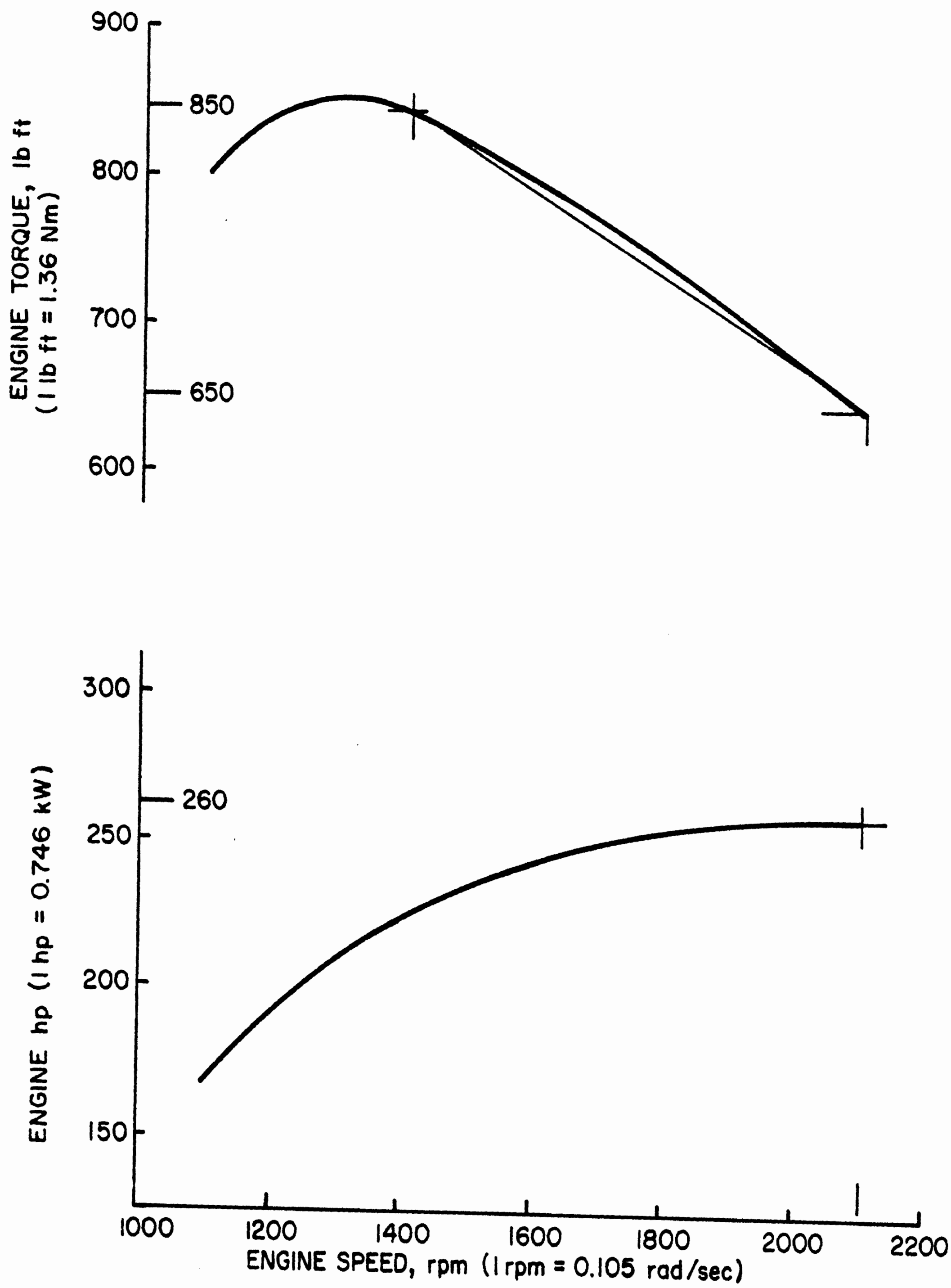


Figure F-6. Engine torque and hp characteristics used in the truck simulation (F-5).

Although a driveline efficiency is used here in place of chassis friction, the thrust and drag forces are nearly equal to those employed in Ref. F-5. However, in computing acceleration (see Fig. F-7) an equivalent weight is employed to account for the inertia of rotating components, and a tire factor of 8.4 rpm/mph, corresponding to a typical 10 x 20 tire, was selected. The analysis in Ref. F-5 used 8.55 rpm/mph, which corresponds to a smaller 9 x 20 tire. These seemingly small changes in tire factor (1.75 percent) and weight (3.5 percent in 9th gear) cause a significant change in the critical length of grade on slight upgrades (see Fig. F-8). On a 2 percent upgrade, for example, the critical length of grade is approximately 2400 ft for a vehicle with an 8.55 rpm/mph tire and a weight of 78,000 lb compared to approximately 2900 ft for a tire with 8.4 rpm/mph and an equivalent weight of 80,710 lb.

On steeper grades (for example, 4 and 6 percent), the level road acceleration capability of the vehicle is a smaller fraction of the existing acceleration (deceleration). Hence, variations in vehicle parameters, such as the tire factor and equivalent weight, have less influence on acceleration performance on steep grades than they do on moderate grades.

The climbing lane criteria, given in Ref. F-5 and subsequently incorporated in Ref. F-1, employ 300 lb/hp which is representative of a relatively low-powered loaded heavy vehicle by 1977 standards (something like 84 percent

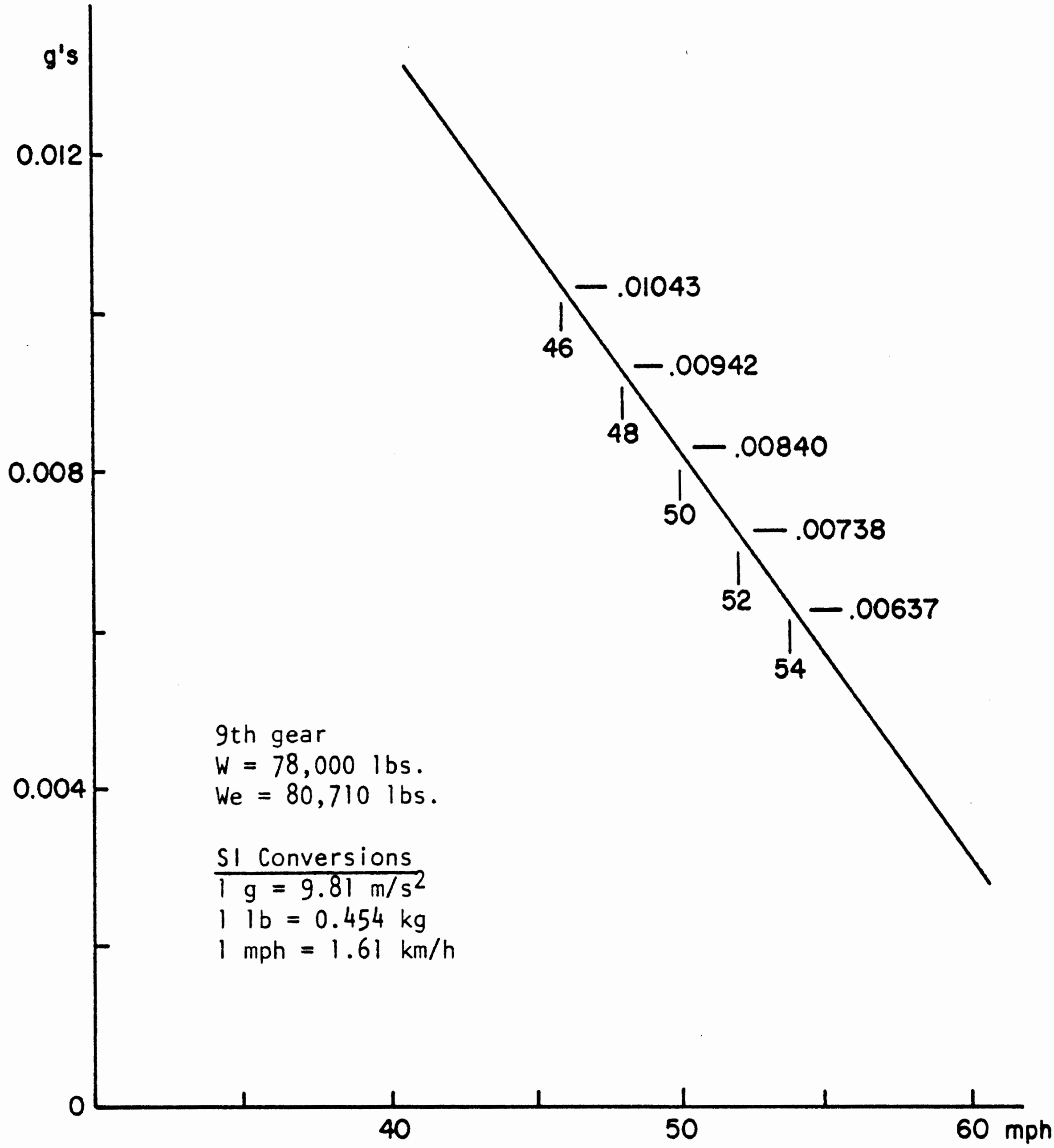


Figure F-7. Level-road acceleration versus velocity for a design heavy truck in 9th gear.

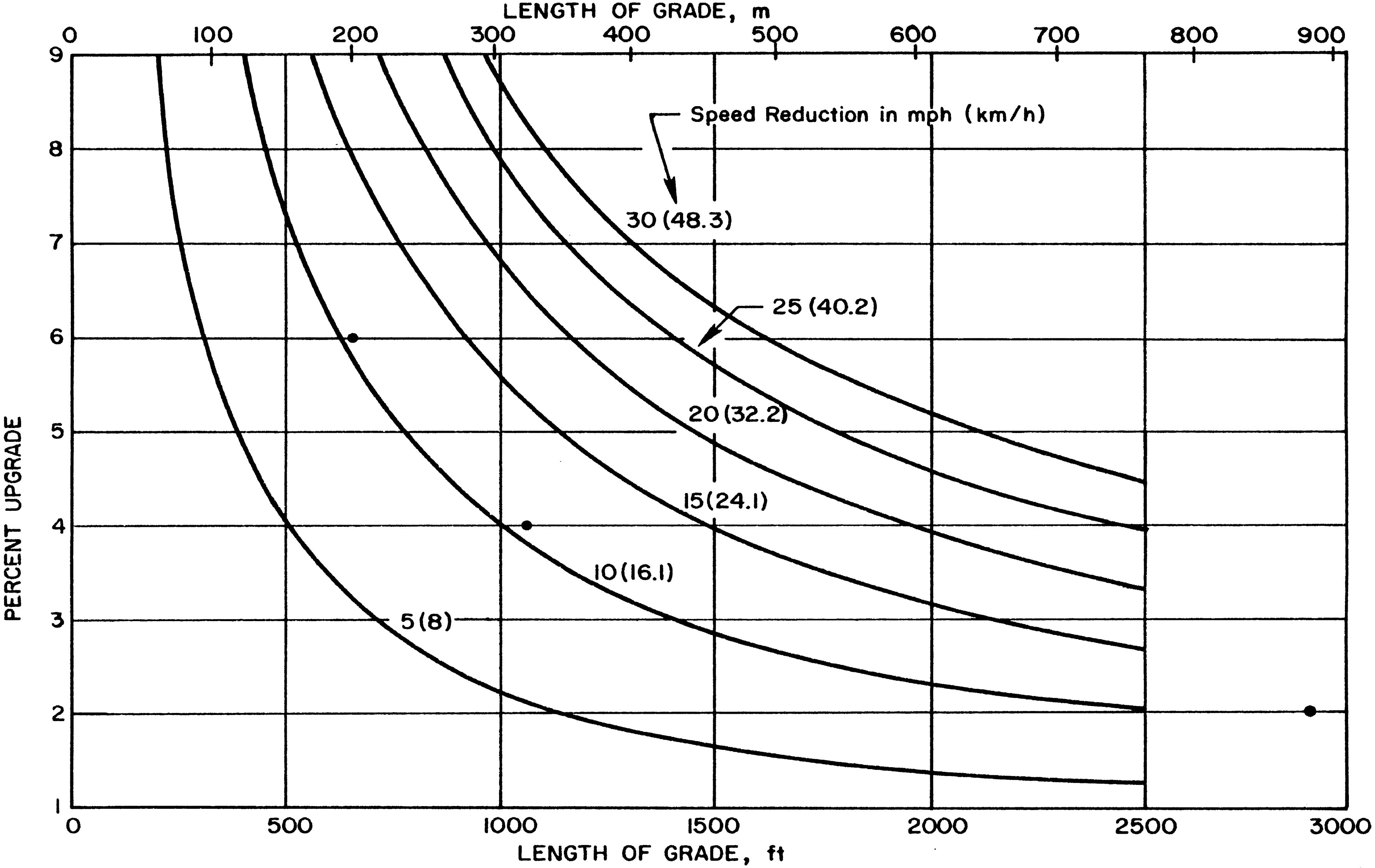


Figure F-8. Critical lengths of grade for design, assumed heavy truck of 300 lb/hp (182.5 kg/kW), entering speed = 55 mph (88.6 km/h).

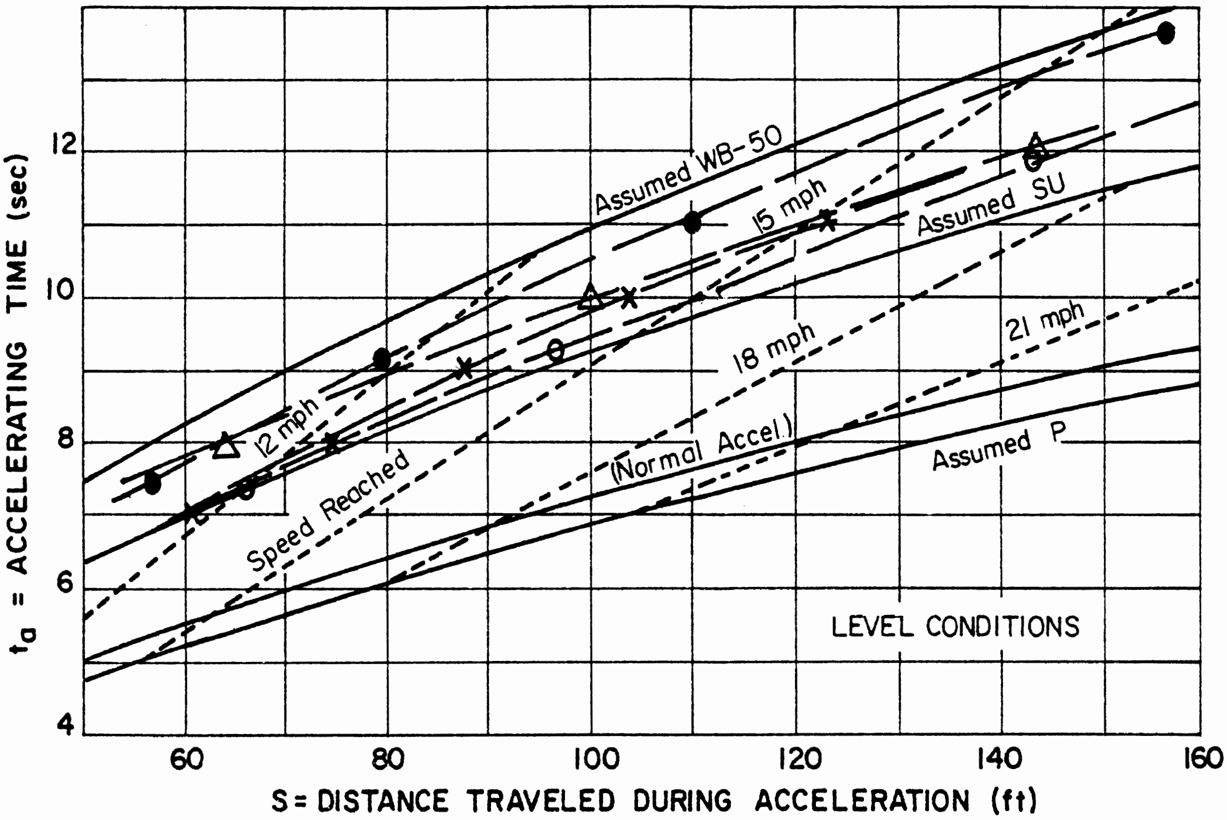
of the vehicles weighing between 60,000 lb and 80,000 lb had greater power to weight ratios). Even though the calculation procedure given in Ref. F-5 does not include the effective mass of the vehicle, the results appear to be representative of heavy-vehicle performance on steep upgrades in the speed range from 55 to approximately 30 mph. At low speeds and on mild upgrades the influence of effective mass should be included in the calculations.

During 1983, the Department of Commerce will conduct a Truck Inventory and Use Survey pertaining to vehicles operated in 1982. These 1982 data could be analyzed, using the procedures employed by manufacturers and the highway research community, to obtain an updated set of curves to be used in evaluating the need for climbing lanes.

Heavy Truck Acceleration; Accelerating Time Versus Distance Traveled During Acceleration. Data on acceleration from a stop are used in determining sight distance at intersections (see Fig. F-9, which includes Figure IX-15, page IX-48 of Ref. F-1). As indicated in the figure, two heavy vehicles, referred to as WB-50 and SU, and a passenger car, P, have been assumed for design purposes.

The WB-50 design-vehicle is intended to represent a large tractor-semitrailer combination. Assuming that a heavy vehicle similar to the one used in the climbing lane application is a suitable design vehicle of the WB-50 class, the acceleration performance of a 300-lb/hp heavy truck can

SI Conversions
1 mph = 1.61 km/h
1 ft = 0.305 m
1 lb/hp = 0.608 kg/kW
1 ft/sec^2 = 0.305 m/s^2



Superimposed Curves

—X——————X———X 273 lb/hp test data 1969 Study (F-6)

—O——————O———O 300 lb/hp Design Vehicle starting in 2nd gear.

—●——————●———● Same as above except starting in 1st gear

—△——————△———△ 2 ft/sec^2 average acceleration (Approximation to 300 lb/hp result from 1969 Tests (F-6)

Figure F-9. Information on acceleration from a stop.

be used here to make a comparison with the data given Figure IX-15 of Ref. F-1 (see Fig. F-9).

Note that in Figure F-9, 4 curves are superimposed on the graphs presented in Ref. F-1. Two of these 4 curves represent the calculated performance of the 300-lb/hp vehicle described in Table F-1. In one of these cases the vehicle is started in first gear, and in the other case the vehicle is started in second gear. As shown, significantly better performance is obtained by starting in second gear (a fact that is well known to truck drivers).

Curves based on tests of heavy trucks (F-6) are also added to Figure F-9. These curves correspond to (1) a 273-lb/hp truck and (2) an average acceleration level of 2 ft/sec^2 approximating a typical truck with 300 lb/hp, operating in 1969 (F-6). These curves agree with the calculated results for the design vehicle when started in second gear (the conventional gear selection for starting on the level).

The assumed WB-50 curve given in the design policy illustrates poorer performance than any of the 4 curves superimposed in Figure F-9. In this sense, the WB-50 curve represents a conservative design policy, especially as long as the trend is towards vehicles with higher horsepower-to-weight ratios, less aerodynamic drag, and less rolling resistance.

The assumed SU curve represents a straight truck with a 20-foot wheelbase. A great variety of vehicles fit within this description. For example, a truck with a 12,000-lb front axle and a 34,000-lb tandem rear axle-set is a possible candidate for a design vehicle of this class. Example predictions of the acceleration performance of this type of vehicle fall near the SU curve given in Ref. F-1. However, in this case the assumed SU curve does not appear to be as conservative as the assumed WB-50 curve. Given current vehicle characteristics, certain fully loaded straight trucks that satisfy the bridge formula may require more time to accelerate across an intersection than that shown in Figure IX-15 of Ref. F-1. A possible method for resolving this situation would be to provide a more complete definition of the SU design vehicle.

Passenger Car Acceleration; Accelerating Time Versus Distance Traveled During Acceleration. In contrast to the situation with heavy trucks (as discussed in the previous section), passenger cars seldom accelerate at maximum performance; therefore, knowledge of the maximum performance capability of the vehicle is not as useful as it is for trucks. That is, an experienced driver uses the maximum performance of a truck while a prudent driver does not challenge the capabilities of the passenger car engine (unless he/she wishes to spin wheels) in accelerating to cross an intersection.

Possibly because of the difficulties in determining "normal" acceleration, the results given in Figure II-13 of Ref. F-1 differ from those given in Table 6.47 of Ref. F-7. In studies of ramps and speed change lanes (F-8), investigators have found the tables given in Ref. F-7 to be more representative of vehicle performance on ramps than the information given in Figure II-13. On the basis of the calculations of acceleration derived from the curve representing the "assumed P" vehicle in Figure IX-15, the average acceleration of the design passenger car is approximately 2.86 mph/sec, compared to a normal acceleration of 3.3 mph/sec given in the Transportation and Traffic Engineering Handbook Ref. F-7 for speed changes from 0 to 30 mph. Or, as illustrated in Figure F-10, the calculated accelerating-time-versus-distance curve (representing a normal acceleration of 3.3 mph/sec) indicates shorter acceleration times than those required by the "assumed P" vehicle. The AASHTO design policy is conservative in that acceleration levels corresponding to those normally chosen by passenger car drivers produce acceleration times that are considerably shorter than those given by AASHTO.

Hearne and Clark (F-9) have recently studied passenger car data reported by Consumer Reports, Motor Trend, and Car and Driver magazines for two acceleration maneuvers, specifically: (1) the time to accelerate from 45 to 65 mph, and (2) the time to accelerate from 0 to 60 mph. That study examined the trends in these measures of new vehicle performance over the period from 1971 to 1979. The resulting acceleration characteristics for the 1970's are compared with the acceleration performance criteria used in

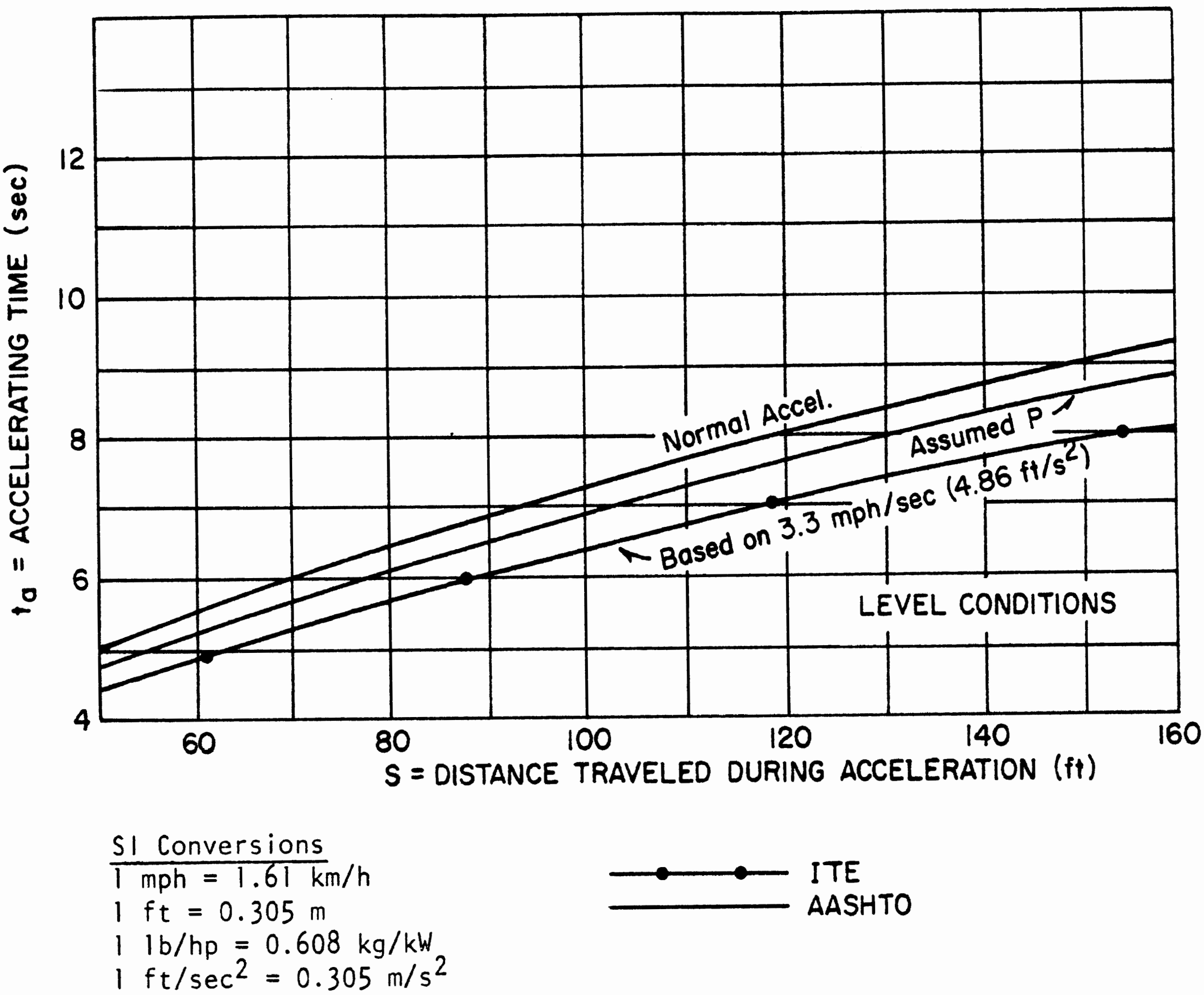


Figure F-10. Comparison of AASHTO and ITE information on acceleration performance.

the "AASHO Blue Book" (F-10). The following findings from Ref. F-9 indicate that even though passenger-vehicle acceleration performance has been decreasing since approximately 1958, the acceleration performance of late model cars exceeds the criteria employed in the AASHO Blue Book. (The AASHO criteria are based on tests performed in 1937.) Between 1970 and 1980, the typical standard-sized car changed from approximately 4,000-lb with a 350-in.^{3} engine to approximately 3,300 lb with less than 250-in.^{3} of engine displacement Ref. (F-9). The implications of these changes are illustrated in the times required to accelerate from 45 to 65 mph and 0 to 60 mph as shown in Tables F-2 and F-3 for model years 1971, 1973, 1975, 1977, and 1979. The times given in Tables F-2 and F-3 represent the car population for each model year since they are obtained by weighting the performance of each vehicle model in accordance with its annual sales volume. The average weighted acceleration time from 0 to 60 mph increased from 12.7 sec in 1971 to 15.5 sec in 1979 (see Table F-2), indicating a decline in automotive performance during the 1970's.

Clearly, overall acceleration performance has decreased during the decade of the 1970's.

Nevertheless, the performance of 1979 and 1981 vehicles exceeds the AASHO criteria based on studies performed in 1937 (see Figs. F-11 and F-12). Assuming that (1) normal acceleration performance is primarily determined by driver

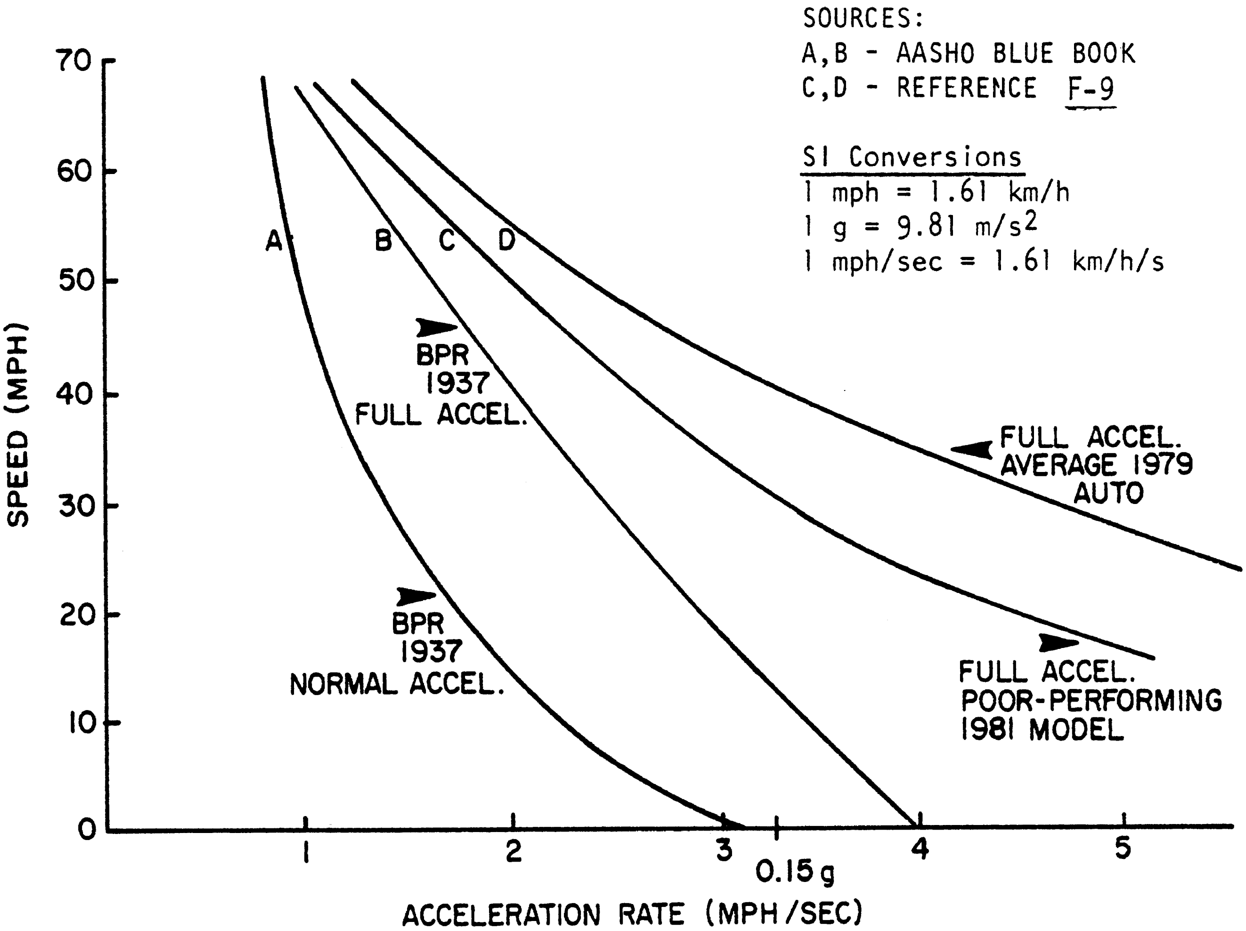


Figure F-11. Normal and full acceleration rates for a range of speeds (F-9).

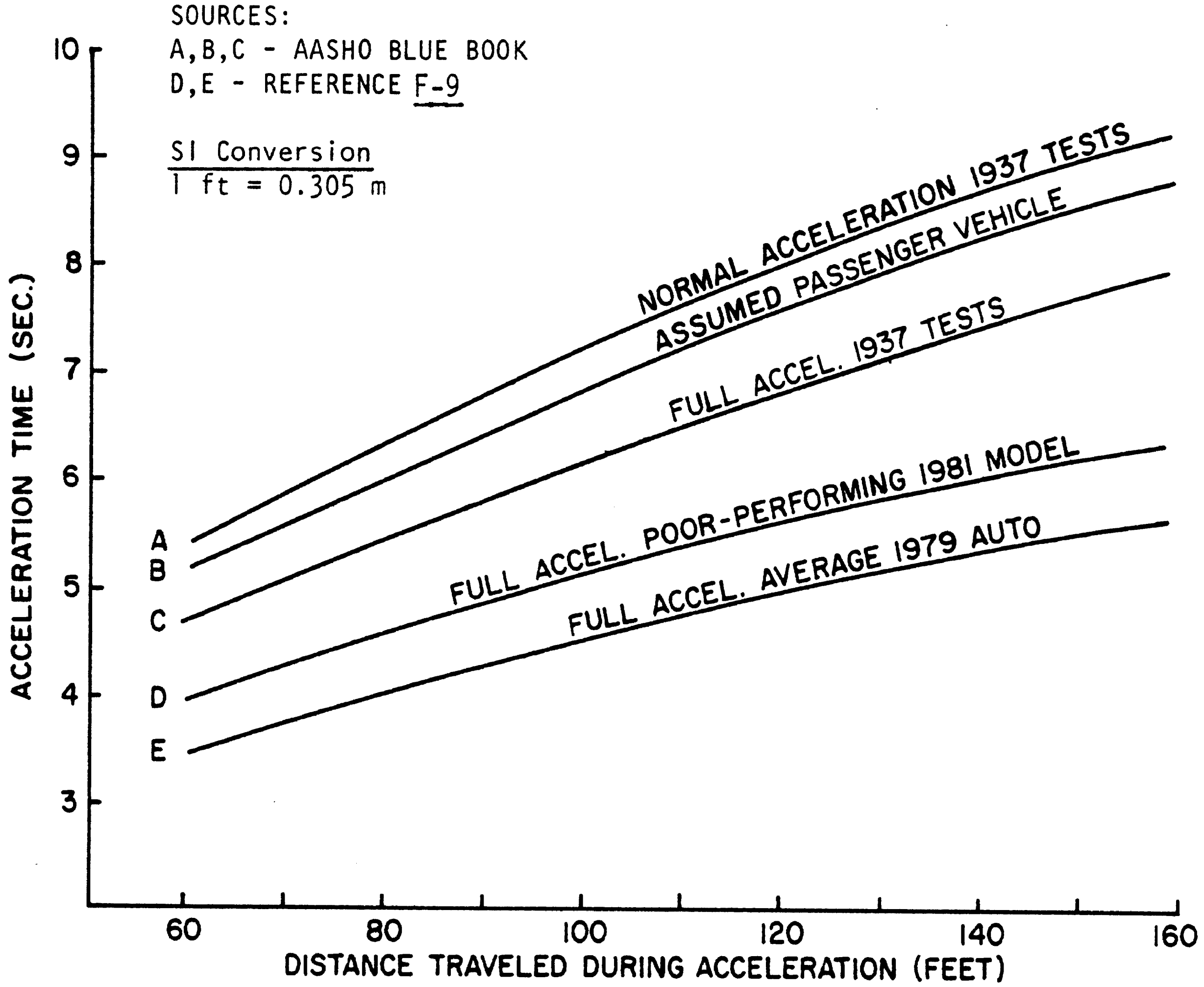


Figure F-12. Relationship between acceleration time and distance traveled during normal and full acceleration (F-9).

Table F-2. Time required for acceleration from 0 to 60 mph for selected model years (F-9).

Year	Time (sec)
1971	12.7
1973	14.1
1975	14.8
1977	15.5
1979	15.5

Table F-3. Time required for acceleration from 45 to 65 mph (F-9).

Year	Time (sec)	Average Accel. Rate (fps)
1971	8.4	3.49
1973	9.4	3.12
1975	9.4	3.12
1977	10.1	2.90
1979	9.9	2.96

"taste" rather than by vehicle characteristics, and (2) drivers continue to prefer the same normal acceleration performance as they did in 1937, then the difference between normal acceleration (1937) and the full acceleration capability of a vehicle may be interpreted as a margin of safety for situations in which the driver finds a need for accelerations that are greater than he/she would prefer to use under normal circumstances. In Ref. F-9 it is found that the average weighted performance of 1979 models and a poorly performing 1981 model exceed (by a wide margin) the normal acceleration performance determined in 1937 (see Figs. F-11 and F-12).

Passenger Car Acceleration; Length of Acceleration Lanes. As stated in Ref. F-2, "The length of acceleration ramp required for an entrance is governed by the difference between the running speed of the last ramp curve or other constraint and that of the freeway. The policy is that the length provided should be sufficient for the motorist to reach a speed five miles per hour less than the average running speed of the freeway by the time the merge into the through lane is completed."

The acceleration characteristics of passenger cars are used to determine the ramp length required. As illustrated in Table F-4 (Table X-5 from Ref. F-1), acceleration performance from low to high initial speeds, V_2', and from low to high entrance speeds, V_2, is considered in this application. In other words, knowledge of the full range of acceleration characteristics is needed.

The performance information presented by AASHTO has been summarized in graphs of speed reached versus distance traveled for initial speeds ranging from 0 to 50 mph in increments of 5 mph (see Fig. F-13). Although Figure F-13 is a convenient form in which to display results, the curves provided by AASHTO are based on relatively low levels of acceleration such as those given by the "BPR, 1937, Normal Accel." curve included in Figure F-11. More recent information on normal acceleration is given in Table F-5 as presented in Ref. F-7.

For comparison purposes, the AASHTO data (F-1) and the ITE information (F-7) are presented on the same figure with full acceleration curves for (1) a 40-W/kg car representing a poorly performing vehicle for any year from 1967 to 1995 (F-12), (2) an average 1979 auto (F-9), and (3) a poorly performing 1981 car (F-9) (see Fig. F-14). Additionally Figure F-14 contains a curve representing a so-called "design" car. Examination of Figure F-14 shows that the ITE information for normal acceleration, based on a 1971 study, falls between the full acceleration curves for the average 1979 auto and the "poor-performing" 1981 model for accelerations less than 0.15g. Compared to the reported performance capabilities of current vehicles, the normal acceleration curve based on ITE information (F-7) appears to be unreasonably high near the limit of vehicle performance.

Table F-4. Minimum acceleration lengths for entrance terminals with flat grades of 2 percent or less.

Highway		L = Acceleration Length (ft)								
		For Entrance Curve Design Speed (mph)								
Design Speed (mph)	Speed Reached (V_2) (mph)	Stop Condition	15	20	25	30	35	40	45	50
		And Initial Speed (V_2') (mph)								
		0	14	18	22	26	30	36	40	44
30	23	190	--	--	--	--	--	--	--	--
40	31	380	320	250	220	140	--	--	--	--
50	39	760	700	630	580	500	380	160	--	--
60	47	1,170	1,120	1,070	1,000	910	800	590	400	170
70	53	1,590	1,540	1,500	1,410	1,330	1,230	1,010	830	580

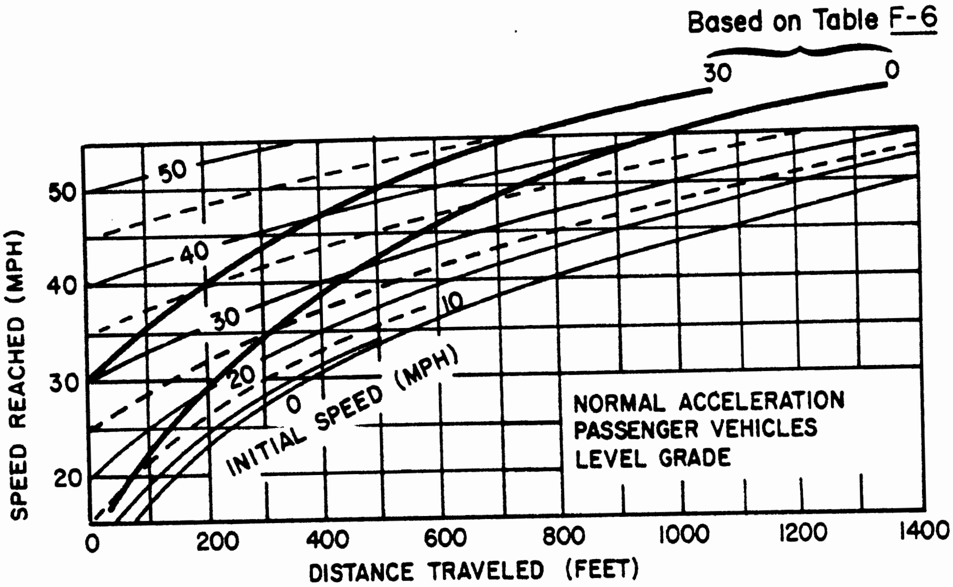


Figure F-13. Normal acceleration from Ref. F-1 plus superimposed "design"curves at initial velocities of 0 and 30 mph.

Table F-5. Normal acceleration of passenger cars.

Speed Change, mph	Acceleration mph/sec
0-15	3.3
0-30	3.3
30-40	3.3
40-50	2.6
50-60	2.0
60-70	1.3

Source: NCHRP Project 2-5A, 1971 (F-11)

Possibly, the decline in performance capability during the period from 1960 to 1980 accounts for the situation in which maximum performance in 1979 is close to the normal acceleration performance determined in a study reported in 1971.

Transportation Research Record 772 (F-12), published in 1980, contains projections of the make up of the passenger vehicle fleet in 1981, 1985, and 1995. These projections are reasoned extrapolations from vehicle data for the years 1967 through 1978. Based on data for 1967 and 1978 and projections for 1981, 1985, and 1995 (see Ref. (F-12), Fig. F-2), less than 10 percent of the passenger vehicles sold will have, or have had, power capabilities less than 40 W/kg (approximately 0.025 hp/lb). An assessment of the acceleration performance of a 40-W/kg car is given in Ref. F-12, and the results have been used to construct the 40-W/kg curve presented in Figure F-14. This curve falls between the data reported in Ref. F-9 representing the average 1979 and poor 1980 vehicles. This agreement lends credence to the proposition that a vehicle similar to the poor 1981 vehicle or the 40-W/kg vehicle approximately represents the acceleration capability of the low performance vehicles of the future.

The question remains as to what curves should be used for designing acceleration lanes. A possible choice is to select a speed-versus-acceleration-rate-characteristic that is deemed to be a rational mixture of driver preferences and

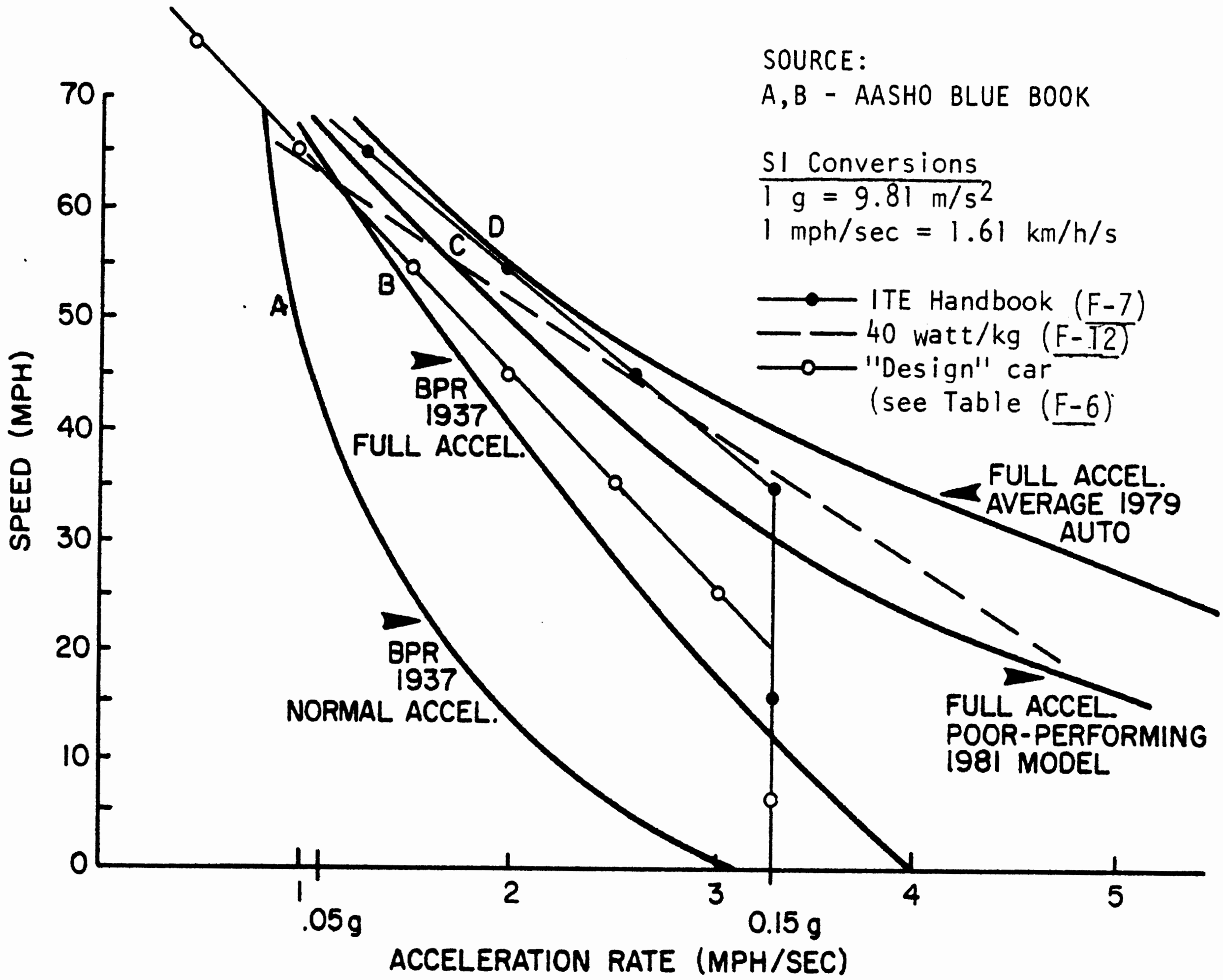


Figure F-14. Comparisons of velocity versus acceleration characteristics of passenger cars.

vehicle capabilities. For example, the curve labeled "Design" car in Figure F-14 represents one such choice. In this case an upper bound on acceleration is set at 0.15 g per the ITE information (F-7) at low speed. At speeds greater than 20 mph, the acceleration rate decreases from roughly 70 percent of a poor vehicle's capability at 20 mph to an amount that corresponds to almost all of that vehicle's capability at 70 mph. Specifically, the average acceleration capabilities for various speed ranges for this "design" car are given in Table F-6. Based on the accelerations provided in Table F-6, the two curves superimposed in Figure F-13 are obtained for the distance required to accelerate to various speeds from initial speeds of 0 and 30 mph.

Examination of the curves presented in Figure F-13 indicates that drastic changes in the lengths of acceleration lanes are implied by the use of the design car concept. For example, the original AASHTO curve indicates that a distance of 1,000 ft is required to accelerate from 30 to 50 mph, while according to the curves for the design car a distance of 535 ft would be required to accelerate from 30 to 50 mph. Clearly, such drastic changes should be examined critically.

Possibly, the longer lengths are needed for entering vehicles to find a gap in the traffic stream. Or, heavy trucks (with nowhere near the acceleration capabilities needed to match passenger cars) are a limiting factor.

Table F-6. "Design" car accelerations.

Speed Range (mph)	Average Acceleration(g's)
0-20	.150
20-30	.137
30-40	.114
40-50	.091
50-60	.068

Nevertheless, if design policy is to be based on the acceleration of the car/driver system, the AASHTO curves should be reexamined and updated.

Passenger Cars; Passing on Two-Lane Highways. For passing on two-lane highways, the design policy (F-1) specifies the sight distances needed for one vehicle to pass another before encountering oncoming traffic. The total passing sight distance specified in Ref. F-1 is divided into 4 parts: (1) initial acceleration distance, (2) distance traveled in the left lane, (3) clearance safety margin, and (4) distance traveled by the opposing vehicle. Vehicle acceleration performance is involved only in the first of these four items.

The design policy provides the following information (see Table F-7) concerning the acceleration performance of the passing vehicle during the initial maneuver.

As observed in Ref. (F-9), the acceleration rates given in the design policy can be compared with vehicle capabilities to provide an indication of the "adequacy of the design values." Referring to Figure F-11, an average acceleration of 1.5 mph/sec can be exceeded up to a maximum velocity that depends on vehicle characteristics:

Vehicle	Max. Velocity for 1.5 mph/sec Acceleration
1937, BPR	52 mph
"Poor-Performing" 1981 model	60 mph
Average 1979 Auto	64 mph

Table F-7. Initial maneuvering characteristics (F-1)

Speed Group, mph (Passing Vehicle)	30-40	40-50	50-60	60-70
Average passing speed (mph)	34.9	43.8	52.6	62.0
Average acceleration (mph/sec)	1.40	1.43	1.47	1.50
Time (sec)	3.6	4.0	4.3	4.5
Initial maneuver distance (ft)	145	215	290	370

Examination of these data and Figure F-11 indicate that the design policy and car acceleration capabilities start to approach each other in the 60 to 70 mph speed group.

However, the fact that the design values are close to the capabilities of low-powered vehicles may not be of great significance. The contribution of the acceleration part to the total passing sight distance is small, approximately 15 percent of the total. In addition, drivers of low-powered vehicles may be expected to refrain from attempting high-speed passing-maneuvers, or at least, to fall back once they observe that they do not have adequate power.

Recreational Vehicles; Hill Climbing. The power-to-mass ratio of recreational vehicles (RV's) are approximately equal to one-half of the power to mass ratios of passenger cars (possibly, because many of RV's are composed of a car and a trailer of nearly equal weight). As a conservative estimate of the lower bound of the 1978 RV population, Glauz et al. (F-12) determined a power to mass ratio of 19.7 W/kg (0.012 hp/lb). They predicted that this lower bound on power-to-mass ratio would apply into the future through 1995. Their estimate of the acceleration performance of the lowest 10 percent performance of RV's is summarized in Figure F-15.

Although the estimated acceleration characteristic given in Figure F-15 is a rough approximation to performance capability near V = 79 ft/sec (approximately 54 mph), we have used that acceleration characteristic (Fig. F-15) to

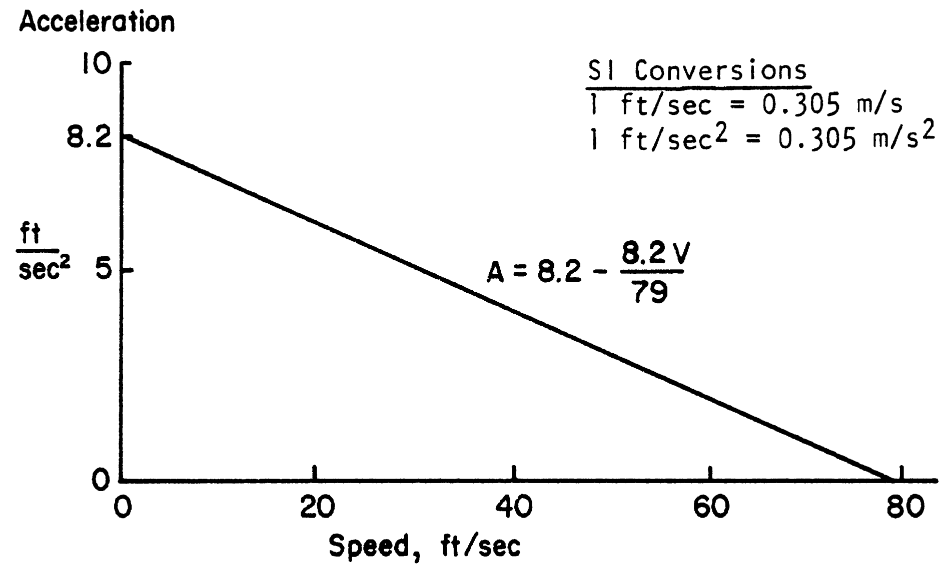


Figure F-15. Acceleration performance of RV's 1975-1995 (F-12).

computed distances for 10-mph speed reductions for upgrades entered at 55 mph. Example results for a 10-mph reduction in velocity are 1,300 ft on a 5 percent upgrade, 900 ft on a 6 percent upgrade, and 700 ft on a 7 percent upgrade. These points are superimposed on Figure F-16 which also contains (1) calculated results obtained in Ref. F-5, and (2) measured data that are used in AASHTO design policy (F-1). Examination of Figure F-16 indicates that the critical length of grade determined for a 0.012-hp/lb recreational vehicle is considerably less than that specified by AASHTO (the dashed line).

This finding is not unexpected, given the difference in hp/wt ratios involved (i.e., 0.022 versus 0.012 hp/lb). However, measured results show that drivers of recreational vehicles do not use all of the power available to them (F-5). Hence the difference between (1) measured performance for a 0.022-hp/lb vehicle, and (2) calculated performance for a 0.012-hp/lb vehicle is not as large as it would be if drivers used almost all of the available power.

No comprehensive source of information on the acceleration performance of recreational vehicles has been identified in this study. Nevertheless a large body of sustained speed data has been obtained and processed in California by Ching and Rooney (F-13). For vehicle/travel-trailer combinations, they (F-13) have observed that sustained speeds of 43 mph on 3 percent upgrades and 30 mph on 6 percent upgrades correspond to 12.5 percentile

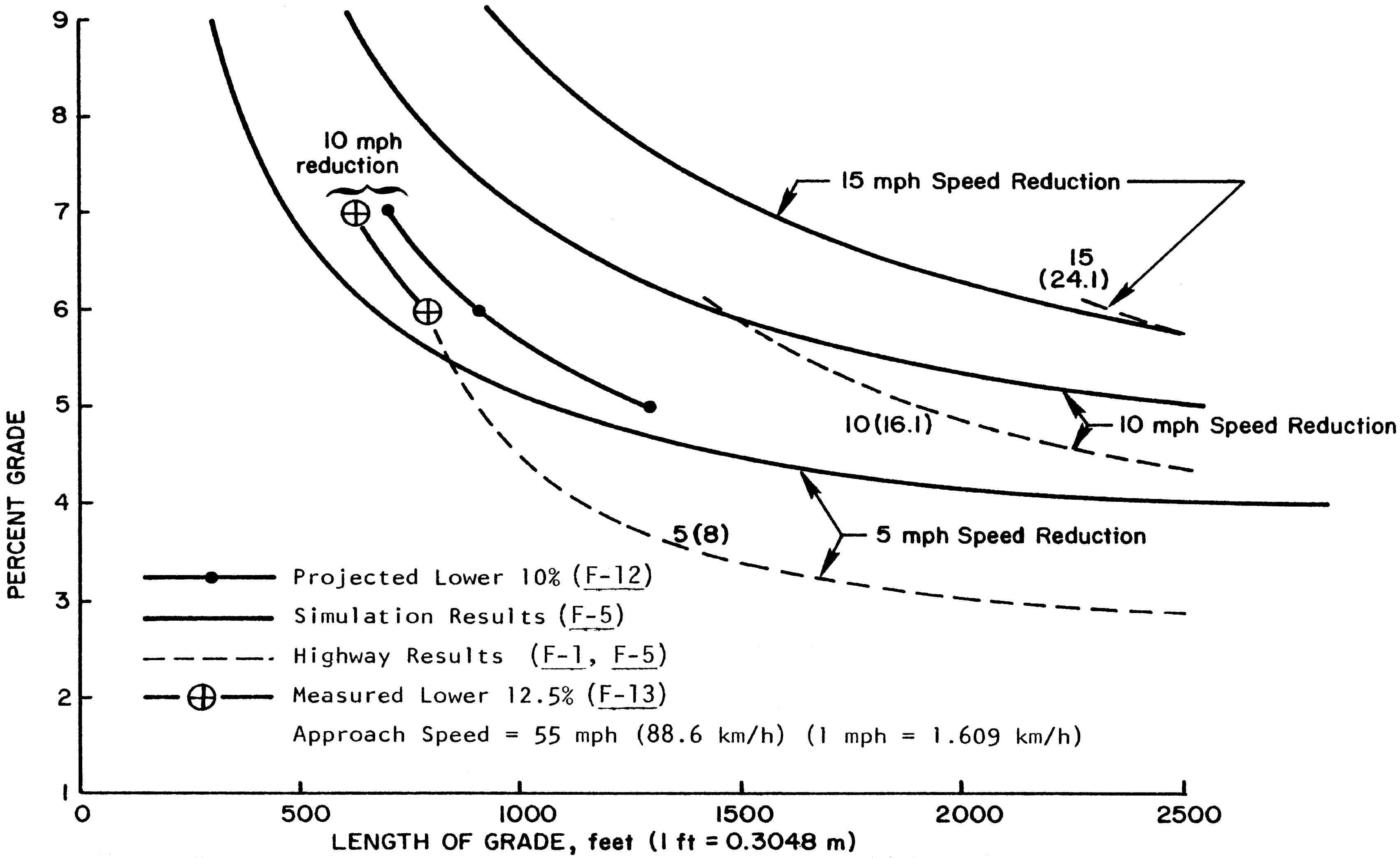


Figure F-16. Critical lengths of grade for design, assumed recreational vehicles and field test data.

vehicles. By assuming an acceleration characteristic of the following form, we have used Ching and Rooney's data to add two more points to Figure F-16:

$$\bar{A} = - RR - AV^2 + B/V$$

where:

$\bar{A}$ = the average acceleration, g's;

RR = a rolling resistance factor equal to 0.02;

A = an aerodynamic drag factor equal to 4.76 (10^{-6}) (chosen to match the data in Ref. F13);

B = a thrust factor equal to 2.53 (chosen to match the data in Ref. F-13); and

V = forward velocity, mph.

The results extrapolated from the sustained speed information given in Ref. F-13 indicate that the AASHTO values of critical length of grade are much longer than those corresponding to a low-powered recreational vehicle. For example, on a 6 percent upgrade the critical length is 1,500 ft according to the AASHTO curve, while it is 800 ft for a low-powered recreational vehicle operated by a driver that only uses approximately 0.007 hp/lb (i.e., 143 lb/hp). This result does not appear to be unreasonable if the low-powered vehicle had a capability of 0.012 hp/lb.

Even though only a few sources of data are available, the approach used in extrapolating from Ching and Rooney's results could be employed to develop design curves for critical length of grade for both average and 12.5 percentile recreational vehicles.

SUMMARY OF OBSERVATIONS

This section summarizes the observations presented in various sections of this appendix and provides additional insights to be considered in developing new design charts.

The following items are based on comparing current vehicle characteristics with the 1981 curves proposed for the AASHTO design policy (F-1).

1. The design curves, proposed in Ref. F-1 for determining the critical length of grade relating to the need for climbing lanes, are representative of the acceleration performance of low-powered heavy commercial vehicles currently in use. The design vehicle has a weight-to-power ratio of 300 lb/hp, which is higher than that applicable to approximately 84 percent of the heavy vehicle fleet (60,000- to 80,000-lb range) in 1977. Results from the 1977 TIU survey indicate that the weight-to-horsepower ratio for the average heavy truck is 248 lb/hp. If the design policy is to be based on the average heavy truck in 1977, the design curves should be recalculated using 248 lb/hp in place of 300 lb/hp.

However, another TIU survey covering 1982 has been conducted by the Bureau of Census. The results of this survey have not been published yet, but when they are, it seems reasonable to sponsor additional work to obtain an up-to-date assessment of the performance characteristics of the current vehicle fleet.

Furthermore, a fundamental question that needs to be addressed is whether average vehicle characteristics or some other level of vehicle performance should be used in the design policy. The basic issue in this case is the likelihood that traffic flow will be impeded to an unacceptable level by slowly moving heavy vehicles. It is possible that in the future an "average" vehicle might not impede traffic significantly, but because of an increased level of truck traffic the number of slowly moving trucks might still be large enough to warrant a climbing lane. The scope of the work performed in this study was not broad enough to resolve the issues connected with selecting an appropriate percentile of weight-to-horsepower ratio to use in the design policy. Possibly if information on (1) the distribution of truck traffic, (2) the performance characteristics of the truck fleet, and (3) the impedance to traffic flow caused by trucks were to be coordinated, then a rationale based on level of service could be developed for determining the percentile of weight-to-horsepower to be employed in the design policy.

2. The design curves, used to determine accelerating time in connection with sight distance at intersections, appear to be conservative with respect to both the tractor-semitrailer (WB-50) and the passenger car (P) design vehicle. That is, even low-powered cars and trucks in the current vehicle fleet are expected to be able to accelerate faster than the design curves imply. The design curve for

the straight truck (SU) is difficult to evaluate because this type of design vehicle is not well defined in the AASHTO policy. From a vehicle characteristics standpoint, the design policy could be improved if the acceleration versus velocity characteristics of the design vehicles were specified.

3. The recommended lengths of acceleration lanes are based on estimates of the normal acceleration rates of passenger cars. A discrepancy exists between the normal acceleration performance presented in the AASHTO design policy and the normal acceleration rates presented in the ITE handbook. The preliminary findings of this study appear to indicate that (1) the ITE values are higher than appropriate for the current vehicle fleet, and (2) the AASHTO values are considerably lower than needed. We suggest that this matter requires further investigation and, as suggested earlier, the presentation of acceleration versus velocity characteristics for design vehicles would aid in clarifying the basis for the speed-distance curves used in highway design. Possibly, detailed results from Ref. F-8 will aid in defining appropriate levels of normal acceleration at various operating speeds.

4. The design values of the average acceleration during the initial maneuvering phase of passing on a two-lane road appear to be reasonable, based on the characteristics of an average 1979 auto and a poorly performing 1981 model.

5. The design policy includes information on the acceleration characteristics of recreational vehicles. These characteristics are used for evaluating the need for climbing lanes in situations where RV's are likely to impede traffic. The design policy is based on a design vehicle that is representative of an average car-travel trailer combination. If deemed necessary, critical lengths of grade for low-powered RV's (12-1/2 percentile) could be calculated using data from Ching and Rooney (F-13).

From a vehicle dynamicist's point of view, the most difficult part of understanding the AASHTO design policy (as it pertains to acceleration performance) derives from a lack of acceleration versus velocity information defining design vehicles and normal acceleration performance. This matter has already been alluded to in the list of suggestions just presented. In the future we anticipate that greater reliance will be placed on computerized methods in performing design analyses. Models of vehicle acceleration performance that are not unduly complex appear to be suitable for use in highway design. Given appropriate computational capabilities, specifications of acceleration performance in terms of acceleration capability at various speeds can be used to calculate results for a wide range of situations. Changes in vehicle characteristics either due to evolution over time or to idiosyncracies of a local vehicle fleet can be readily accounted for, if suitable (generally accepted) models are available. The dependence on specialized design charts may shift to the use of fundamental information that can be used in a variety of important applications. At some time in the future, both fundamental information and particularly important results based on that fundamental information could be presented in a manner consistent with current models of acceleration performance.

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