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16. Abstract

This report documents an evaluation of driver characteristics and how they affect highway design and traffic operations standards. The study involved the identification of all standards which are a function of a driver characteristic; the development of a population profile for each characteristic; calculation of the sensitivity of each standard to realistic changes in a driver characteristic; and recommendation of changes to current specification values for driver characteristics and to standards. The study found generally inadequate consideration of driver characteristics in the development and application of current standards. The report provides detailed examination of the driver characteristics perception-reaction time, driver eye height, vision, information processing capacity, age, sex, and pedestrian walking speed. The report recommends modifications to several design and operations standards which currently do not accurately depict the actions required of the driver in a particular situation or do not adequately compensate for the needs of the driving population.

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I. INTRODUCTION

PROBLEM BACKGROUND

Highways are designed and operated to meet the mobility needs of people and their vehicles. Since the driver and the pedestrian are an integral part of highway design and operation, it is axiomatic that the capabilities of these users be considered in the formulation of design and operation standards, regulations or guidelines.

This being true, it is quite surprising that the characteristics of the users are given little formal attention in the design of the highway facilities they use. Consider the fact that in none of the current design manuals (e.g. 1,2) published by the American Association of State Highway and Transportation Officials (AASHTO) is there a specific discussion of a "design driver"* while there is for design speed, design volume and design vehicle. There are specifications, of sort, for driver characteristics of driver eye height, perception-reaction times, etc., but these are prescribed for specific standards. Likewise, in the Manual of Uniform Traffic Control Devices (3), only casual reference is made to satisfying driver characteristics with only a few specific specifications established.

The notion of a design driver has always been a difficult concept to address. There are several human characteristics, both anthropometric and physical performance, that affect driving and walking. For many of these characteristics there is a considerable variation among the public. Even for any individual there may be a variation in his/her physical performance depending upon the highway situation or the person's condition. Also, it is not clear how driver characteristics are related to effective highway design and safety. It is understandable, then, that a "design driver" has not been formulated in official design guides.

Periodically, highway design standards should be reviewed in the light of changed conditions. In regard to drivers, there is a need to obtain basic data on any changes in the population distribution and assess the effects of these changes on highway design and operation standards. One of the difficulties in deciding how far to carry highway design changes is that the proportion of drivers affected by standards is not known.

*This study deals with both drivers and pedestrians; hence whenever "driver" is used it is meant to include pedestrians as well.

STUDY OBJECTIVES AND SCOPE

Concern for the above mentioned issues has led to this study. Its specific objectives are enumerated below:

1. Identify highway design and traffic operations standards, regulations and guides that involve driver characteristics.
2. From available sources derive the distribution of the identified driver characteristics in the U.S. driver population.
3. Determine the sensitivity of the standards to changes in the driver characteristic specification.
4. From distribution data determine the proportion of the driving population excluded or inconvenienced by changes in driver characteristic specifications.
5. Indicate where present driver characteristic specifications are too conservative (established specification is too severe in relation to the population distribution) or too liberal.
6. Indicate where a more precise determination of the population distribution for a specific driver characteristic would appear to be justified.

The scope of the study deals with all highway geometric design and traffic operation standards, regulations and guidelines* as adopted by the Federal Highway Administration (FHWA), AASHTO, the Institute of Transportation Engineers (ITE) and other nationally recognized agencies. For the most part, the standards were those contained in the AASHTO manuals and the MUTCD.

In developing data on driver characteristics, available literature was the primary source supplemented by contacts with various agencies and individuals. Laboratory or field research was beyond the scope of the study.

*Unless otherwise specified, the term standard will be used to include regulations or guidelines.

ORGANIZATION OF REPORT

The remainder of the report is organized into the following chapters:

- Chapter II describes the geometric design and operations standards which have a driver characteristic as a basis and identifies the driver characteristic and current specification.
- Chapter III presents data on the driver distribution profile for the relevant driver characteristics.
- Chapter IV discusses how the various standards are affected by a change in the driver characteristic.
- Chapter V summarizes the key findings and presents recommendations.

II. IDENTIFICATION OF STANDARDS

In order to identify the relevant standards, numerous geometric design and traffic operations manuals, handbooks and guides were reviewed. A complete listing of these is provided in Table 1. For each of these an indepth review was performed to identify those standards which are based on a driver characteristic stated either explicitly or implicitly. These standards with their relevant driver characteristics are identified in this chapter.

The order of discussion is first geometric design standards and then traffic operations and traffic control device standards. Within these two groups, standards which are based on an explicit specification of a driver characteristic are presented first followed by standards that imply a driver characteristic.

GEOMETRIC DESIGN STANDARDS

Although there are several design guides related to geometric design, those prepared by AASHTO are the primary references and are the sources of the identified standards. FHWA does not publish geometric design standards but does adopt by reference those prepared by AASHTO (4).

During the course of this project, AASHTO, through its committee structure, had prepared drafts of a new design policy manual entitled A Policy on Geometric Design for Highways and Streets (5). Even though not officially approved as an AASHTO policy, it was used for this project since it represented the most recent position of design engineers and would likely be approved and adopted eventually. Also, as it turned out, our review of the drafts uncovered some errors and inconsistencies which could be taken into consideration for the final draft.

Major Design Controls

It should be noted from the outset that highways are designed on the basis of some key parameters; namely, the current or projected volume and directional distribution of traffic, vehicle mix, the level of service, and the desirable speed and the degree of access control. Once these parameters are set then certain specific features can be designed. For example, the minimum and desirable stopping sight distance is determined from the desired design speed.

The driver is not a major design control. That is to say, when the overall design type of a highway is being determined, the types and characteristics of the driver are not explicitly considered. Driver characteristics, however, are considered within the specific elements of geometric design as will be evident from the following discussion of the relevant geometric design standards.

Stopping Sight Distance

AASHTO states the following regarding stopping sight distance:

> "Sight distance is the length of roadway ahead visible to the driver. The minimum sight distance available on a roadway should be sufficiently long to enable a vehicle traveling at or near the design speed to stop before reaching a stationary object in its path. Although greater length is desirable, sight distance at every point along the highway should be at least that required for a below-average operator or vehicle to stop in this distance.
>
> Stopping sight distance is the sum of two distances: the distance traversed by the vehicle from the instant the driver sights an object necessitating a stop to the instant the brakes are applied and the distance required to stop the vehicle from the instant brake application begins. These are referred to as brake reaction distance and braking distance, respectively." (5)

In mathematical terms, stopping sight distance is formulated in the following expression:

$$SSD = 1.47\ PV + \frac{V^2}{30(f \pm g)} \qquad (1)$$

$$\left(SSD = 0.28\ PV + \frac{V^2}{255(f \pm g)}\right)^*$$

where: SSD = stopping sight distance, ft (m)

P = brake reaction time, sec.

V = initial speed, mph (km/h)

f = coefficient of friction between tires and roadways

g = grade.

*As a convention, the equation with the SI (metric system) equivalents will always be shown in parentheses below the equation using the English units whenever it is different.

TABLE 1--References Reviewed to Identify Standards Based or Dependent on Driver Characteristics

AMERICAN ASSOCIATION OF STATE HIGHWAY & TRANSPORTATION OFFICIALS (AASHTO)

A Policy on Geometric Design of Highways and Streets, Draft
Geometric Design Guide for Resurfacing, Restoration and Rehabilitation (R-R-R) of Highways and Streets, 1977
Highway Design and Operational Practices Related to Highway Safety, Second Edition, 1974
A Policy on Design of Urban Highways and Arterial Streets, 1973
Geometric Design Guide for Local Roads and Streets, 1971
A Policy on Design Standards for Stopping Sight Distance, 1971
Manual for Signing and Pavement Marking of the National System of Interstate and Defense Highways, 1970
Geometric Design Standards for Highways Other than Freeways, 1969
A Policy on Design Standards, Interstate System, June 20, 1967
A Policy on Geometric Design of Rural Highways, 1965

FEDERAL HIGHWAY ADMINISTRATION

Traffic Control Devices Handbook, An Operating Guide, Revised Edition, Draft
Work Zone Traffic Control Standards and Guidelines, (contains Part VI of the Manual on Uniform Traffic Control Devices and Part VI of the Traffic Control Devices Handbook, An Operating Guide, Revised Edition), April, 1980
Design of Urban Streets, Technology Sharing Report, FHWA-TS-80-204, January, 1980
Roadway Delineation Practices Handbook, January, 1980
Standard Highway Signs, 1979 Edition
Manual on Uniform Traffic Control Devices for Streets and Highways, 1978 edition (including Revision #1 issued 12/79)
Roadway Lighting Handbook, Implementation Package 18-15, December, 1978
Railroad-Highway Grade Crossing Handbook, Technology Sharing Report, FHWA-TS-78-214, August, 1978
User's Guide to Positive Guidance, June, 1977
Standard Alphabets for Highway Signs and Pavement Markings, 1977 Metric Edition
Traffic Control Devices Handbook, An Operating Guide, December, 1974
FHPM 6-2-1-1 "Design Standards for Highways"

INSTITUTE OF TRANSPORTATION ENGINEERS

Transportation and Traffic Engineering Handbook, edited by John Baerwald, Englewood Cliffs, NJ, Prentice-Hall, 1976
Guidelines for Urban Major Street Design, (Tentative Recommended Practice), May, 1979
Guidelines for Driveway Design and Location, (Recommended Practice), 1975
Freeway Entrance Ramp Displays, (Recommended Practice), 1975
A Program for School Crossing Protection, (Recommended Practice), 1972
Proper Location of Bus Stops, (Recommended Practice), 1967
Recommended Practices for Subdivision Streets, 1965

NATIONAL COMMITTEE ON UNIFORM TRAFFIC LAWS AND ORDINANCES

Uniform Vehicle Code and Model Traffic Ordinance, 1968 edition (including Supplement III issued 1979)

TRANSPORTATION RESEARCH BOARD

Interim Materials on Highway Capacity, Transportation Research Circular 212, 1980
Geometric Design Standards for Low-Volume Roads, Compendium 1, 1978
Highway Capacity Manual, Transportation Research Board Special Report 87, 1965

OTHER

Introduction to Transportation Engineering by Everett C. Carter and Wolfgang S. Homburger, Reston, VA: Reston Publishing Company, 1978
Fundamentals of Traffic Engineering, 9th edition, by Wolfgang S. Homburger and James H. Kell, Berkeley, CA: Institute of Transportation Studies, University of California, 1977
Traffic Engineering Theory and Practice by Louis J. Pignataro, Englewood Cliffs, NJ: Prentice-Hall, Inc., 1973
Traffic Flow Theory and Control by Donald R. Drew, New York, NY: McGraw-Hill Book Company, 1968

The AASHTO (5) standards for stopping sight distance are rounded from the computed values. In all but one instance, the rounded values exceed the computed values, thereby providing slightly more than 2.5 seconds for perception-reaction time. The minimum value for a design speed of 50 mph (80 km/h) is less than the actual computed value (375 ft (114.3 m) vs 376.4 ft (114.7 m)). However, the effect of this shortfall is that the maximum allowable perception-reaction time is reduced only to 2.48 sec.

As formulated above, there are three basic human performance characteristics considered for stopping sight distance:

- Visual Capability - ability to detect and perceive an object in the roadway
- Information Processing - ability to assess the information perceived and decide upon alternate course of action, i.e., decision making
- Brake Reaction - response time in moving foot to brake pedal.

Collectively these characteristics are referred to as brake reaction, P, or more precisely termed the perception brake reaction time. AASHTO (5) states that "perception time is the time required for motor vehicle operators to come to the realization that the brakes must be applied. It is the time lapse from the instant an object is visible to the driver to the instant he realizes that the object is in his path and that a stop must be made". The brake reaction time is "the time required to apply brakes". This was formerly labeled as the perception-intellection-emotion-volition (PIEV) time.

The current AASHTO specification for this driver characteristic is 2.5 seconds. As specified in the AASHTO's Policy on Geometric Design of Rural Highways (1) this value was determined from an assumed perception time of 1.5 seconds and a brake reaction time of 1.0 seconds. The values do not relate to any specific percentile of driver performance but, rather, were selected as being "large enough to include the time taken by nearly all drivers under most highway conditions.

Passing Sight Distance

AASHTO (5) has a design policy related to minimum passing sight distance required to safely complete normal passing maneuvers on two lane roads. The distances recommended by AASHTO are based on the summation of the following four distances:

- d_1, the distance traversed during perception and reaction time and during the initial acceleration to the point of encroachment on the left lane;
- d_2, the distance traveled while the passing vehicle occupies the left lane;
- d_3, the distance between the passing vehicle at the end of its maneuver and the opposing vehicle; and
- d_4, the distance traversed by an opposing vehicle for two-thirds of the time the passing vehicle occupies the left lane, or 2/3 of d_2.

The initial maneuver distance, d_1, has two components: a time for perception and reaction and a time during which the vehicle is brought to the point of encroachment on the left, or passing, lane. Field observations, principally those documented by Prisk (6), indicated that equation (2) below offers a relatively accurate estimation of the distance needed by the average driver to make the initial maneuver in a delayed, rather than flying, pass:

$$d_1 = 1.47t_1 \left(V - m + \frac{at_1}{2}\right) \qquad (2)$$

$$\left(d_1 = t_1 \left(\frac{V - m + \frac{at_1}{2}}{3.6}\right)\right)$$

where: d_1 = initial maneuver distance, ft (m)

t_1 = initial maneuver time, sec

a = average acceleration, mph/sec (km/h/s)

V = average speed of the passing vehicle, mph (km/h)

m = difference in speed between the passing vehicle and the passed vehicle, mph (km/h).

Note that equation (2) does not include a specific driver characteristic variable even though it is acknowledged that there is a perception and reaction process. This driver characteristic is actually overlapped with the acceleration/maneuver time prior to encroachment on the passing lane. That is to say that while the driver is initiating a passing maneuver, the driver is both perceiving/reacting and accelerating/maneuvering. In combination, the driver and vehicle characteristics form a performance characteristic--initial maneuver time. AASHTO estimates the total time for the initial maneuver to fall within the 3.6 to 4.5 second range based on the results of empirical studies.

The distance d_2 travelled by the passing vehicle in the left lane, like the d_1 distance described above, is estimated by AASHTO based on field observations. The formula used to compute the design values is as follows:

$$d_2 = 1.47 \; V \; t_2 \qquad (3)$$

$$(d_2 = 0.28Vt_2)$$

where: d_2 = distance traveled in the passing lane, ft (m)

t_2 = time that the passing vehicle occupies the left lane, sec

V = average speed of passing vehicle, mph (km/h)

This distance is based on observed driver performance which could be considered to be a function of both driver and vehicle characteristics. Driver/vehicle performance is integrally related to the vehicle capabilities and the control skills of the driver. From an analytical perspective of the driving task, it would be impractical to attempt to separate the two.

The minimum preferred clearance lengths, d_3, between the passing and opposing vehicles at the end of the passing maneuver vary between 100 feet (30.5 m) and 300 feet (91.4m) depending on vehicle speeds. The values were based on field observations of driver performance. Although not specified, an inherent driver characteristic is the driver's ability to judge the distance between the two vehicles and the relative speeds and the driver's acceptance of minimum clearance threshold. This situation is analogous to gap acceptance at intersections or for lane merging.

The distance, d_4, traversed by the opposing vehicle is computed as two-thirds of the d_2 distance. The basis for this figure is that passing sight distance should include the distance traversed by an opposing vehicle during all but the first phase of the total passing maneuver. Since d_4 is simply a function of d_2, then the discussion of the driver characteristic for that element applies here.

Decision Sight Distance

Decision sight distance is a new geometric design standard being included in the AASHTO's "A Policy on Geometric Design of Highways and Streets" (5). It has been defined as "the distance required for a driver to detect an unexpected or otherwise difficult-to-perceive information source or hazard in a roadway environment that may be visually cluttered, recognize the hazard or its threat potential select an appropriate speed and path, and initiate and complete the required safety maneuver safely and efficiently" (7). McGee et al. (8) analytically derived and empirically validated a range of decision sight distances based on premaneuver and maneuver times.

The distances are displayed in a table in the AASHTO policy (5) but can be derived from the following formula:

$$DSD = 1.47VT \qquad (4)$$

$$(DSD = 0.28VT)$$

where: DSD = decision sight distance ft (m)

V = design speed, mph (km/h)

T = total time for detection and recognition plus decision and response plus time to change lanes*, sec

Decision sight distances is meant to be provided "whenever there is a likelihood for error in either information reception, decisionmaking, or control actions" (5). Examples would be interchanges and intersection locations where unusual or unexpected maneuvers are required, changes in cross section such as toll plazas and lane drops, etc.

Decision sight distance is very much dependent upon several driver characteristics, those being the time it takes for a motorist to detect and recognize an object or situation requiring some action, the time it takes to decide what course of action to take, and finally, the physical response time to initiate the manuever.

*Since in many cases changing lanes would be the desired maneuver, e.g., for lane drops, a time for lane changing was used as the maneuver element. If, however, a different maneuver is more critical to a particular design situation, the time to complete that maneuver could be used in lieu of lane changing time.

Also, it could be argued that the maneuver time is a driver characteristic in that the time it takes to change a lane, for example, is dependent not only upon the characteristics of the vehicle but also the driver's performance. However, it seems more logical to treat this aspect as a vehicle characteristic.

The specifications for the driver characteristics are presented as a low and high value and dependent upon speed. For the combined detection plus recognition characteristic, the values are 1.5 to 3.0 seconds for design speeds of 50 mph (80 km/h) or lower. For higher speeds, the range is 2.0 to 3.0 seconds. For the combined decision and response initiation, a range of values is 4.2 to 6.5 for the lower speeds and 4.7 to 7.0 seconds for the higher speeds.

The ranges were provided with the general guideline that the lower end is the minimum acceptable for situations of moderate complexity or visual clutter and the upper is desirable for highly complex or visually cluttered locations. The values do not apply to any specific driving population; although in selecting them the value of the mean plus one standard deviation from the field studies (8) was used as a comparison value.

Intersection Sight Distance

AASHTO (5) states the following in regard to intersection sight distance:

> "The operator of a vehicle approaching an intersection at grade should have an unobstructed view of the whole intersection and a sufficient length of the intersecting highway to permit control of the vehicle to avoid collisions. When traffic at the intersection is controlled by signals or signs, the unobstructed view may be limited to the area of control. The minimum sight distance considered safe under various assumptions of physical conditions and driver behavior is directly related to vehicle speeds and to the resultant distances traversed during perception, reaction time, and braking."

The minimum intersection sight distances have been computed by AASHTO (5) for five general cases:

- I - Enabling Vehicles to Adjust Speed
- II - Enabling Vehicles to Stop
- III - Enabling Stopped Vehicles to Cross the Major Highway
- IV & V - Safe Distances for Passenger (P) Vehicles Turning Left Onto Two-Lane Highways from a Stopped Position.

The last two cases are new standards for AASHTO. Each case is discussed separately.

Case I - Enabling Vehicles to Adjust Speed

At an intersection where no approach leg is controlled by stop signs, yield signs or traffic signals, a driver of a vehicle approaching an intersection must be provided adequate sight distance both to perceive the potentially conflicting movement of a crossing vehicle and to take the necessary countermeasure. In this case, only enough sight distance is provided to enable approaching vehicles to adjust speed and avoid a collision. The AASHTO policy provides distances for various vehicle speeds. These distances are rounded off values derived from the following equation:

$$D = 1.47\ Vt \qquad (5)$$

$$(D = 0.28Vt)$$

where: D = minimum sight triangle distance, ft (m)

V = vehicle speed, mph (km/h)

t = 3 seconds

The three seconds is the sum of two seconds for perception and reaction and one second to activate braking or accelerating to regulate speed.

The equation above is technically incorrect since it applies the same vehicle speed for the full three seconds. In fact, for the last second there is, presumably, a changing speed, either higher or lower. However, given the short time of only one second, the actual distance is closely approximated by the formula.

The driver characteristic here is the perception-reaction time. One of the two approaching drivers has to perceive the other oncoming vehicle and react to it. The specification for this characteristic is set at 1.5 to 2.0 seconds, although 2.0 seconds is used in deriving the recommended values. The additional second is technically not a driver characteristic but rather a time period over which speed is changed. No statement is made as to what percentile of drivers the specification applies.

Case II - Enabling Vehicle to Stop

The AASHTO (5) policy for Case II Intersection Sight Distance requires that a driver of a vehicle moving toward an uncontrolled intersection be able to see the intersecting highway in sufficient time to stop the vehicle before reaching the intersection. "Seeing the intersecting highway" means sighting a vehicle on the adjacent approach not yet in the intersection proper.

The distances recommended are those used for the design at any other section of highway, i.e., stopping sight distance. Consequently, the driver characteristic is the perception-reaction time which is specified as 2.5 seconds.

Case III - Enabling Stopped Vehicles to Cross Major Street

Case III applies to intersections controlled by stop signs on the minor road. The current AASHTO(5) standard calls for the driver of a stopped vehicle at the intersection to be able to see enough of the major highway to safely cross before a vehicle on the major highway reaches the intersection. AASHTO states, "The length of the major roadway open to view must be greater than the product of its design speed and the time necessary for the stopped vehicle to start and cross the road". The AASHTO formulation is as follows:

$$D = 1.47\ V\ (J + t_a) \qquad (\underline{6})$$

$$\left(D = 0.28\ V(J + t_a)\right)$$

where: D = minimum or desirable sight distance along the major highway from the intersection, ft (m)

V = design speed on the major highway, mph (km/h)

J = sum of the perception time and the time required to actuate the clutch or actuate an automatic shift, sec

t_a = time required to accelerate and traverse the distance, S, to clear the major highway pavement, sec

S = the distance that the crossing vehicle must travel to clear the major highway, ft (m)

= D + W + L

D = distance from near edge of pavement to the front of a stopped vehicle, ft (m)

W = pavement width along path of crossing vehicle, ft (m)

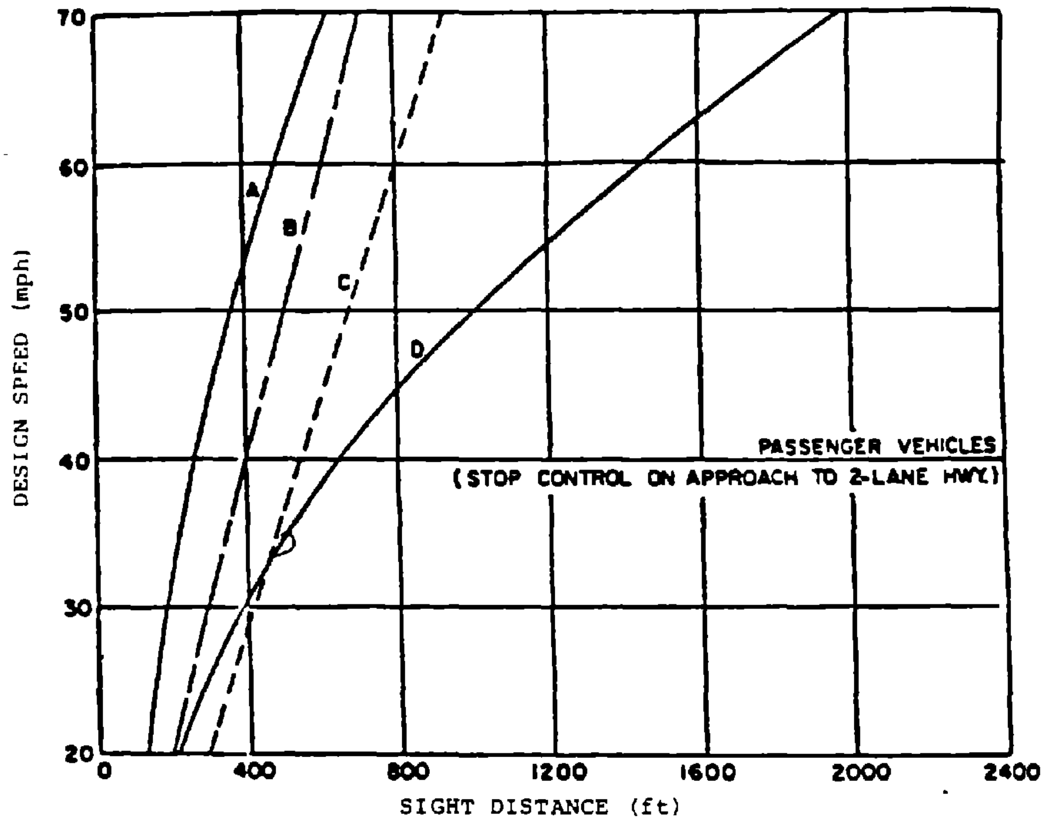
L = overall length of vehicle, ft (m)

The driver characteristic in the above formulation is the perception-reaction time, J, which "represents the time necessary for the vehicle's operator to look in both directions on the roadway, to perceive that there is sufficient time to cross the road safely, and to shift gears, if necessary, preparatory to starting" (5). The specification is 2.0 seconds which "represents the time taken by a small percentage of slower drivers". AASHTO further indicates that a lower, unspecified value might apply in urban and suburban areas where there are many stop sign controlled intersections.

Case IV and V - Safe Distance for P Vehicles Turning Left Onto Two-Lane Highways from a Stopped Position

These are two new cases which were included into the draft AASHTO design policy (5). Case IV is for a passenger (P) vehicle turning left from a stopped position across the path of a vehicle approaching from the left. Case V is for a passenger vehicle turning left and is overtaken by a vehicle approaching from the right. (The case for a vehicle turning right is not explicitly discussed but it would be somewhat less than Case V.)

The AASHTO policy does not provide formulas for determining the two sight distance requirements; rather, it includes Figure 1 which is a plot of speed versus sight distance for Cases II-V. Background information (9) related to the development of the plots for Cases IV and V indicate that the same driver characteristic applies, i.e. the perception-reaction time prior to entering the intersection. The differences between Cases III, IV and V is with the relevant manuever times, which for the purposes of this study are not considered driver characteristics.



A - Safe stopping sight distance (AASHTO). (Case II)
B - Safe sight distance for P vehicle crossing 2-lane highway from stop. (Case III)
C - Safe sight distance for P vehicle turning left into 2-lane highway across P vehicle approaching from left. (Case IV)
D - Safe sight distance for P vehicle to turn left into 2-lane highway and attain average running speed before being overtaken by vehicle approaching in same direction. (Case V)

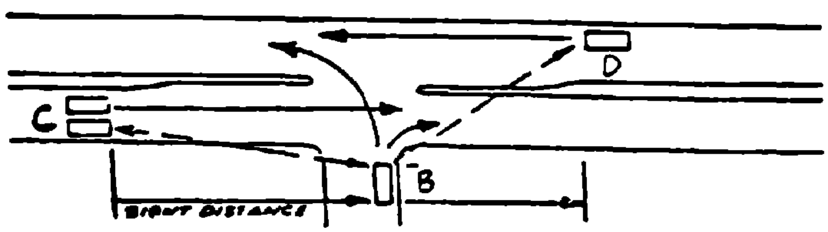


FIGURE 1 - Intersection Sight Distance at At-Grade Intersections

Source: AASHTO (5) (Note: original source: The Dynamic Design for Safety, October 1975, and expanded by data furnished by Jack E. Leisch & Associates)

Note: The SI (metric) conversions are 1 mph = 1.61 km/h and 1 ft = 0.3 m.

Driveway Sight Distance

Another specific case where sight distance deserves special attention is at driveways. However, there is a great deal of inconsistency among the basic references. Only general guidelines on location and spacing of driveways appear in the draft of A Policy on the Geometric Design of Highways and Streets (5). This AASHTO policy cites two volumes of a report entitled "Technical Guidelines for the Control of Direct Access to Arterial Highways" (10) as suggested references for the major design and control features. The minimum driveway sight distances recommended in these reports are the outdated minimum stopping sight distances and the outdated desirable stopping sight distances that appeared in A Policy on Geometric Design of Urban Highways and Arterial Streets (2). These sight distances were based on an explicit specification of a driver perception-brake reaction time of 2.5 seconds.

The 1980 Federal Highway Administration report, "Design of Urban Streets" (11), recommends the adequate driveway sight distances formulated in "NCHRP Report 93, Guidelines for Medial and Marginal Access Control on Major Roadways" (12). These required sight distances are tabled below:

Through Speed mph (km/h)	Minimum Sight Distance ft (m)
20 (32)	235 (70)
30 (48)	430 (130)
40 (64)	700 (210)
50 (80)	1,015 (305)
60 (97)	1,400 (420)

These values were developed on the assumption that vehicles have to turn into the traffic stream rather than just cross the street. The assumptions used were a perception-reaction time of 1 second and the time required to accelerate to the operating speed at a rate of 3.0 feet/second^2 (0.9m/sec^2) for the vehicle entering the street and a minimum safe following headway of 1 second between the entering vehicle and the main street vehicle. The 1 second perception-reaction time is lower that that used by AASHTO for intersection sight distance standards. Still the values are nearly identical with those determined from curve D of Figure 1, which represents Case V, the longest distance standard for an intersection.

One of the Institute of Transportation Engineers recommended practices, Guidelines for Driveway Design and Location (13), also specifies driveway sight distances. Different sight distances are recommended for:

(1) passenger cars exiting from driveways onto two-lane roads,

(2) passenger cars exiting from driveways onto four- and six-lane roads,

(3) semi-trailers exiting from driveways onto two-lane roads,

(4) semi-trailers exiting from driveways onto four- and six-lane roads,

(5) passenger cars entering one-way driveways by left turns, and

(6) semi-trailers entering one-way driveways by left turns.

A table with recommended distances is provided for each of the six cases cited above. How the values were determined is not described, but the distances were calculated to enable exiting vehicles:

(1) upon turning left or right, to accelerate to the operating speed of the street without causing approaching vehicles to reduce speed by more than 10 miles per hour, and

(2) upon turning left, to clear the rear half of the street without conflicting with vehicles approaching from the left.

The driver characteristic, perception-reaction time, must have been considered in the development of the distances as evidenced by the following passage from the ITE document:

> The sight distances.....are for urban conditions. In order to convert these to rural conditions, where driver reaction times are longer sight distances should be increased by 10 percent.

Therefore, the driveway sight distance standard is based on an unspecified driver perception-reaction time.

Railroad-Highway Grade Crossing Sight Distance

An important element in the design of railroad grade crossings is the provision of adequate sight distance especially at locations not controlled by signal or gates. This situation is analogous to highway intersection sight distance requirements in that there must be an adequate corner sight triangle as well as sight distance along the railroad for vehicles stopped at the crossing.

Sight distance requirements for railroad-highway grade crossings have appeared in several references including AASHTO's A Policy on Geometric Design for Rural Highways and Streets (5), NCRHP Report 50 (14), and the Railroad-Highway Grade Crossing Handbook (92). However, the most current standard is now included in Traffic Control Devices Handbook, Part VIII Traffic Control Systems for Railroad-Highway Grade Crossings (17).

This reference establishes sight distances for three events which can occur at a highway-railroad grade crossing. These events are:

- the motorist moving at highway speed can observe the approaching train in a sight line which will safely allow the highway vehicle to pass through the grade crossing prior to the train's arrival at the crossing;
- the motorist can observe the approaching train in a sight line which will permit the highway vehicle to be brought to a stop prior to encroachment in the crossing area; and
- the motorist has stopped and can observe the approaching train in a line of sight that will safely allow the highway vehicle to accelerate and clear the crossing prior to the arrival of the train.

The first two results in a corner sight distance triangle which, for the purpose of this analysis, will be labeled Case I, and the third event results in a sight distance along the railroad from a stopped vehicle position which will be labeled Case II. Each is discussed separately in regard to the driver characteristic and its specification.

Case I - Corner Sight Distance Triangle--The first of the two sight distance requirements applies to a visiblity triangle such that a driver can see an approaching train from a certain critical distance from the crossing. The triangle is defined by two sides: the distance along the tracks (D_T) and the distance along the highway (D_H). These distances are developed from the following two equations:

$$D_H = 1.47\ V_v t + \frac{V_v^2}{30f} + D + d_e \qquad (7)$$

$$\left(D_H = 0.28\ V_v t + \frac{V_v^2}{255f} + D + d_e\right)$$

$$D_T = \frac{V_T}{V_v}\left(1.47\ V_v t + \frac{V_v^2}{30f} + 2D + L + W\right) \qquad (8)$$

$$\left(D_T = \frac{V_T}{V_v}\left(0.28\ V_v t + \frac{V_v^2}{255f} + 2D + L + W\right)\right)$$

where: D_H = sight distance along the highway for a vehicle to cross tracks safely even though a train is observed at the same instant, or to safely stop the vehicle without encroachment of the crossing area, ft (m)

D_T = sight distance along the railroad tracks to permit the same vehicle manuevers as for d_H ft (m)

V_v = assumed velocity of the vehicle, mph (km/h)

V_T = velocity of the train, mph (km/h)

t = perception/reaction time, sec

f = coefficient of friction

D = distance from the stop line or front of the vehicle to the nearest rail, ft (m). This value is assumed to be 15 feet (4.5 m)

d_e = distance from the driver to the front of the vehicle, ft (m). This value is assumed to be 10 feet (3 m)

L = length of vehicle, ft (m). This is assumed to be 65 feet (19.5 m)

W = distance between outer rails, ft (m). For a single track this value is 5 feet (1.5 m).

The basis for equation (7) is that a driver travelling at speed V_v should be able to see the crossing (the tracks, or train on it) such that he can stop at the stop line. In other words it is the stopping sight distance plus the distance from the driver eye to the front of the vehicle, and the distance from the stop line to the nearest rail.

The basis for equation (8) is that a driver travelling at speed V_v should be able to see an approaching train travelling at speed V_T, at a distance D_T along the track so that he can travel the distance from the point established by equation (7), i.e., the stopping sight distance, to the other side of the crossing with the rear of the vehicle at the opposite side stop line by the time that the train reaches the crossing.

Both equations incorporate a driver characteristic--perception-brake reaction time. Its current specification is 2.5 seconds. Nowhere is there any discussion of the origin or validity of the 2.5 second value. Presumably it is carried over from the perception-brake reaction time specification used for stopping sight distance.

Case II - Sight Distance Along Railroad for Stopped Vehicles to Cross in Front of Train--The second sight distance requirement is to provide the driver sufficient sight distance along the railroad in order to cross in front of a train from a stopped position at the crossing. This would apply for all crossings regardless of the type of control because there is always the likelihood that vehicle will be stopped at the crossing. Unless exempted (as indicated by a supplemental sign) vehicles carrying passengers for hire, school buses or vehicles carrying flammable or hazardous materials are required to stop at all crossings.

The required sight distance can be determined from the following formula:

$$D_T = 1.47\, V_T\left(\frac{V_G}{a_1} + \frac{L + 2D + W - d_a}{V_G} + J\right) \quad (9)$$

$$\left(D_T = 0.28\, V_T\left(\frac{V_G}{a_1} + \frac{L + 2D + W - d_a}{V_G} + J\right)\right)$$

where: D_T = sight distance along the railroad required for a stopped vehicle to cross in front of a train, measured from either the left or right edge of the travelled way, whichever is nearer the approaching train, ft (m)

V_T = speed of the train, mph (km/h)

V_G = maximum speed of vehicle in first gear, assumed 8.8 fps (2.64 m/sec)

a_1 = acceleration of vehicle in first gear, assumed 1.47 ft/s^2 (0.44 m/sec^2)

L = length of vehicle, assumed 65 feet (19.5 m)

D = distance from stop line to nearest rail, assumed 15 feet (4.5 m)

W = distance between outer rails, for single track, W = 5 feet (1.5 m)

J = sum of the perception time and the time required to activate the clutch or an automatic shift (assumed 2 sec)

d_a = distance vehicle travels while accelerating to maximum speed in first gear; or:

$$d_a = \frac{V_G^2}{2a_1} \quad \text{or: } \frac{8.8^2}{(2)(1.47)} = 26.4 \text{ feet } (7.92 \text{ m})$$

The formula merely states that the distance required is equal to the speed of the train times the time it takes for a vehicle to reach maximum speed in first gear and travel across the tracks to a postiion where the rear of the vehicle is at the opposite side stop line. Also included in this process is a time for perception-reaction which is the driver characteristic. The specification for the driver characteristic is 2.0 seconds which is the same that is used for Intersection Sight Distance, Case III.

Crest Vertical Curve Length

As noted by AASHTO (5), the major control for safe operation on crest vertical curves is the provision of ample sight distance for the design speed. For most situations stopping sight distance is required but for some complex decision areas, decision sight distance should dictate.

The formulas provided by AASHTO for determining the length of parobolic vertical curves are in terms of the algebraic difference in grade and sight distance:

For $S < L$

$$L = \frac{AS^2}{200(\sqrt{H_e} + \sqrt{H_o})^2} \quad (10)$$

For $S > L$

$$L = 2S - \frac{200(\sqrt{H_e} + \sqrt{H_o})^2}{A} \qquad (11)$$

where: L = length of vertical curve, ft (m)

S = sight distance, ft (m)

A = algebraic difference in grades, percent

H_e = height of driver's eye above roadway surface, ft (m)

H_o = height of sighted object above roadway surface, ft (m)

Lengths of crest vertical curves are explicitly based on two driver characteristics: the perception brake reaction time required for stopping sight distance and driver eye height. The specification for the former is 2.5 seconds as discussed under Stopping Sight Distance.

Driver eye height is defined by AASHTO (5) as the vertical distance between the roadway surface and the driver's eyes. This vertical distance is a function of both driver and vehicle characteristics. The principal driver characteristic influencing the eye height is the driver's anthropometric measurements; in particular seated eye height, weight, and driving posture. The seated eye height is the distance between the seat and the eyes of the seated individual; obviously, this measurement when combined with the height of the seat relative to the roadway surface yields the driver eye height. In addition, however, the weight of the individual has a direct bearing on how compressed the seat springs and cushion are (and thus the height of the seat) when the driver is seated. Also, the driver has a preferred driving posture as reflected in the front-to-back positioning of the seat (which usually has a vertical component as well as horizontal) and the seat back angle (if it is adjustable in the particular vehicle model).

In AASHTO's A Policy on Geometric Design of Rural Highways (1) and in the MUTCD (3), the driver eye height is specified as 3.75 feet (1.14 m)* which was only slightly lower than the median eye height for the 1960 model year. The current draft of AASHTO's revised policy manual (5) specified a driver eye height of 3.5 feet (1.07 m) which is purported to represent the average for vehicles since 1960.

*A reduction to 3.5 feet (1.05 m) was recently proposed for the MUTCD.

Sag Vertical Curve Length

Lengths of sag vertical curves have been designed in the past using four different methods: 1) headlight sight distance, 2) rider comfort, 3) drainage control, and 4) a general appearance rule-of-thumb. AASHTO (1,5) has established the headlight sight distance method as the criterion to establish design values for sag vertical curve length.

The basic premise of the headlight sight distance method is that sight distance at night is dependent on the height of the headlamps above the roadway surface, the direction of the light beam, and the geometrics of the sag vertical curve. The formulas used to compute this distance as listed by AASHTO (1) are as follows:

for $S < L$

$$L = \frac{AS^2}{200\ (H + S \tan B)} \qquad (12)$$

for $S > L$

$$L = 2S - \frac{200\ (H + S \tan B)}{A} \qquad (13)$$

where: L = Length of sag vertical curve, ft (m),

S = Sight distance, ft (m),

A = Algebraic difference in grades, percent,

B = Divergence angle of light beam from the longitudinal axis of the vehicle headlight, degrees,

H = Headlight height, ft (m)

AASHTO (5) recommends the use of 2.0 feet (0.61m) for the headlight height and 1° for the upward divergence of the light beam from the longitudinal axis of the vehicle. Based on these two assumptions, the formulas reduce to:

for $S < L$

$$L = \frac{AS^2}{400 + 3.5S} \qquad (14)$$

$$\left(L = \frac{AS^2}{(122 + 3.5S)}\right)$$

for $S > L$

$$L = 2S - \frac{400 + 3.5S}{A} \qquad (15)$$

$$\left(L = 2S - \frac{(122 + 3.5S)}{A}\right)$$

The sight distance on a sag vertical curve should be designed to be no smaller than the stopping sight distance needed for vehicles traveling at the roadway design speed. The length of the curve is, therefore, a function of stopping sight distance which can be substituted for "S" in the above formulas. With length of sag vertical curve being dependent, in part, on stopping sight distance, the driver characteristic is the perception-brake reaction time.

Horizontal Curvature

The maximum degree of curvature, or the minimum radius, is a limiting value for a given design speed determined from the maximum rate of superelevation and the maximum side friction factor (AASHTO, 1). The minimum safe radius, R, can be calculated from the standard curve formula:

$$R = \frac{V^2}{15\ (e + f)} \qquad (16)$$

$$\left(R = \frac{V^2}{127\ (e + f)}\right)$$

where: R - radius of curve, ft (m)

V - vehicle speed, mph, (km/h)

e - rate of roadway superelevation, ft/ft(m/m)

f = side friction factor

At first glance it would seem that there is no explicit driver characteristic involved here. But further examination of the basis for selecting the design values of side friction factor, f, indicates that there may be.

From the dynamics of a vehicle operating on a curve and for given superelevation there is a speed, known as equilibrium speed, at which a car steers itself around the curve and the side friction factor, f, is zero. At other than equilibrium speed, side friction or a side thrust develops which is felt by the driver. There is a maximum value of "f" at which skidding is imminent and this depends principally upon the speed of the vehicle, the condition of the tires, and the characteristics of the roadway surface. As reported in the draft of the revised AASHTO design manual (5) studies show that maximum side friction factors developed between new tires and wet concrete pavement range from about 0.5 at 20 mph (32 km/h) to approximately 0.35 at 60 mph (97 km/h). For normal wet concrete pavement and smooth tires the value is about 0.35 at 45 mph (72 km/h). These values are applicable to automobiles.

However, curves are not designed on the basis of these maximum side friction factors. The AASHTO manual (5) states "the portion of the side friction factor that can be used with comfort and safety by a vast majority of drivers should be the maximum allowable value for design. In selecting maximum allowable side friction factors for use in design, one criterion is the point at which the centrifugal force is sufficient to cause the driver to experience a feeling of discomfort and cause him to react instinctively to avoid higher speed. The speed on a curve, at which discomfort due to the centrifugal force is evident to the driver, can be accepted as a design control for the maximum allowable amount of side friction."

So for this standard, there is a driver characteristic, driver (dis)comfort, which is a term to describe the driver's kinesthetic feeling as he drives through a curve. The discomfort threshold is likely to vary by the driver and the vehicle, specifically its center of gravity. Figure 2, extracted from AASHTO (5), summarizes the results of early studies of side friction factors. Special note is made of the Arizona curve which purports to represent the values at each speed where comfort ends and discomfort begins, i.e., the discomfort threshold. The straight line represents the values assumed for design and, therefore, represent the specification of the driver characteristic.

Lateral Clearance to Sight Obstructions on Horizontal Circular Curves

Sight distance for drivers of vehicles on horizontal curves can be obstructed by the terrain, cut slopes, walls, buildings, guardrail, etc. on the inside of the curve. In order to provide adequate sight distance for stopping or passing, it is necessary for these obstructions to be set back from the roadway pavement a sufficient distance for the driver to see across the inside of the curve. The required set back can be calculated from the following equation:

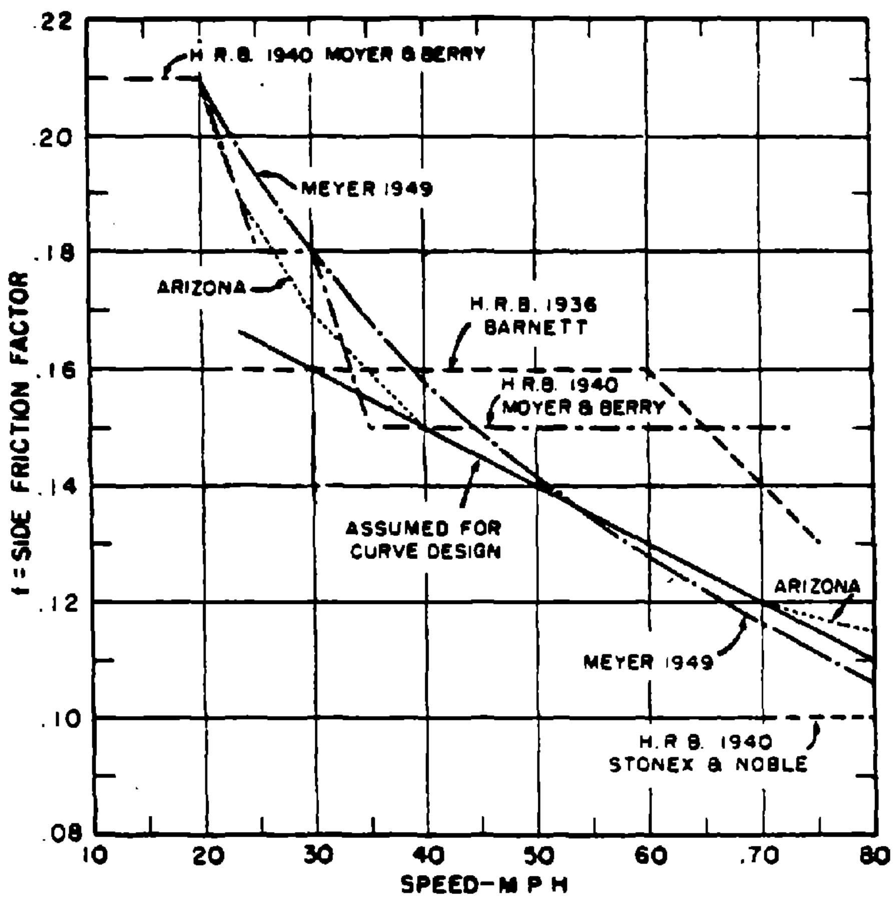


FIGURE 2--Maximum Safe Side Friction Factors

Source: AASHTO (5)

$$m = R \text{ vers } \frac{(28.65) S}{R} \qquad (17)$$

$$\left(m = R \text{ vers } \frac{(90) S}{R}\right)$$

where: m = minimum lateral clearance (or the middle ordinate of the horizontal curve) measured from the centerline of the inside lane to the sight obstruction, ft (m)

S = sight distance measured along the centerline of the inside lane, ft (m)

R = radius of the curve measured to the centerline of the inside lane, ft (m)

vers = 1 - cos

AASHTO (5) provides design standards for the provision of stopping sight distance. The AASHTO guidelines note that design for the provision of passing sight distance is usually impractical except on very flat curves. Because sight distance is an independent variable in the equations, either stopping or passing sight distance can be inserted to calculate the appropriate outputs. Using the minimum stopping sight distance as the lower limit and the desirable stopping sight distance as the upper limit, the minimum lateral clearance can be determined.

This AASHTO standard is simply an application of stopping sight distance formulation which in turn is based directly on the driver characteristic perception-brake reaction time. The AASHTO specification for perception-brake reaction time is 2.5 seconds.

Sight Distance Measuring Criteria

The various design sight distance requirements are verified through measurement. Sight distance is the distance along a roadway that an object of specified height is continuously visible to the driver. It is measured from a specified driver eye height to a specified object height and often recorded graphically on plans. In A Policy on Geometric Design of Highways and Streets (5), the height of driver's eye is specified as 3.5 feet (1.07 m) above the roadway surface. Consequently, stopping sight distance, passing sight distance on two-lane highways, decision sight distance, intersection sight distance, railroad-highway grade crossing sight distance and all other sight distances shall be measured from a driver's eye height of 3.5 feet (1.07 m).

Although the driver's eye height is an explicit specification of a driver characteristic, the object height is not. However, it could be considered indirectly related to a driver characteristic: vision and, in particular, visual acuity.

The object height specification for measuring stopping sight distance and decision sight distance is 6 inches (0.15 m). "The basis for its selection was largely an arbitrary rationalization of possible hazardous object size and a driver's ability to perceive and react to a hazardous situation." (5)

The current object height specification for measuring intersection sight distance, design passing sight distance for two-lane highways, and stopping sight distance on two-directional, one-lane roads is 4.25 feet (1.30m). This value represents the present average vehicle height. It is also noted by AASHTO (5) that "passing sight distances calculated on this basis (using 4.25 feet (1.30m) above the pavement as the object height) are also considered adequate for night conditions because the beams of the headlights of an opposing vehicle generally are seen in the daylight".

In the MUTCD (3), a 3.75 feet (1.07m) object height is specified to measure sight distance for placing no passing zone markings on completed highways. No rationale could be ascertained for this specification.

TRAFFIC OPERATIONS AND TRAFFIC CONTROL DEVICE STANDARDS

As with the geometric design standards, there are a variety of references which contain nationally recognized standards related to traffic operations and traffic control devices. However, the one reference which contained the majority of driver characteristic based standards is the Manual on Uniform Traffic Control Devices for Streets and Highways (3) approved and published by FHWA. Another reference which provided the standards included the ITE's Transportation and Traffic Engineering Handbook (16) and the Traffic Control Device

Handbook (17) which is currently under revision by FHWA. A discussion of the relevant standards follows.

Adequate Gap Time for School Crossing Traffic Signal Warrant

Warrants for traffic signals are listed in the MUTCD (3). One warrant pertains to school crossings and is stated as follows:

> A traffic control signal may be warranted at an established school crossing when a traffic engineering study of the frequency and adequacy of gaps in the vehicular traffic stream as related to the number and size of groups of school children at the school crossing shows that the number of adequate gaps in the traffic stream during the period when the children are using the crossing is less than the number of minutes in the same period. (Section 4C-6)

The recommended traffic engineering study is prescribed in the MUTCD (3):

> A recommended practice for determining the frequency and adequacy of gaps in the vehicular traffic stream is given in the Institute of Transportation Engineers publication, A Program for School Crossing Protection. (Section 7A-3)

The ITE publication (18) presents a step-by-step procedure to determine if traffic control is warranted. The second of the four step procedure is to compute an adequate gap time using the following equation:

$$G = 3 + \frac{W}{3.5} + 2(N-1) \qquad (1d)$$

$$\left(G = 3 + \frac{W}{1.07} + 2(N-1)\right)$$

where: G = adequate gap time, sec

W = width of roadway ft (m)

N = number of rows

The equation assumes that:(1) pedestrians cross in rows of 5 with 2 seconds between rows, (2) the average walking speed is 3.5 feet per second (1.07 m/s), and (3) it takes pedestrians 3.0 seconds to look both ways at the crossing, to make the decision to cross, and to start to walk across the street.

From the above discussion it can be seen that this standard is dependent on an adequate gap time which is a function of two pedestrian characteristics--pedestrian perception-reaction time and walking speed.

The pedestrian perception-reaction time is defined in the ITE recommended practice, A Program for School Crossing Protection (18), as "the number of seconds required for a child to look both ways, make a decision, and commence to walk across the street". The current specification employed in the standard is 3.0 seconds. A design percentile is not specified.

The pedestrian walking time is defined as "the number of seconds required to walk across the roadway of specified width without coming into conflict with passing vehicles" and is equal to the width of the roadway, in feet, divided by the walking speed, in feet per second. The current specification for walking speed employed in the standard is 3.5 feet per second (1.07 m/s). Although not explicitly stated, it can be inferred that the average walking speed is the average of children crossing the street individually, unimpeded by other children. The effect of other children is considered in the term, 2(N-1).

Vehicle Change Interval for Traffic Signals

For a vast majority of traffic signals in this country, there is a yellow vehicle change (clearance) interval following the green indication and prior to the red indication. The length of this interval is specified in at least three basic references: MUTCD (3) contains a standard for vehicle change intervals, the ITE Transportation and Traffic Engineering Handbook (16) has a standard for yellow change and clearance intervals, and the Traffic Control Devices Handbook, An Operating Guide (17) contains a standard for phase change intervals.

The MUTCD states that "the exclusive function of the steady yellow interval shall be to warn traffic of an impending change in the right-of-way assignment. Yellow vehicle change intervals should have a range of approximately 3 to 6 seconds. Generally, the longer intervals are appropriate to higher approach speeds". The MUTCD continues to say that after the yellow change interval is terminated, a short all-red clearance interval may be used in order "to permit the intersection to clear before cross traffic is released". Quite simply, the yellow change interval is intended to serve solely as a warning "that a red indication will be exhibited immediately thereafter when vehicular traffic shall not enter the intersection". In other words, according to the MUTCD, the 3-6 second standard yellow change interval is intended to be long enough so that at its termination all vehicles not yet in the intersection are capable of being brought to a stop. The MUTCD does not stipulate that the intersection be cleared of vehicles at the beginning of the red indication, only that no vehicles enter the intersection after the red indication is displayed. The MUTCD does not provide a mathematical formulation so there is no way to determine if a driver characteristic was considered in arriving at the 3-6 seconds.

The ITE Transportation and Traffic Engineering Handbook (16) treats both yellow change and clearance intervals. For yellow change intervals, the following equation is used to determine the minimum duration:

$$y_1 = t + \frac{1}{2}\frac{V}{a} \qquad (19)$$

where: y_1 = yellow interval, sec

t = perception-reaction time of driver, sec

V = approach speed, ft/s (m/s)

a = deceleration rate, ft/s^2 (m/s^2)

For a yellow clearance interval, which allows vehicles that have entered the intersection legally sufficient time to clear the point of conflict prior to the release of opposing pedestrians or vehicles, a longer period is required. The equation for this condition is:

$$y_2 = t + \frac{1}{2}\frac{V}{a} + \frac{W + L}{V} \qquad (20)$$

where: y_2 = nondilemma yellow interval, sec

W = width of intersection, ft (m)

L = length of vehicle, ft (m)

t, a, V = as described above.

In these equations there is an explicit driver characteristic--perception-reaction time. The specification for this characteristic is one second. Although not mentioned in the Handbook (16) the 1 second value can perhaps be traced back to a 1934 MIT research effort (19) which found that 95 percent of the sampled drivers had brake reaction times of one second or less when in an alerted condition.

The design deceleration rate, a, reflects the limitations of both the vehicle and the driver. For the vehicle the limitation is the coefficient of friction exerted between the vehicle tires and the roadway surface. The limitations imposed by the driver are essentially comfort-related, i.e., how much "g" force the driver is willing to accept. This combined vehicle/driver characteristic is being considered as part of another study which is evaluating vehicle characteristics related to geometric design.

Pedestrian Signal Timing

The MUTCD (3) contains standards for both pedestrian clearance and pedestrian walk intervals, which when combined form the basis for timing pedestrian signals. The pedestrian walk interval standard reads as follows:

> "Under normal conditions, the WALK interval should be at least 4 to 7 seconds in length so that pedestrians will have adaquate opportunity to leave the curb before the clearance interval is shown." (Section 4D-7)

Before the pedestrian actually begins crossing the intersection there are two processes which are assumed; perception-reaction time and sidewalk queue time.

Perception-reaction time (for the purposes of the following analysis of pedestrian walking speed) is assumed to include attention/fixation time, perception time, and response time for the decision-making process on whether or not to cross a street.

Once a pedestrian has perceived that it is safe and legal to cross the street, the pedestrian must move from the original observation point to the street prior to crossing the street and the time taken to leave the curb is negligible. However, in instances where large pedestrian volumes are present, pedestrians will form a queue at the curb and thus the pedestrian must also traverse part of the sidewalk prior to leaving the curb. This amount of time taken for this maneuver is a function of both the number pedestrians present (and thus the density) and the in-motion walking speed of pedestrians in advance of the other pedestrians. The 4 to 7 second pedestrian WALK interval is intended to accommodate both the pedestrian perception-reaction time and the discharge time from the curb.

The pedestrian clearance interval, which consists of a flashing DON'T WALK indication, standard reads as follows:

> "The duration of the clearance interval should be sufficient to allow a pedestrian crossing in the crosswalk to leave the curb and travel to the center of the farthest travelled lane before opposing vehicles receive a green indication." (Section 4D-7)

A formula is not provided but from the above statement it can be assumed to be:

$$PCI = \frac{D}{WS} \qquad (21)$$

where: PCI = pedestrian clearance interval, sec

D = distance from the near curb to the center of the farthest travelled lane, ft (m)

WS = walking speed, ft/sec (m/sec)

Walking speed is the pedestrian characteristic. The MUTCD assumes a "normal" walking speed of 4 fps (1.2 m/s).

Traffic Signal Face Location

The MUTCD (3) specifies the location of traffic signal faces in the following paragraph.

> "Except where the width of the intersecting street or other conditions make it physically impractical, at least one and preferably both of the (required) signal faces...shall be located between two lines intersecting with the center of the approach lanes at the stop line, one making an angle of approximately 20 degrees to the right of the center of the approach extended and the other making an angle of approximately 20 degrees to the left of the center of the approach extended, ...and not less than 40 feet nor more than 120 feet beyond the stop line." (Section 4B-12)

Implied in this standard is the driver characteristic--cone-of-vision which refers to that portion of the visual field which is normally in focus. By stating that the signals must be located within 20 degrees either side of center, a cone-of-vision specification of 40 degrees is implied.

Traffic Signal Face Visibility

The required minimum visibility distance for traffic signal faces is specified in the MUTCD in Section 4B-12. A formulation for determining the distances is not provided but based on the listed distances for the various 85 percentile speeds, they can be calculated using the following formula:

$$D = 100 + 15\ (V-20) \qquad (22)$$

$$(D = 30.5 + 2.85\ (V - 32))$$

where: D = minimum signal visibility distance, ft (m)

V = 85th percentile speed, mph (km/h)

In a review of this standard by FHWA, the required visibility distances were judged to be too low especially at the lower speeds. Consequently, a revised set of minimum visibility distances have been suggested for the MUTCD. Furthermore, the new Traffic Control Devices Handbook (17) will recommend these revised distances as a minimum visibility distance but also recommend higher desirable values. While not specified in either the revised MUTCD standard or in the Handbook, the two equations are:

a) Minimum visibility distance

D_m = desirable stopping sight distance, ft (m) + 50 ft (15m) (23)

b) Desirable visibility distance

D_d = desirable stopping sight distance + 3 sec times speed in fps (m/s) (24)

In tracing the development of the "desirable visibility distance" it was found that the additional three seconds was to allow for "detection and identification of the signalized intersection and the indications applicable to the approach." (20) In essence then this represents an additional perception-reaction time for the drivers. In summary, minimum and desirable visibility distances are based on the driver characteristic of perception-reaction time which is specified at 2.5 seconds for the minimum level and 5.5 seconds for the desirable level.

Sign Letter Height

Sign legibility involves the recognition of information on a sign by a motorist at a safe enough distance to allow sufficient time to comfortably execute appropriate driving maneuvers. In terms of sign legibility, legibility distance is the necessary distance for the motorist to read the sign before passing it, and involves both the processes of detection and recognition.

Letter legibility values, in terms of legibility distance per inch of letter height, have been determined in a series of field trails using 412 different people as observers (21). Subjects were instructed to record the distance at which they could read place names of varying letter height and width while standing, for different illumination conditions. Eightieth percentile legibility distance values (based on individuals with 20/20 vision) obtained during normal (day) illumination conditions were divided by the corresponding place name letter heights, and the following values were then established as constants to calculate sign letter height:

Letter Width	Letter Legibility, ℓ, in Feet Per Inch of Letter Height (m/m)
Narrow	33.0 (396:1)
Medium	42.5 (510:1)
Wide	50.0 (600:1)

The FHWA minimum letter and numeral size standards for highway guide signs referenced in Table II-1 of the MUTCD, apparently have been established by the application of the following formula for letter height (37).

$$H = \frac{L}{\ell} \qquad (25)$$

where: H = Minimum letter height required for sign letter height, rounded up to the nearest even inch

L = necessary legibility distance, ft

ℓ = appropriate letter legibility constant (ft per inch of letter height) according to the width of the chosen alphabet series.

From the above discussion it can be stated that there are three driver characteristics implicitly considered in sign letter height: (1) visual acuity, (2) minimum glance/reading time, and (3) perception-reaction time. No specifictions are explicitly established although they can be implied. For visual acuity it can be assumed to be a static visual acuity score of 20/20 since the testing that was done was based on subjects with 20/20 vision. For minimum glance/reading time and perception-reaction time, in the late 1930's Forbes (21) reported test results of 1.0 and 1.5 seconds, respectively. As was the case for the visual acuity tests, these tests likewise were conducted on subjects with normal vision.

III. PROFILES OR DRIVER CHARACTERISTICS

One of the major objectives of this study was to derive the distribution of the driver characteristics included in highway design and operation standards for the U.S. driving population. This chapter of the report describes what has been gleaned from available literature and data sources that can be used to establish a distribution. If a full distribution is not derivable, then, where possible, averages, minimum/maximums or some representative values are provided.

There are many human characteristics that are applicable to driving. For instance, McKnight (23) identified nearly 80 distinct driver characteristics, traits or conditions that are relevant to the tasks associated with driving. If one were to model the driving task in all its aspects (similar to McKnight's work and others) undoubtedly all of these characteristics would have to be considered. Individual characteristics alone and in combination affect the ability of the driver to perform the numerous tasks associated with driving.

Many of the characteristics listed for the driver also apply to the pedestrian. In addition there are other specific characteristics unique to a pedestrian such as walking speed, handicaps, etc.

However, while there are indeed numerous human characteristics involved in driving and walking, there are relatively few that are considered, or need to be considered, in standards for highway geometric design and operation. An in-depth review of numerous design manuals guide, handbooks, etc. revealed that there are only 21 specific design standards that explicitely consider a driver characteristic in its formulation. For these design standards there are only 10 driver characteristics since a few include the same driver characteristics or variations of it. Table 2 lists these standards, the driver characteristic and the specification.

The standards review also identified other specific standards which are based on a driver characteristic indirectly or in a purely qualitative way. Most of these standards involve the same characteristics listed in Table 2 but are not explicitly mentioned, and of course, there are no specifications.

Each of the principal driver characteristics is discussed in the remaining sections. In most cases the order of discussion is to describe or define the characteristic, how it is measured, what standards it applies to, what data is available on quantifying driver performance for the characteristic and, finally, what the driver population distribution, or average or some statistic is that can be used for the sensitivity analysis.

AGE AND SEX

Neither the age nor the sex of the driver are explicitly considered in any highway design or operations standards. However, the age and sex distributions of the driving population have a direct bearing on the distribution of driver characteristics. As a result they have affected and will continue to affect highway design and operational standards.

With regard to age, it can be seen from Table 3, that the distribution of the driver's age has changed considerably from the earlier years when most of the design standards and driver characteristic specifications were formulated. As of 1979, the percentage of drivers 60 years or older was 16 percent as compared to 11.4 percent in 1960 and only 5 percent in 1940. Table 4 further points to an increasing share of the elderly population driving--in 1969, 77.0 percent of the 50-54 age group had driver licenses; by 1979, this same group (now aged 60-64) had increased its percentage to 83.9. The 1979 percentage for the 50-54 age group has risen 10 percentage points to 87.0; thus, an increasing percentage of the individuals passing the age of 60 hold driver licenses.

The U.S. Bureau of the Census "Series II" population projections show an increase in the percentage of the total population which is over age 65. In 1980, the percentage was 11.2; by the year 2000, the percentage is projected to be 12.2; and by the year 2030, the percentage is projected to be 18.3 percent (24). If the present fertility rate continues at the current level of 1.8 instead of climbing to the Census "Series II" rate of 2.1, the future proportion of older people will be even larger. Based on these projections, it can be safely assumed that by the year 2000 the percentage of elderly (over 60) drivers will increase 10-20 percent from the current level of 16.0 to approximately 18 or 19 percent.

While elderly drivers are increasing as a percentage of all drivers they apparently are not driving as much. Data available from the U.S. Bureau of

TABLE 2--Standards Based on an Explicit Specification of a Driver Characteristic

STANDARD	DRIVER CHARACTERISTIC	SPECIFICATIONS *
Sight Distance, General:		
Stopping Sight Distance	Brake Reaction Time	2.5 sec.
Design Passing Sight Distance	Initial Manuever Time	3.6 - 4.5 sec.
Decision Sight Distance	Detection & Recognition Time	1.5 - 3.0 sec.
	Decision & Response time	4.2 - 7.0 sec.
	Maneuver Time	3.5 - 4.5 sec.
Sight Distance Measuring Criteria	Eye Height	3.5 ft.
Sight Distance, Specific:		
Intersection Sight Distance		
• Case I	Perception - Reaction Time	1.5 - 2.0 sec.
	Time to Brake or Accelerate	1.0 sec.
• Case II	(indirectly) Brake Reaction Time	2.5 sec.
• Case III	Perception Time & Time to Actuate an Automatic Shift	2.0 sec.
Railroad-Highway Grade Crossing Sight Distance		
• Case I	Perception - Reaction Time	2.5 sec.
• Case II	Perception - Reaction Time	2.0 sec.
Sight Distance Along a Ramp	(indirectly) Brake Reaction Time	2.5 sec.
Sight Distance Through a Grade Separation	(indirectly) Brake Reaction Time	2.5 sec.
Sight Distance At a Ramp Terminal	Perception Time & Time to Actuate an Automatic Shift	2.0 sec.
Horizontal Alinement:		
Lateral Clearance to Sight Obstructions on Horizontal Circular Curves	(indirectly) Brake Reaction Time	2.5 sec.
Vertical Alinement:		
Crest Vertical Curve Lengths	Eye Height	3.5 feet
	(indirectly) Brake Reaction Tiem	2.5 sec.
Sag Vertical Curve Lengths	(indirectly) Brake Reaction Time	2.5 sec.
Sag Vertical Curve Lengths Through a Grade Separation	Eye Height	3.5 feet
	(indirectly) Brake Reaction Time	2.5 sec.
Traffic Control Devices:		
Adequate Gap Time for School Crossing Warrant for Traffic Signal Installation	Pedestrian Walking Speed	3.5 ft./sec. (3)
	Pedestrian Perception - Reaction Time	3.0 sec. (18)
Traffic Signal Face Location	Cone of Vision	40 degrees (3)
Yellow Vehicle Clearance Interval	Perception - Reaction Time	1.0 sec.(16)
Pedestrian Clearance (DON'T WALK) Interval	Pedestrian Walking Speed	4.0 ft./sec. (3)

*Specifications come from AASHTO (5) unless otherwise noted by reference number in parenthesis.

TABLE 3--Distribution of Drivers by Age-Group and by Sex

AGE GROUP	1940 (1)	1950 (1)	1960 (1,2)	1965 (2)	1969 (3)	1979 (4)
16-19	12	11	7.2	9.8	9.0	8.3
20-24	18	12	11.2	10.4	13.1	13.3
25-29	11	12	12.7	9.6	11.3	13.0
30-34	13	13	12.5	10.1	9.3	11.6
35-39	12	12	11.6	11.1	9.1	9.3
40-44	11	11	10.3	10.8	9.7	7.6
45-49	8	9	9.1	9.7	9.7	7.1
50-54	6	7	7.8	8.5	8.2	7.1
55-59	4	6	6.2	6.8	6.8	6.8
60-64	2	3	4.7	5.2	5.4	5.6
65-69	1	2	3.1	3.7	3.7	4.5
70 and Over	2	2	3.6	4.3	4.7	5.9
Male	NA	NA	70	61	56.3	53.4
Female	NA	NA	30	39	43.7	46.6

Sources: (1) Marsh, B.W., (25)
(2) National Safety Council, Secondary Source, Berg (26)
(3) U.S. Bureau of the Census, (24)
(4) FHWA, (27)

TABLE 4--Percent of Population Registered to Drive by Age-Group and by Sex

AGE GROUP	MALE DRIVERS		FEMALE DRIVERS		TOTAL	
	1969 (1)	1979 (2)	1969 (1)	1979 (2)	1969 (1)	1979 (2)
16-19	70.1	61.2	53.6	52.6	61.9	56.9
20-24	90.6	97.7	76.5	87.5	83.3	92.6
25-29	96.1	100 (3)	79.4	96.0	87.6	100 (3)
30-34	93.2	100	77.4	95.1	85.1	100
35-39	94.9	100	75.8	91.8	85.1	98.0
40-44	94.6	100	74.5	87.6	84.3	94.8
45-49	94.0	98.6	72.9	83.3	83.1	90.8
50-54	92.6	95.9	62.6	78.7	77.0	87.0
55-59	90.5	96.0	53.4	75.9	71.1	85.5
60-64	87.4	97.5	45.6	71.8	65.1	83.9
65-69	74.0	92.8	38.9	58.8	54.6	73.9
70 and Over	61.8	82.2	20.2	35.0	37.0	53.2
TOTAL	87.0	94.1	61.5	75.5	73.6	84.4

Sources: (1) U.S. Bureau of the Census, (24)
(2) FHWA, (27)

Note: (3) Number of registered drivers equaled or exceeded census estimated population

Census indicates that drivers 60 and over account for about 10 percent of the total vehicle miles traveled and this has not changed from 1969 to 1979 (see Table 5).

With regard to sex, it can be seen from Table 3 that the percentage of the registered driving population which is female is approaching 50 percent. But although female drivers accounted for 46.7 percent of the registered drivers in 1979, only 27.6 percent of the VMT traveled in the U.S. is driven by females. While slightly longer brake reaction time have been observed for females (93), the principal influence of sex relates to their size and more specifically their lower eye heights.

TABLE 5--Annual VMT Driven by Age-Group and by Sex

AGE GROUP	PERCENTAGE OF TOTAL VMT 1969 (1)	1979 (2,3)
16-19	4.8	4.7
20-34	37.1	42.1
35-49	32.3	28.5
50-59	16.0	14.5
60 and Over	9.9	10.1
MALE	73.0	72.4
FEMALE	27.0	27.6

Sources: (1) U.S. Bureau of the Census, (24)
(2) FHWA, "Highway Statistics, 1979" (27)
(3) FHWA, "1977 Nationwide Personal Transportation Study" (28)

VISION

Various senses serve as input channels in driving, but vision is undoubtedly one of the more important. The driver depends largely on visual input to provide information regarding the driving environment. The driver's responses to that environment depend in part on his/her basic visual capabilities and how effectively they are employed at any given time; however, physiological factors alone cannot account for the efficieny of a driver's vision or the quality of his/her driving. This may explain why direct relationships between visual ability and driver performance have not been established.

Any driver characteristic requiring or incorporating visual perception detection or recognition will be impacted by the visual characteristics of the driving population. These would include information processing capacity, detection and maneuver time, perception-reaction time, etc. The factors in vision most pertinent to the driving task as they relate to the standards of interest include: visual acuity (static, dynamic, and kinetic), field of view, cone of vision, and color vision. The following sections consider each of these factors in terms of their physiological bases, their effects on driver characteristics forming the basis for design specifications, and where possible their distributions in the driving population.

Visual Acuity

Visual acuity has been defined in a number of ways, but is most easily expressed as the ability to resolve detail at a distance or the smallest visual angle of detail that can be discriminated. The limit of human visual acuity appears to be in the vicinity of one half minute of arc under clinical testing conditions.

A distinction can be made between three types of visual acuity: static, dynamic, and kinetic. Static visual acuity involves resolution when the target is stationary in the observer's field of view. Dynamic acuity involves resolution when there is angular movement of the target in the observer's field of view, regardless of whether it is the target or the observer in motion. Kinetic visual acuity involves resolution in those instances where the distance between target and observer is being changed without an angular movement of the target in the field of view. In this case, the target is moving towards or away from the observer, or the observer is moving towards or away from the target.

All three forms of acuity are important in driving. When both the driver and the target are stationary, such as stopped for a light and trying to read a street sign, static visual acuity is of concern. In approaching and trying to read an overhead information sign, the driver is moving towards the target, but the target remains in the cone of vision. This involves kinetic visual acuity. Resolving a child's ball bouncing into the street from the sidewalk or reading a sign mounted off the shoulder as one drives towards it involves dynamic acuity.

Static visual acuity is one of the few driver performance characteristics that is tested during licensing and for which standards are established. According to data gathered by NHSTA (29), 35 States require 20/40 vision or better in both eyes with glasses, 5 States require at least 20/50 vision, another 5 States require at least 20/60, and still another 5 require only 20/70 (although some of these last five have special requirements). Drivers in the last categories will have significantly less time to respond to visual stimuli since they will not be able to discriminate the target object until they are much closer to it than a driver with "average" visual acuity.

There is no standard test for dynamic or kinetic visual acuity, although some have been devised for research purposes (see Burg (30)). Currently no States test for dynamic or kinetic visual acuity.

The most comprehensive source of data on visual acuity in drivers comes from the extensive data collection efforts in California by Burg (31). Figure 3 shows the cumulative percentile distribution of static visual acuity scores for 669 passenger car drivers of both sexes (37 percent female) and all ages (16 years to 86 years). A prototype testing device was used to arrive at an acuity score comparable to Snellen test scores. Tests were conducted under normal illumination, low level illumination (night driving), and under two experimental conditions designed to simulate veiling and spot glare.

Scaling from the normal illumination curve, the following values are found:

- 50%tile -- 20/20 acuity or better
- mean -- about 20/26 acuity
- 85%tile -- 20/33 acuity or better
- 95%tile -- 20/40 acuity or better

Under illumination levels representative of night driving (2.3 to 0.2 ft-L (7.9 to 0.7 cd/m^2)), these values are much worse as shown below:

- 50%tile -- 20/28 acuity or better
- mean -- 20/132 acuity
- 85%tile -- 20/175 acuity or better
- 95%tile -- 10/190 acuity or better

While this data is for California drivers, there is no apparent reason for believing that the distribution would be much different for drivers from other states.

More recent data on driver's visual acuity is provided by Davison and Irving (32) for British drivers. The authors compared their results with those supplied by Burg for 3,848 California drivers (not the same sample as describe earlier). The comparison, indicating nearly equal proportions, is shown in Table 6.

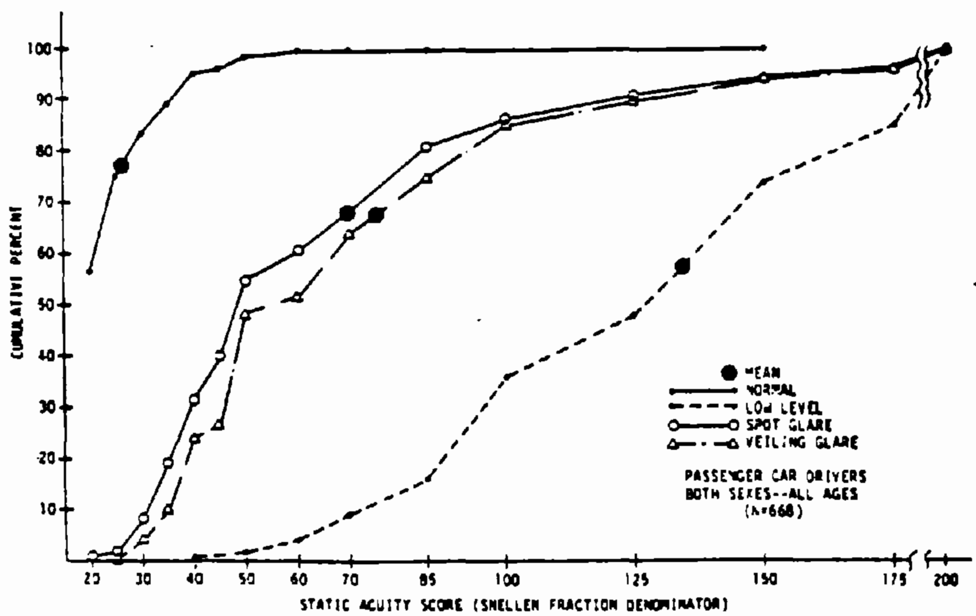


FIGURE 3--Cumulative Distribution of Static Acuity Scores for Passenger Car Drivers, Both Sexes, All Ages

Source: Burg (31)

TABLE 6 --Comparison of Acuities of UK and USA Drivers

Criterion	UK Drivers (N = 1368)	USA Drivers (Burg (31)) (N = 3848)
20/20 or better	90.7	85.8
20/30 or better	97.0	96.1
20/40 or better	99.0	99.0
20/60 or better	99.6	99.8
Worse than 20/60	0.4	0.2

Source: Davison & Irving (32)

Both static and dynamic visual acuity decline with advancing age, especially after about age 45. Trends in static acuity as determined by Davison and Irving and by Henderson and Burg (33) indicate 30 to 50 percent worse static visual acuity for a 70 year old compared to a 45 year old.

Dynamic visual acuity is generally poorer than static acuity, and this difference becomes more pronounced with increasing angular velocity of target movement. Further, the degenerative effects of age on acuity is more pronounced for dynamic acuity. These relationships were investigated by Burg (30) and by Allen et al (34). It also appears from their data that males have a slight but consistent advantage in terms of acuity (static and dynamic) over females.

Acuity is one of several factors affecting sign legibility. Therefore, one would expect that the elderly with poorer acuity would have less time to read signs. This was found to be the case from the work of Sivak et al. (35) who observed a significant reduction in nighttime legibility distance for older test subjects compared to younger subjects.

Field of View/Cone of Vision

The inner-most, interior surface of the eye, the retina, is the light-sensitive area containing the receptors for sight. These receptors are divided structurally and functionally into two groups, the rods and the cones. The cones function under conditions of high illumination and give rise to color vision. The rods function under conditions of low illumination and give vision only in shades of gray.

The point on the retina at which the central ray of incident light focuses is known as the fovea. The fovea covers about 1mm, or 2 degrees, of the retina and the central portion, or 1 degree, contains only cones. Cones continue to predominate out to about 5 degrees, beyond which point the rods are most numerous, reaching their maximum density at about 16 degrees from the center of the fovea.

It is this retinal physiology which determines the driver characteristics of field of view and cone of vision. The driver must of necessity possess some breadth of lateral visual awareness in order to pass approaching vehicles safely and to be aware of vehicles, pedestrians, or animals approaching from the side.

He must be able to see more than straight ahead (field of view), while "straight-ahead" must be defined in terms of an area of clearest perception (cone of vision).

Cone of vision refers to that portion of the visual field which is normally in focus; the primary line of sight plus a certain angular extension corresponding to distribution of cone cells in the fovea. Field of view is the area visible to the subject without head or eye movement, and is determined by the distribution of rods in the retina.

Any highway design standard based in part on a driver characteristic involving visual perception is impacted by the concepts of field of view and cone of vision. For example, perception-reaction time will be affected by the initial location of the stimulus. If the stimulus falls outside of the field of view, it will not be noticed and will not be responded to. If its image initially falls within the cone of vision, its detection and recognition should prove relatively rapid. As the initial image of the stimulus object approaches the periphery of the field of view, detection time increases, motion of the object becomes more critical for detection, and the image must be transferred to the cone of vision (through head or eye movement) for recognition to occur. All of these factors increase perception-reaction time.

Field of view without any movement of the head is generally considered to extend approximately 180 degrees laterally binocularly, and 130 degrees vertically (Henderson and Burg (33)). Figure 4 presents the cumulative distribution of lateral field of view of 17,249 California drivers, as derived from data by Burg (36). The mean value based on these figures is 171.6 degrees, and there is no reason to believe at this point that California drivers would differ significantly from drivers in other States.

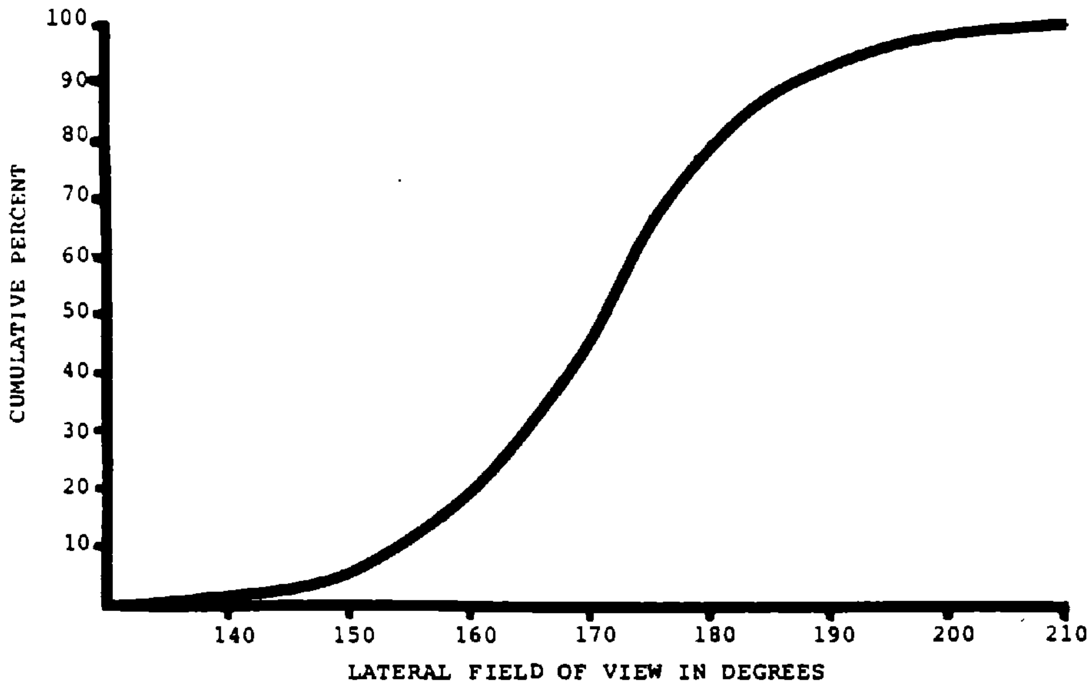


FIGURE 4--Cumulative Distribution of Lateral Field of View in Degrees Derived from Statistics by Burg (36) for 17,249 California Drivers

Source: Burg (36)

Currently, only 14 States require testing of, and set standards for, field of view for licensing purposes. Within this group, one requires only 70 degrees, one 100 degrees, one 110 degrees, three 120 degrees, one 130 degrees, and seven require 140 degrees for binocular field of vision (29). Based on the data of Burg, it would appear that practically all drivers (over 95 percent) have a field of view more than the maximum limit of 140 degrees.

Cone of vision is generally accepted to be 10 degrees (5 degrees to either side of the line of sight) on a theoretical level as reported by Mitchell and Forbes (37). However, no population data has been located which would define the exact nature of the characteristic distribution.

Color Vision

Only eight States currently set minimum standards of color perception for passenger car licensing. Of these eight, three require red, green, amber discrimination, and one requires red-green discrimination (29).

Table 7 presents the frequency of various types of color deficiencies in terms of the percentage of the population possessing the various visual systems. From these figures we can see that 0.0025 percent of the general population is totally color blind. Another 1.06 percent lack red-green color vision, and 3.15 percent are either red-green weak. Thus, a total of approximately 4.21 percent (predominately males) of the general population has some degree of difficulty with red-green color perception. Another 0.0026 percent have some degree of difficulty with yellow-blue perception. These figures most liley apply to the driving population as well, given so few States examine for color blindness.

DRIVER EYE HEIGHT

Driver eye height is defined by AASHTO (1) as the vertical distance between the roadway surface and the driver's eyes. This vertical distance is a function of both driver and vehicle characteristics. The principal driver characteristic influencing the eye height is the driver's anthropometric measurements; in particular seated eye height, weight, and driving posture. The seated eye height is the distance between the seat and the eyes of the seated individual. Obviously, this measurement when combined with the height of the seat relative to the roadway surface yields the driver eye height. In addition, however, the weight of the individual has a direct bearing on how compressed the seat springs and cushion are (and thus the height of the seat)

TABLE 7--Percentage of Population with Various Color Deficiencies

Designation by Number of Components	Nontheoretical Designation (Kries)	Percentage of Population that have these Visual Systems			
		Male		Female	
Anomalous Trichromatism	Protanomaly (red weakness)	1.0		0.02	
	Deuteranomally (green weakness)	4.9		0.38	
	Tritanomaly (yellow-blue weak)	0.0001		0.0000	
			5.90		0.40
Dichromatism	Protanopia (green blindness)	1.0		0.02	
	Deuteranopia (red &/or green blindness)	1.1		0.01	
	Tritatanopia (yellow-blue blindness)	0.0001		0.0000	
	Tetartanopia (yellow-blue blindness)	0.0001		0.0000	
			2.10		0.03
Monochromatism	Total Color Blindness	0.003		0.002	
Abnormal Systems			8.0		0.43
Normal Systems			92.0		99.57

Source: Reference (39)

when the driver is seated. Also, the driver has a preferred driving posture as reflected in the front-to-back positioning of the seat (which usually has a vertical component as well as horizontal) and the seat back angle (if it is adjustable in the particular vehicle model). The principal vehicle characteristic influencing driver eye height is the seat height; that is, the vertical distance between the seat and the roadway surface. The seat height dimension, however, is not solely dependent on the vehicle design; it also varies as function of how the vehicle is loaded. Another aspect of the vehicle which affects driver eye height is the seat back angle (if the seat back if adjustable, the seat back angle is variable and depends on the driver's preferred driving posture).

Measurement Techniques

A variety of techniques have been used over the years to measure driver eye height; each falls within either of two basic categories: dynamic and static. (The results from 10 studies using these techniques are summarized in Table 8.) Static measurements are those taken while the vehicle is stationary. This technique generally involves the measurement of applicable seat dimensions (e.g., seat height and seat back angle) for all (or a sample of) vehicle makes for the study year(s) and combining this data with measurements taken of drivers seated in a mock-up driver's seat. This scientific approach of breaking the characteristic (driver eye height) into its components and analyzing each individual component certainly has its advantages. In terms of vehicles, historical trends can be drawn on the average seat height (and, for that matter, the lowest, highest or any percentile in between) for a particular fleet year based on vehicle sales. The measurements of individual drivers can be disaggregated by age and sex and subsequently be manipulated to depict total exposure that reflects changes in VMT occurring for age and sex groups. A refinement to the static measurement technique has been the development of eyellipse distributions for stationary vehicles that account for both the variety of seat back angles and the elliptical motion of the eyes when driving. Recent trends have been away from the static measurement technique and toward the dynamic measurement technique because of its direct results from "real world" operations of vehicles. A variation of the static technique has been the determination of driver eye height by assuming a standard distance between the top of the vehicle and the driver's eyes for particular percentiles of drivers. For example, studies (40) have shown that the median (50th percentile) driver eye height is approximately 10 inches (0.25 m) less than that overall vehicle height in American-made cars.

Dynamic Measurements are those made while the vehicle is in motion. The driver is photographed while operating the vehicle and the eye height is calculated from the photographs. These dynamic measurements depict actual vehicle loading conditions and actual driving postures and thus could be more representative of actual driver eye heights than the static measurements. However, the dynamic measurement technique has several shortcomings when compared with the static measurement technique. Perhaps the most important shortcoming is the potential for more imprecision in the eye height measurement than occurs in the static technique. For example in the 1960 study conducted by C.E. Lee (40), the smallest legible gradation in the photographs corresponded to a change in driver eye height of 0.25 inches (6.35 mm); in the 1979 study conducted by W. Cunagin and T. Abrahamson (41), the smallest measureable gradation corresponded to a change in driver eye height of 1.0 inches (25.4 mm).

Site selection for the field measurements is critical to the validity of the technique. The horizontal and vertical geometrics of the roadway and the presence of peripheral distractions (e.g., signs) will affect driver eye height. The time of day for the measurements also has a direct bearing on driver eye height; in particular, the position of the sun and the presence of glare can cause the driver to significantly change the driving posture and thus the eye height. For this reason, dynamic measurements are typically conducted during the mid-day. However, the common practice of taking mid-day measurements suggests another shortcoming of the dynamic technique--the potential for the sample to not be representative of all VMT. It could be conjectured that the mid-day driving population is comprised of a greater percentage of the young, the elderly and females than is found the rest of the 24-hour period. Likewise, locale can be a significant factor; both a college town and rural areas would have younger drivers than large cities. This potential skewing of the driver eye height distribution needs to be recognized in any analysis of dynamic measurements taken of an uncontrolled sample.

TABLE 8--Historical Summary of Automobile Driver Eye Height Study Results

YEAR OF STUDY	VEHICLE YEAR STUDIED	DRIVER EYE HEIGHT (in.) — See Note 1 MINIMUM	95%	85%	50%	WEIGHTED MEAN	STATIC(S) OR DYNAMIC(D)	AUTHOR OR REFERENCE	COMMENTS
1958	Mid-1930's				56.5		S	Stonex (46)	
	1950	43			48.5				
1960	FLEET			47.4	50.8		D	Lee (46)	Recommended 3.95 ft (47.4 in) as Design Standard; Sample of 761
1963	1962	39.5			48	46.3	D	Lee (47)	Recommended 3.5 ft (42 in) as Design Standard; Australian Study.
1970	1969		39.4			44.4	S	Segar (48)	
1970	1970				47.2		D	Anderson (49)	Australian Study
1974	1971-3		41.2			43.8	S	Boyd (45)	Canadian Study
1978	1975-6	40		41.9		44.3	D	Boyd (45)	Domestic Cars Only; Mean for Compacts was 41 in.
1978	1976	39.7	40.7	41.9		43.4	S	MVMA (44)	Domestic Cars Only
	1978	39.7	40.3	41.4		43.4			
1979	FLEET	40.2		41.8		43.3	D	Cunagin (41)	Sample of 161; only 11% of Compacts and 27% of Full size cars exceeded 3.75 ft. standard
1981	1976	36.3*	37.8*			43.3	S	Lee (38)	*All other "Static" Testings in this Table defined 95th percentile as the average driver in a 95th percentile car. This study defined it as the 95th percentile driver in the 95th percentile car. Likewise, the "minimum" Figures are based on average driver and 95th percentile driver, respectively, in the shortest height car.
	1979	36.3*	37.8*			43.1			

NOTE: (1) Minimum is the shortest observed driver eye height. The percentiles indicated what percentage of sample population exceeded the identified height. Weighted mean is balanced to reflect distribution of auto sales.

The metric conversion units are 1 in = 25.4 mm and 1 ft = 0.3 m.

Driver Eye Height Specifications

Starting in the late 1940's, the commonly used driver eye height for highway design was 54 inches (1.37 m). Several studies in the late 1950's and early 1960's prompted AASHTO to adopt a driver eye height of 45 inches (3.75 ft) (1.19 m) in A Policy on Geometric Design of Rural Highways (1). The latest MUTCD (3) issued by the U.S. DOT in 1978 also uses the 45 inch driver eye height standard. AASHTO is currently developing an updated version of the "Blue Book." The latest version (5) recommends a driver eye height of 42 inches (3.5 ft) (1.07 m). This downward movement of recommended driver eye heights has followed the American trend toward increased reliance on foreign compact cars and downsizing of full size and intermediate car models. Similarly, the Roads and Transportation Association of Canada (42) has adopted an eye height value of 1.05 m (41.3 in). The standards for driver eye height that are used in European countries are typically lower than the current American standard of 45 inches (1.14 m) and proposed standard of 42 inches (1.07 m). As reported at an OECD meeting in 1976 (43), the United Kingdom uses 41.3 inches (1.05 m) and Germany and France use 39.4 inches (1.0 m). Australia at the time was also using the 3.75 feet (45 in) (1.14 m) standard.

Distribution of Driver Eye Height for Driving Population

As shown in Table 8, four studies of auto driver eye height have been reported in the past four years. Despite the availability of data from these recent studies, it is not possible to directly develop a percentile distribution of American auto driver eye heights for the current or a recent point in time. Each of the four has a flaw.

The 1978 study "Driver Eye Height Comparison for 1976 and 1978" by MVMA (44) presents a distribtuion of driver eye heights based on actual sales of 1976 and 1978 American-made automobiles. The listed driver eye height for each model automobile is based on the estimated seated characteristics of the "average" driver. Thus, the 85th percentile driver eye height listed for 1978 by MVMA should not be construed to be the same as the 85th percentile of all individuals that drive 1978 fleet automobiles; instead, it depicts the 50th percentile driver seated in the 85th percentile automobile of the 1978 fleet. As noted, the distribution pertains only to American-made autos which in 1976 and 1978 comprised 81.1 and 81.9 percent of all new car sales in the U.S.

The MVMA study does, however, provide useful data on relationships between different percentiles of vehicle seat heights. The study found that the 85th, 90th and 95th percentile heights dropped 0.3-0.5 inches (8-13 mm) between 1976 and 1978. Also, the difference in the median driver's eye height seated in the 50th percentile car from the height estimated for the 95th percentile car was 2.7 inches (69 mm) in 1976 and 3.1 inches (79 mm) in 1978.

The 1978 study "Determination of Motor Vehicle Eye Height for Highway Design" by Boyd (45) gathered data on automobiles considered representative of 1974 through 1977 model years based on relative sales. The study involved dynamic measurements of randomly sampled cars and thus was more indicative of the actual distribution of driver height and statures than was the MVMA study. However, in the Boyd study only 3 of the 15 models reported were small or compact cars and none were imports. Ward's Automotive Yearbook for 1975 estimated that in 1975 approximately 12 percent of all operating automobiles in the United States were imported. Despite this shortcoming, the study's relationships between percentiles are useful because of their accounting for "real life" distributions of varying driver sizes in varying vehicle sizes. The Boyd study, as shown in Table 8, found that the 85th percentile driver had an eye height 2.4 inches (61 mm) less than the median driver eye height.

The 1979 study by Cunagin and Abrahamson (41), "Driver Eye Height: A Field Study," involved dynamic measurements of random automobiles and their drivers. The technique resulted in a fairly representative sample of the actual automobile fleet in 1978 (42 percent of the observed automobiles were compacts or smaller; according to Ward's Automotive Yearbook the current split between compacts and larger cars is practically even). Despite the fact that this study involved dynamic measurements of a properly (relatively) distributed sample, two shortcomings of the study prohibit the direct extrapolation of the data as a means of determining the current distribution of driver eye heights. First, the photograph techniques used by Cunagin et al. limited the estimated driver eye heights to be read to no greater level of precision than one-tenth of a foot (0.03m). This relative imprecision is especially important when it's realized that the 1978 MVMA study described above revealed one-tenth foot (0.03m) increments between the 75th, 85th and 95th percentiles. The second weakness of the study is its relatively small sample size--only 148 automobiles were photographed as compared to 195 by Boyd et al. in 1978 and 761 by Lee in 1960.

The 1981 study by Lee and Scott (38) "Driver Eye Height and Vehicle Sales Trends" which was based on a 1980 study by R.L. Lee used static measurements to estimate driver eye heights. Unlike previous static measurement technique studies, this analysis estimated the eye height of the 95th percentile driver seated in the 50th, 85th, 95th and 100th percentile vehicle. These estimates were based on the assumption that the 95th percentile driver has an eye height 1.7 inches (43 mm) less than that of the 50th percentile (or median) driver which in turn is estimated to be 10 inches less than that of the vehicle itself. This figure of 1.7 inches (43 mm) was drived from the Society of Automotive Engineers eyellipse template.

In contrast, a 1969 study by Stoudt (50) of static seated individuals indicated a 50th-to-95th percentile increment of 2 inches (51 mm) for males and 3.3 inches (84 mm) for all drivers when males and females are weighted according to their driving exposure; a 1980 study by Haslegrave (51) revealed a 50th-to-95th percentile increment of 2.3 inches (58 mm) for males and 3.3 inches (84 mm) for all drivers. The identical results of the Stoudt and Haslegrave studies would indicate that the 1.7 inches (45 mm) used in the Lee study should be approximately doubled. Another probably erroneous assumption made in the Lee study is that a 95th percentile driver (albeit derived using the "1.7 inch rule") seated in the 95th percentile auto will yield the 95th percentile driver eye height. Although this assumption is possible, it is nevertheless more likely that the "shortest" 5 percent of all drivers do not just drive the "shortest" 5 percent of the auto fleet. Therefore, the Lee study estimates for 95th percentile driver eye heights are based on two erroneous assumptions which have conflicting (and possibly negating) effects: (1) the underestimate of a 1.7 inch (43 mm) increment between the 50th and 95th percentile driver would indicate that the 95th percentile eye height is calculated too high, and (2) by combining the 95th percentile auto with the 95th percentile driver, the study produces a 95th percentile driver eye height which is calculated too low. One final point, the assumed 10 inch (254 mm) distance between the eye height and the top of the vehicle may be too large because cars have become smaller and more compact since the 10 inch (254 mm) increment was measured.

Current Driver Eye Height Estimates

By using the data from the above four studies (and recognizing the limitations of the data), the distribution of auto driver eye heights for the current fleet is estimated to be as follows:

- 50th percentile (median) is estimated to be 43.1 inches (1.09m)--Lee study of median heights for 1979 domestics and imports yielded a figure of 43.1 (1.09m); in 1978 Cunagin estimated a total fleet median eye height of 43.3 inches (1.10m) which would have been expected to decrease a small amount in the past 3 years.
- 85th percentile is estimted to be 41.1 inches (1.04m)--the incremental change in total fleet driver eye heights between the 50th and 85th percentiles measured by Boyd and Cunagin were 2.4 and 1.5 inches (61 and 38 mm) respectively; by applying this factor to the 50th percentile estimate and averaging the result, a height of 41.1 inches (1.04m) is obtained.
- 95th percentile is estimated to be 40.2 inches (1.02m)--by applying the MVMA--developed relationships between the median, 85th and 95th percentile vehicle heights in 1976 and 1978, an average figure of 40.2 inches (1.02m) is obtained.

These values are, of course, crudely developed estimates. Comprehensive field tests offer the only proper means of obtaining reliable estimates.

Truck Driver Eye Height

The 1978 study by Boyd (45) estimated a median driver eye height of 100.8 inches (2.56m) for cab-behind-engine truck models and 94.1 inches (2.39m) for cab-over-engine truck models (American-made only). The "1977 Census of Transportation, Truck Inventory and Use Survey" (52) estimated a distribution of VMT by cab-behind and cab-over models to be 52.7 percent to 47.3 percent. The U.S. FHWA annual publication "Highway Statistics" (27) estimated that in 1979 these truck types accounted for approximately 4.4 percent of all VMT in the U.S. Thus a design that does not account for the highest 5 percent driver eye heights effectively excludes all large trucks. An estimated maximum driver eye height could be computed by combining the maximum median driver eye height for various COE truck makes (which is 102.8 inches, 2.61m) with the relationship between 50th and 99th percentile driver seated eye heights (3 inches, 76mm) as observed by Stoudt and Haslegrave and verified by Sanders (53). The resultant "maximum" driver eye height is 105.8 inches (2.69m). The "minimum" driver eye height for large trucks probably approximates the "minimum" median driver eye height observed by Boyd (45) for CBE trucks--89.3 inches (2.27m). Each of these driver eye height values is greater than the 72 inch (1.83m) specification value used in the 1965 AASHTO "Blue Book" to develop minimum lengths for sag vertical curves through grade separations.

Projected Future Trends

The task of projecting driver eye height trends for the future is a difficult one, not only because of the uncertainty of future auto sizing trends but also because of uncertainty as to the accuracy of the estimates for the current driver eye height distribution. Lee (38) showed that the 95th percentile vehicle and the lowest domestic vehicle have been holding steady for the past 5 years at vehicle heights of 39.4 and 38.0 inches (1.0 and 0.96m), respectively. As these compacts become a greater proportion of the total vehicle fleet, the median driver eye height will decrease. However, the 95th percentile will remain relatively stable at its current estimated level of 40 inches (1.02m) with its probable lower limit being approximately 39.5 inches (1.00m). In addition, the 85th percentile value will approach (but probably never reach) the 95th percentile as the percentage of compacts and subcompacts increases.

PERCEPTION-REACTION TIME

Perception-reaction time consists of three components: the time it takes for a driver to perceive and recognize a particular object or situation, the time it takes to decide what action should be taken, and the time it takes to initiate the action (e.g., move foot onto the brake pedal and activate the brakes).

The duration of each of these components is a function of the particular situation and of the location, type, and relative movement of the object to be sighted. For that reason it is necessary to address the driver characteristic perception-reaction time in terms of the particular standard involved. Discussed below are several highway design and operations standards and their appropriate perception-reaction time values including those for stopping sight distance, intersection sight distance, railroad grade crossing sight distance, passing sight distance, and vehicle clearance interval.

Stopping Sight Distance

The standard for stopping sight distance pertains to a situation where a driver of a vehicle on a tangent section, vertical curve, or horizontal curve sights an object in the roadway ahead and stops the vehicle prior to reaching the object (thus, the commonly used term perception-brake reaction time).

The current 2.5 second specification value appears to be based on the results of the Johansson and Rumar study (54) which measured the brake reaction times of 321 drivers under an anticipated condition and a much smaller sample under surprise conditions. The researchers concluded that on 10 percent of the occasions (test), brake reaction time was estimated to be 1.5 seconds. On what basis the additional one second was added to arrive at 2.5 seconds is not clearly stated in AASHTO Manual (5) but presumably it was added to account for the perception time. A careful review of the Johansson and Rumar study reveals that what was really measured is brake reaction time exclusive of any perception time since the subject, regardless of whether they were alerted or not, knew they were to apply the brakes upon hearing a signal (a horn) in the car. As stated in the AASHTO Manual the 2.5 seconds is supposed to be "large enough to include the reaction time required for nearly all drivers under most highway conditions" (emphasis added). It is implied that 90 percent of drivers constitutes "nearly all" drivers.

Perception-brake reaction times can be determined in either of two ways: experiments which measure the entire perception-brake reaction time or by simply adding the individual values experimentally determined for each of the components, perception, decision, and limb movement. The first method is preferred because it is more realistic. The processes of detection, perception, decision making and physical response are often overlapping and can not simply be added as step-by-step tasks. For instance, the driver can take his/her foot off the accelerator while he/she decides whether or not to stop.

There are numerous studies which have attempted to develop data on perception-brake reaction time or components of it. A good summary of most of these is found in a recent paper by Taoka (55). Table 9 summarizes the results of the various studies on brake reaction time. The first group are experiments that were conducted under simulated conditions in the laboratory or field controlled conditions. As such the values (primary

TABLE 9--Summary of Studies on
Brake Reaction Time Studies

Investigator/Date	Sample Size	Mean	Standard Deviation	Percentile Values 50	75	80	85	90	95	99	Comments
LABORATORY/FIELD CONTROL STUDIES											
1. Greenshields/1936/	1,461 13 27	0.496 0.86 0.74	0.0913								laboratory automobile
2. Forbes & Katz/ 1936 & DeSilva & Forbes/ 1937	907	0.64									
3. Jones et al (reported in Forbes & Katz/ 1936)	889 truck drivers	0.697	0.121								
4. Konz (1967)	12 40	0.59 0.47									
ON THE ROAD STUDIES											
1. Moss and Allen/ 1925	46	0.54							0.88	1.01	response to auditory detection
2. Massachusetts Institute of Technology/1934	144 men 36 women	0.66		0.60				0.85	1.0		response to brake light of car ahead
3. Drew/1968	1,000	0.57 men 0.62 women									Alerted condition
4. Norman/1953	53	0.73		0.73	0.77	0.83	0.85	0.89	0.96	1.1	alerted condition
5. Johansson & Rumar/ 1971	321	0.75		0.63	0.82	0.86	0.92	1.05	1.21	1.60	expected, auditory
	5	0.89		0.85	1.11	1.16	1.24	1.42	1.63	2.16	surprise, auditory
6. Mortimer/1970	80	1.30		1.42		1.88			2.56	3.29	response to brake light of car ahead
7. Sivak/1981	311	1.23	0.62	1.04	1.55	1.70	1.88	2.15	2.36	2.68	response to brake light of car ahead

means) are considered only brake reaction times under expected conditions. Taoka refers to it as "simple laboratory response time", which is not indicative of actual driving situations.

The second group are results of field driver response experiments which attempt to duplicate actual conditions. All of the studies have deficiencies inherent in their procedure which makes their results less than ideal. Most measured subjects were already alerted and anticipating a signal and some were responding to an auditory signal. Visual perception of objects, other than a brake light ahead that would require a motorist to stop, were not considered in these studies.

Visual perception can involve several components: latency, eye movement, fixation and focusing (detection), and finally recognition. For the purposes of this study, an object is perceived once it has been detected and recognized as the object.

For a laboratory study, latency is defined as the delay between the time the stimulus is presented and the time the eyes begin to move to the stimulus. This has relevance to the highway when the object is in the peripheral vision of the motorist either because the object is off the side of the road while the motorist is fixating down the road or, more importantly, if the object is in the travel lane and the motorist is fixating away from the object. Such might be the case if the motorist is inattentive (daydreaming, fatigue, etc.), or distracted or in the course of normal head and eye movements. That a driver might not be fixating down the travel lane is common enough that this scenario should be considered in the perception process. (An argument against this assumption can be based on Rackoff and Rockwell's (56) studies of eye movements and fixations.) During the day they found that their test subjects fixated straight ahead 92.6 percent of the time on freeways and 64 percent on rural highways. However, these subjects were in a more attentive state and had helmets on which would limit their "normal" head movements).

Data on latency eye movement times are provided from laboratory studies by Bartlett and his colleagues (57) which examined the cumulative distribution of latencies of eye reaction to stimuli located 10, 20, and 40 degrees off the visual axis. The various percentile values for the 20 degree curve, which is not unrealistic for driving situations, are as follows:

Percentile	50	75	80	85	90	95	99
Latency (sec)	.24	.27	.29	.31	.33	.35	.45

These data are based on only 3 subjects. Eye movement times for a target 20 degrees off the visual axis averages about 0.09 seconds according to White el al (58). This value is compatible with the 0.15 to 0.33 seconds cited by Matson, Smith & Hurd (59) as the time for moving the eye to fixate to the left or right in scanning an intersection scene--a situation with a much wider angle than 20 degrees.

The time it takes to bring the object into focus on the retina can be considered minimum fixation time. Data on this component is skimpy and each is qualified by its own experimentation apparatus, precedure, and purpose. Matson, Smith & Hurd (59) cite a range of 0.1 to 0.3 seconds for fixation time and Mourant et al. (60), reported mean fixation times of 0.27 seconds of various objects during open road driving. No studies were uncovered that would yield reliable distribution profiles for this component.

The last component of the perception process is termed the recognition phase and is defined here as the time for the brain to interpret the image that the eye has focused on as a recognizable object. For many targets this recognition phase is, in all likelihood, instantaneous with detection. But as objects become less familiar to the motorist and where legibility and reading are required, this recognition phase can take on a measurable time period. The object height used for stopping sight distance is 6 inches, which was arbitarily selected by AASHTO as "representative of the lowest object that can create a hazardous condition and be perceived as a hazard by a driver in time to stop before reaching it" (5). Objects this low would be animals, rocks, or vehicle debris such as a muffler. More common objects particularly at intersections would be pedestrians and vehicles, both of which exceed the 6 inch (1.5 m) object height.

The fact that recognition time is a mental process makes it nearly impossible to measure it alone. The recognition component can not be isolated from the total information gathering process and, consequently, is measured only as part of the total perception phase. Data which could be used to approximate this component is available from the work of Ells and Dewar (61) and Ells et al. (62). Ells et al. found the mean response time of 12 subjects responding to sign targets after being detected to be from 0.42

to 0.48 seconds. In another similar study, Ells and Dewar found this to be about 0.6 to 0.7 seconds.

Although it has not always been recognized, there can be a decision process involved in the perception-brake time. For the purposes of stopping sight distance, the amount of decision time is probably inversely proportional to the amount of time remaining before collision. This is to say, if a panic maneuver is necessary to avoid a collision with an opposing vehicle, then the decision time is likely instantaneous with the moment of perception. However, a review of the literature could neither confirm nor refute this hypothesis. The most pertinent data available from the literature is that of Lunenfeld (63) which states that 85th percentile driver decision times would be as follows for both expected and unexpected situations:

Information Content (Items)	Decision Time Expected	Decision Time Unexpected
0	0 sec.	0 sec.
1	0.7 sec.	1.0 sec.
2	1.3 sec.	1.6 sec.
3	2.0 sec.	2.6 sec.

For the case of sighting an object in the roadway, the decision is relatively simple and thus it is likely the decision time will fall between 0 seconds and a maximum of 1.0 seconds.

The last component is brake reaction. The values suggested are those from the Johansson and Rumar study (54) under an unalerted condition.

To summarize, Table 10 shows the various percentile values for the individual components of the perception-brake reaction time as suggested by the literature. Three totals are shown. If it is assumed that all the components should be included, then "Total A" applies; if it is assumed that the driver is fixating down the road, then latency and eye movement could be deleted and "Total B" applies, and if it is assumed that there is no decision making component but latency and eye movement are included then "Total C" applies. In all cases, it has been assumed that the driver is in an unalerted condition and that he is not expecting to have to stop.

For most of the upper percentile levels, these values are higher than the current specification of 2.5 seconds. While higher, they are not unrealistic when compared to the perception-brake reaction times cited by Mortimer (64) and Sivak (65) (see Table 9). However, they should not be considered a statistically reliable distribution of the driving population. They are based on estimates, assumptions and data from experimental procedures not truly indicative of actual conditions. Furthermore, they are derived from summations of components of the process. As discussed previously, this may not be realistic because the human is capable of time-sharing sensory, information processing and psychomotor tasks.

It is worthwhile noting that recent Canadian research as reported by Scott (42) recommends the use of a variable time value for desirable perception-reaction time. The desirable time would vary as a function of vehicle speed--as speed increases, the perception-reaction time likewise increases. The Canadian desirable values range from 2.5 seconds at a speed of 25 mph (40km/h) to 3.5 seconds at a speed of 85 mph (137 km/h) These values were developed from an analysis of the theoretical points in time at which various proportions of a 6 inch (0.15m) object come into view on a crest vertical curve. Low and high speed curves were investigated and it was found that on higher speed curves the "viewed" portion of an object grows more slowly than it does on a lower speed crest vertical curve.

Decision Sight Distance

As discussed in Chapter II, Decision Sight Distance is a new geometric design standard to be included in the revised AASHTO Manual (5). Essentially, the standard recognizes that for certain complex highway situations, such as freeway interchanges, toll plazas, lane drops, etc. enough sight distance ought to be provided so that the driver can detect, and recognize the situation, decide on alternative courses of action, select the appropriate couse, and perform the required maneuver in a safe and efficient manner. It also assumes that the situation or object to be sighted may be in a visually cluttered environment which may require longer perception times as well as decision times.

The driver characteristics, therefore, is a perception-reaction time but it involves more complex information processing and reaction than for stopping sight distance. The only driver performance data for this characteristic was that analytically developed by McGee et al. (8) and then empirically validated using only 19 test subjects varying in age from 16 to 60, both male and female. This data became the basis for selecting the range of perception-reaction times found in the standard. The values which represent the total time for detection,

TABLE 10--Perception-Brake Reaction Time for Various Percentiles of Driving Population

	Percentile of Drivers					
Element	50	75	85	90	95	99
1. Perception						
a. Latency	0.24	0.27	0.31	0.33	0.35	0.45
b. Eye Movement	0.09	0.09	0.09	0.09	0.09	0.09
c. Fixation	0.20	0.20	0.20	0.20	0.20	0.20
d. Recognition	0.40	0.45	0.50	0.55	0.60	0.65
2. Decision	0.50	0.75	0.85	0.90	0.95	1.00
3. Brake Reaction	0.85	1.11	1.24	1.42	1.63	2.16
Total A (1a-d+2+3)	2.3	2.9	3.2	3.5	3.8	4.6
Total B (1c,d+2+3)	2.0	2.5	2.8	3.1	3.4	4.1
Total C (1a-d+3)	1.8	2.1	2.3	2.6	2.9	3.6

recognition, decision and response are dependent upon speed and range from 5.7 to 10 seconds. The lower value was established as the minimum applicable to situations of moderate complexity or visual clutter and the upper value as desirable for highly complex or visually cluttered locations.

Intersection Sight Distance--Case I and II

Case I and II intersection sight distance enables the driver of a vehicle approaching an uncontrolled intersection to determine whether a speed adjustment or a complete stop is necessary in order to avoid a collision with another vehicle (or vehicles) approaching the intersection on a conflicting leg.

The perception-reaction process in this case is the ability of a driver to perceive a vehicle moving across his/her path, judge its trajectory in relation to his/her vehicle and then decide whether some speed adjustment is necessary to avoid collision. A literature review did not uncover any studies on how long it takes drivers to perform this task. In the absence of any empirical research, estimates of the actual distribution of perception-reaction times for the driving population may be based on a sum of the times for the components of the process determined from the available literature.

If one were to model the driver's task for this situation, the following steps would likely be considered:

1. Driver picks up (through peripheral vision) an object moving towards the intersection.
2. After a latency period, eye and/or head movement to detect the object.
3. Object is recognized as vehicle.
4. Opposing vehicle's speed and time to reach intersection are estimated.
5. Decision is made whether deceleration or acceleration is required.
6. Decided action is initiated.
7. Vehicle speeds up or slows down to avoid collision.

This is a relatively simple model of the driver's action and does not consider any overlapping of the discrete steps. Nonetheless, by assigning time values to each of the steps and then summing, at least a reasonable upper value can be established.

Table 11 provides the time values for the elements of the perception reaction time as suggested by the literature. The latency and eye movement times are from the work of White et al. (58) which investigated latencies of eye reaction to stimuli located 10, 20, and 40 degrees off the visual axis. In this case, the values for the stimulus 40 degrees off center are appropriate since the vehicle in the adjacent leg is likely to be first sighted in the far periphery. It should be noted that if both vehicles are traveling at the same speed, the sight angle is 45 degrees off-center; if the vehicles are traveling at different speeds, the sight angle for the driver of the slower vehicle is greater than 45 degrees. However, the driver is within sight distance of the intersection and thus is alerted of potential conflicting vehicles and is already scanning the conflicting approach legs. Therefore, the latency/eye movement value for objects 40 degrees off-center would seem to be a reasonable estimate.

Fixation and recognition time estimates can be assumed to be the same as for the stopping sight distance situation.

The most pertinent data available from the literature relative to the decision making process is that of Lunenfeld (63) which states that 85th percentile driver decision times would be as follows for both expected and unexpected situations:

Information Content (Bits)	Decision Time (sec) Unexpected Situation
0	0
1	1.0
2	1.6
3	2.6

Using the definitions of the differing levels of information content presented by Lunenfeld, for the purposes of intersection perception reaction time characteristic the decision time is likely to fall between 1.0 second and a maximum of 1.6 seconds.

The last component is brake reaction. The values suggested are those from the Johansson and Rumar (54) study under an alerted condition. In this case the alerted condition values are justified since it is assumed that the driver will already be in that condition because of

TABLE 11 --Perception-Reaction Time for Various Percentiles of Driving Population Related to Cases I & II, Intersection Sight Distance, (Sec.)

	Percentile of Drivers					
Elements	50	75	85	90	95	99
Latency	0.28	0.33	0.34	0.36	0.39	0.45
Eye Movement	0.12	0.12	0.12	0.12	0.12	0.12
Fixation	0.20	0.20	0.20	0.20	0.20	0.20
Recognition	0.40	0.45	0.50	0.55	0.60	0.65
Brake Action	0.63	0.82	0.92	1.05	1.21	1.60
TOTAL	2.60	3.20	3.40	3.70	4.00	4.60

the awareness of the intersection and visual detection of the vehicle.

The totals (as shown in Table 11) for the estimated percentile values range from 2.6 seconds for the 50th percentile to 4.6 for the 99th percentile.

Intersection Sight Distance--Case III, IV, and V

Case III, IV, and V intersection sight distance enables the driver of a stopped vehicle on the minor leg of an intersection to see enough of the major highway to either cross the highway or merge into the traffic stream safely.

The driver characteristic perception-reaction time is defined by AASHTO (5) as "the time necessary for the vehicle's operator to look in both directions on the roadway, to perceive that there is sufficient time to cross the road safely, and to shift gears, if necessary, preparatory to starting. It is the time from the driver's first look for possible oncoming traffic to the instant the car begins to move."

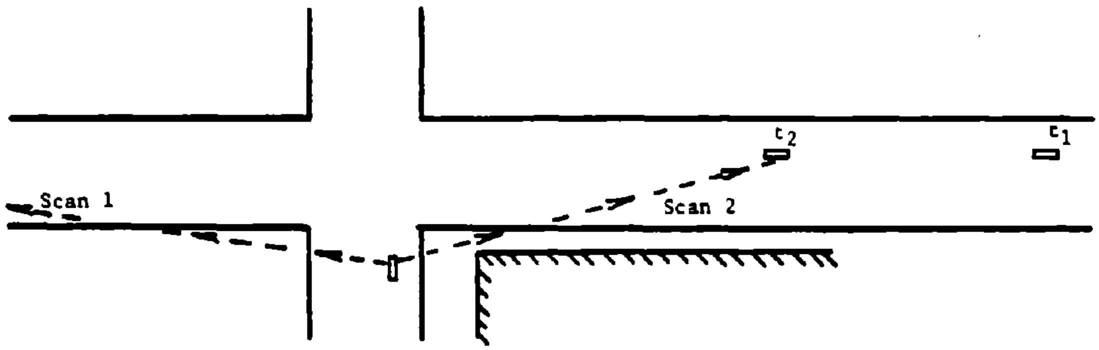
In order to develop an estimated distribution of driver characteristic values for the driving population, it is necessary to divide the driver characteristic into a series of steps. Basically the steps are:

(1) head/eye movement to scan intersection,

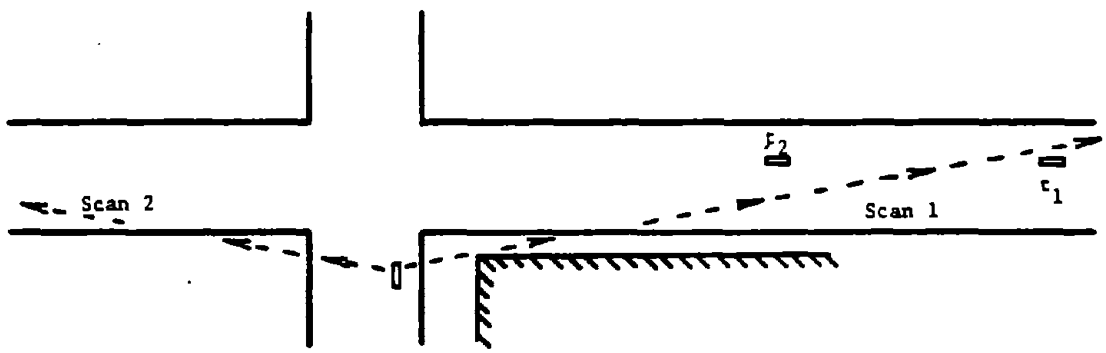
(2) fixation/decision, and

(3) reaction (i.e., move foot from brake to accelerator).

The AASHTO definition of the driver characteristic includes time for the driver "to look in both directions on the roadway." A careful critique of the driver's actual required scan movements reveals that only one head movement (not two as is called for in the definition) is needed in order for the driver to safely cross the intersection. No matter how many times a driver may scan the two intersection approach legs, the critical head/eye movements on which the decision to proceed or stay is made are the last two. If the driver scans one direction (as illustrated in Figure 5) and sees no approaching vehicle, the decision would be to proceed if a scan of the other direction reveals no approaching vehicle either. In scenario 2 of Figure 5, the driver has performed the two scans and has decided to proceed (even though another scan to the right would reveal



SCENARIO 1: The first scan of the driver of the stopped vehicle is to the left; no approaching vehicle is sighted. Driver moves head and scans to the right where an approaching vehicle is sighted. Driver decides to not attempt to cross. Approaching vehicle had been at the position denoted t_1 when the driver was first scanning to the left.



SCENARIO 2: The first scan of the driver of the stopped vehicle is to the right; the approaching vehicle at t_1 cannot be seen due to the sight obstruction. Driver then scans to the left where no approaching vehicle is sighted. Driver decides to cross intersection because no vehicles had been sighted in either direction. At that point in time, the vehicle approaching from the right is at position t_2.

FIGURE 5--Case III Intersection Sight Distance -- Driver of Stopped Vehicle Scanning Approaching Roadways

an approaching vehicle). The distance which the approaching vehicle travels before the stopped vehicle starts across the intersection is the former vehicle's velocity multiplied by the sum of the time it takes for the driver of the stopped vehicle to move his/her head/eyes to the left, to decide to proceed, and to move his/her foot to the accelerator. Thus, the head/eye movement component of the driver characteristic perception-reaction time should account for a scan of only one leg of the intersection.

No empirical research has successfully measured the total perception-reaction time for drivers of stopped vehicles for the driving population, it is necessary to assign values to individual elements of perception-reaction time and sum them.

Robinson (66) timed driver head movements at an intersection and found that an average scan to one direction (head movement plus fixation plus decision) took 1.1 seconds. Johansson (67) measured brake reaction time for drivers in an alerted condition. Brake reaction time is appropriate in this application because accelerator reaction is simple a motor movement equal and opposite to brake reaction; the driver is in an alerted condition due to the intersection scan. Johansson found values ranging from 0.63 seconds at the 50th percentile to 1.21 seconds at the 95th percentile. Interestingly, if the 85th percentile value of 0.92 seconds is added to Robinson's 1.1 second described above, a total perception-reaction time of 2.02 seconds results (only 0.02 seconds higher than the current specification).

The 1965 edition of the Traffic Engineering Handbook (91) provides the following data as the total time required for a driver to scan one leg of an intersection:

Shift (head/eye movement)	0.15 - 0.33 sec
Fixate on object	0.10 - 0.30 sec
TOTAL	0.25 - 0.63 sec

The time needed for the driver to decide to proceed can be estimated in the same manner as was done earlier under Stopping Sight Distance. The estimated values range from 0.50 seconds at the 50th percentile to 0.95 seconds at the 95th percentile. By summing this decision time and the head/eye movement time with Johansson's reaction time, another range of estimated values for perception-reaction time results--1.38 to 2.79 seconds. Note that the average of these two values is 2.08 seconds. If the mid-range values for head/eye movement are estimated to be the 85th percentile values, the 85th percentile value for the total perception-reaction time would be:

0.24 sec	head/eye movement
0.20 sec	fixation
0.85 sec	decision
0.92 sec	reaction
2.21 sec	TOTAL--estimated 85th percentile value

Railroad-Highway Grade Crossing Sight Distance--Case I

Case I Railroad-Highway Grade Crossing Sight Distance involves a crossing which has no control other than the standard cross buck sign. The driver must first detect and recognize that a railroad crossing is ahead, and that there is no active control, then search left and right for an approaching train and, if one is visible, then must decide whether to stop or maintain speed depending upon the relative speed and position of his/her vehicle and the train. This being the case, the perception-reaction should be longer than for just stopping sight distance.

The literature review did not identify any studies that developed data applying directly to the perception-reaction time for the railroad grade crossing situation. In the absence of any empirical data, reasonable estimates of the perception-reaction time can be made by considering the individual elements of the process.

Table 12 shows the time values for the individual elements of the perception-reaction time as suggested by the literature. The components include: eye fixation on the target (i.e., the railroad grade crossing itself or the crossbuck sign), recognition time (to recognize that the crossing has no active control), time to search left and right for on-coming trains, decision time (to decide whether or not to stop), and finally brake reaction time (activate the brakes if a stop is selected). The individual values cam from material presented earlier.

The values range from a low of 2.3 seconds for the stimated 50th percentile value to a high of 4.2 seconds for the 95th percentile. The 85th percentile value is estimated to be 3.5 seconds, one second longer than the 2.5 second specification.

TABLE 12--Estimated Perception-Reaction Times for Various Percentiles of Driving Population Related to Railroad-Highway Crossings

Element	Times (seconds) for Percentiles of:		
	50	85	95
1. Perception of RR X ing			
a. Eye Fixation 1/	0.20	0.20	0.20
b. Recognition 1/	0.40	0.50	0.6
2. Search Time for Train 2/	0.50	1.00	1.26
3. Decision 1/	0.50	0.85	0.95
4. Brake Reaction 3/	0.63	0.92	1.21
TOTALS (Rounded)	2.3	3.5	4.2

1/from Table 10
2/from Reference (16)
3/from Reference (54)

It should be emphasized that these are estimated times, analytically derived by adding estimated times for discrete elements of a human information detection-processing-reaction process. Findings from empirical studies might find them to be conservative or liberal. However, they at least provide a range to conduct the sensitivity analysis.

Railroad-Highway Grade Crossing Sight Distance--Case II

Case II Railroad-Highway Grade Crossing involves a driver stopped at a crossing stop line having sufficient sight distance to proceed safely through the grade crossing.

The literature search did not identify any studies which determined the time taken by motorist in searching for trains while at a stop. In absence of such data values can be derived, as was done with other characteristics, by considering the components of the process.

The 1965 edition of the Traffic Engineering Handbook (91) states that the time required for a driver to scan an intersection from left to right is as follows:

shift to right	0.15 - 0.33 sec
fixate on right	0.10 - 0.30 sec
Total scan time	0.25 - 0.63 sec

No indication is given as to how what percentile of drivers these ranges of values might apply. Added to these components should be a time for decision and then reaction. The decision component is not very complex, merely to confirm whether or not a train is there. The 0.5 second at the 50th percentile and the 0.85 second at the 85th percentile suggested for stopping sight distance would be appropriate to this situation. The reaction is to move one's foot from the brake to the accelerator. Alerted reaction times as measured by Johansson (54) are applicable to this situation. The above values can be summed to arrive at a range of total perception-reaction time, as follows:

Scan/Fixate Time	0.25 - 0.63 sec
Decision	0.50 - 0.85 sec
Reaction	0.63 - 0.92 sec
Total	1.38 - 2.40 sec
Rounded	1.4 - 2.4

It can not be stated for certainty what percentile of drivers these values apply, but it would be reasonable for this analysis to assume 1.4 seconds is the 50th percentile and 2.4 seconds the 85th percentile.

Passing Sight Distance

The driver characteristic perception-reaction time in the AASHTO standard for passing sight distance is actually overlapped with the acceleration/maneuver time prior to encroachment on the passing lane. That is to say that while the driver is initiating a passing maneuver, the driver is both perceiving/reacting and accelerating/maneuvering. In combination the driver and vehicle characteristics form a performance characteristic--initial maneuver time. Field observations documented by Prisk (6) and Normann (68) prior to 1960 indicate that the total time for the initial maneuver falls within the 3.6 to 4.5 second range.

In every passing maneuver the passing driver reaches a critical position which if passed requires the passing driver to complete the passing maneuver (rather than decelerate and pull behind the otherwise passed vehicle) in order to avoid an opposing vehicle.

Since it is at this position that the passing driver must decide to complete or abort the maneuver, it, therefore, defines the location at which an opposing vehicle that would conflict with the completion maneuver must be able to be perceived. Basically, the driver has two options when an opposing vehicle appears at the instant the passing vehicle reaches the critical position: to complete the passing manuever or to decelerate and pull back behind the vehicle that was to be passed. During the time that the driver of the passing vehicle is perceiving and initiating reaction to an opposing vehicle, the passing vehicle continues to travel at the passing speed. Therefore, if the driver decides to complete the passing maneuver, the perception-reaction time does not contribute additional time (or distance) to the total time (or distance) needed to complete the passing maneuver. However, if the passing driver decides to abort the passing maneuver, the perception-reaction time does add to the total time and distance needed for the "pass abort" maneuver.

It can be assumed that a driver in the "critical position" will require no time for perception or decision if an opposing vehicle appears. With regard to reaction, the brake reaction times observed by Johansson and Rumar (54) for drivers in an alerted condition would seem the most appropriate. They range from a median value of 0.63 seconds to an 85th percentile of 0.92 seconds to a 95th percentile of 1.21 seconds.

Vehicle Change/Interval for Traffic Signals

A 1934 MIT research effort (19) found that 95 percent of the sampled drivers had brake reaction times of one second or less when in an alerted condition. Subsequent studies in the early 1960's of driver reactions to the amber signal by Gazis, Herman, and Maradudin (69) and by Olson and Rothery (70) continued the use of the 1.0 second specification. However, a recent field study by Wortman and Matthias (71) observed driver perception-reaction times that were significantly greater than the specification values. At all sites in the study, the mean observed perception-reaction time was greater than 1.0 second. In fact at most of the intersections the 85th percentile value approached two seconds.

Experimentally-derived data on driver perception-reaction time was discussed in detail earlier under Stopping Sight Distance. Applicable to the yellow signal case is the data presented by Johansson and Rumar (54) on brake reaction times for "alerted" subjects. The stimulus in the Johansson and Rumar study was auditory (not visual) and thus could be expected to require little if any perception time. Likewise, because the subjects were instructed to perform a particular task upon hearing the stimulus, no appreciable decision time would be expected. The Johansson and Rumar distribution is as follows:

	"Alerted" Brake Reaction Time, sec
50th Percentile	0.63
85th Percentile	0.92
95th Percentile	1.21

It should be noted, however, that this distribution depicts a pure case of brake reaction. That is, the decision is made instantaneously upon perceiving the circumstances (no decision time) and perception of the situation occurs simultaneously with the onset of the situation (i.e., no perception time). Obviously, no driver is able to decide to brake the vehicle at the same instant the yellow indication starts. Rather, the driver must first detect and/or identify that the signal indication has turned yellow and decide whether to continue through the intersection or to stop prior to the intersection. Detection/identification of a signal phase change depends greatly on the amount and criticality of other information that must be processed. Some other factors that compete for a driver's attention include traffic conditions, approach speed, directional uncertainty, and proximity to the intersection. (Note: King provides a thorough discussion of techniques to minimize these distractions in Guidelines for Uniformity in Traffic Signal Design Configurations (20)). With these distractions and other potential temporary blockages (e.g., trucks), it is quite conceivable that a driver will not instantaneously detect a signal phase change.

After the signal phase change is detected, the driver must still decide whether to continue through the intersection or to stop. This decision is more complex than the one facing a driver who sights an impassable object lying in the road. And the "signal phase change" decision is less complex than that faced by a driver approaching an uncontrolled intersection who must judge relative speeds of potentially conflicting vehicles. For the decision under discussion here, the driver must simply decide if it is possible to stop based on the speed of the vehicle, distance from the intersection, roadway conditions (e.gs., wet vs dry; level vs grade), and traffic conditions (e.g., is the vehicle ahead going to stop?). The same decision time distribution presented earlier under Stopping Sight Distance can be assumed as follows:

	Decision Time, sec
50th Percentile	0.50
85th Percentile	0.85
95th Percentile	0.95

If latency, fixation and recognition times are assumed to be zero (i.e., instantaneous recognition of the amber signal phase change), the decision-brake reaction time estimates become the perception-brake reaction time estimates as follows:

	Perception-Brake Reaction Time, sec
50th Percentile	1.13
85th Percentile	1.77
95th Percentile	2.16

These empirically-derived values compare quite favorably with the observed values documented by Wortman and Matthias (71). For example, the mean value observed by Wortman was 1.30 seconds as compared to the 1.13 seconds median value derived above. Wortman's average 85th percentile value for all study intersections was 1.8 seconds; the estimate above is 1.77 seconds.

It should be noted that the estimates derived above assume the driver's instantaneous perception and recognition of the amber signal phase change. In some, if not most, cases the driver will indeed give primary attention to the signal when the vehicle approaches and passes through the point at which the appropriate decision changes from "stop if signal changes to yellow" to "proceed even if signal changes to yellow". In other words, the experienced driver is aware of this threshold point and knows that it is not critical to focus attention on the signal well before or well after this point but that it is critical around that distance from the intersection. In some cases the driver may not be able to focus attention on the signal when the vehicle is near the threshold point due to other factors such as traffic congestion. In these instances, the driver does not instantaneously perceive and recognize the amber signal phase change. This lag time actually would be added to the perception-brake reaction time estimates derived above.

Adequate Gap Time for School Crossing Traffic Signal Warrant

The pedestrian perception-reaction time is defined in the ITE recommended practice, A Program for School Crossing Protection (18), as "the number of seconds required for a child to look both ways, make a decision, and commence to walk across the street". It is assumed to include attention/fixation time, perception time, and response time for the decision making process on whether or not to cross a street. Abrams and Smith (72) observed five different intersections for a total of 17 hours. They observed a range of average perception-reaction times from 0.61 to 2.53 seconds; the observed mean and median values were 1.31 and 1.18 seconds, respectively. It should be noted that the estimated 85th percentile value for driver perception-reaction time while stopped at an unsignalized intersection was estimated as 2.21 seconds, roughly comparable to the Abrams and Smith data. Several words of caution about these estimates, however, are necessary. First, the Abrams and Smith study examined pedestrian perception-reaction time at signalized intersections, and that was perception of and reaction to pedestrian WALK/DON'T WALK signals. The task facing a pedestrian at an unsignalized intersection is significantly more complex.

The second caution is that the data presented above (both the field data and the laboratory data used for intersection estimates) is strictly based on measurements of the adult population. Without any data to support this claim, it is nevertheless a reasonable assumption that a "safe" value for a child's perception-reaction time should exceed that for an adult. Therefore, the current specification value of 3.0 seconds appears to be a realistic estimate.

INFORMATION PROCESSING CAPACITY

In most driving task models (e.g., Forbes (73), Shinar (74)) the driver is conceived as a processor of information received through his sensing skills. The driver detects various inputs through his sensing skills, organizes and interprets these as recognizable objects, events, situations, etc., weighs alternative actions and then makes a decision leading to a vehicle response, if necessary. The process of sensing-perception-decision-making can be considered collectively as information processing and it is generally held that there is a capacity limitation for each driver.

Dewar (75) talks of channel capacity as the upper limit of a person's ability to process information coming into the nervous system. King and Lunenfeld (76) in their discussion of channel capacity cite Miller's (77) definition as the asymptotic value where an increase in the input of quantity of information yields no increase in the transmitted quantity of error-free information.

Since the driver is continuously processing information during this travel; it is a driver characteristic inherent in many aspects of highway design and traffic operations. However, it has particular relevance in the design of sign information systems since the motorist has a limit as to how much sign copy he can read, comprehend, and make decisions on. This driver characteristic is acknowledged, at least indirectly, in the standards for design of guide sign legend.

The MUTCD in Section 2E-9 specifies that directional copy on a guide sign should not exceed three lines and that there should not be more than three signs each with one destination on one support. The MUTCD explicitly states that "...three destinations and the directional copy are as much as most drivers can comprehend readily at high speed."

The support for this maximum of three pieces of information seems to be based on the early work of Mitchell and Forbes who wrote in 1942 (37) regarding the design of letter sizes:

"Minimum Reading Time and Safety Factor.--The shortest possible glance from the road to read the sign and back to the road consumes from 0.6 to 1.0 seconds. During this glance the maximum amount of copy which can be read by the ordinary person is from three to four familiar words. This time value results from the fact that it takes approximately 0.2 seconds for the eye to stop, focus and read, and another 0.2 seconds for the eye to start and move through one of the 5 degree of 10 degree jumps. To be conservative, therefore, 1.0 seconds is adopted as the time necessary for a single minimum glance from road to sign and back to the road, allowing the shortest possible glance at both the road and the sign.

If more than three familiar words are included in the copy, it has been shown that the time for reading the sign may be increased to as much as from 3 to 11 seconds, and such signs, therefore, are impracticable on high-speed highways. Where they are unavoidable the reading time should be increased by one second for each additional three or four familiar words, thus making allowance for the driver to glance back to the road between glances at the sign.

It is still necessary to add a time interval as a safety factor in case the sign is not seen at once. The smallest possible safety factor is one 1-second glance; that is, the minimum reading time is 2 t_g, in which t_g is the time required for a single glance or 1.0 second and when signs contain the minimum number of words. When the sign contains more than three words:

$$t_g = \frac{N}{3} + 1.0$$

in which N equals the number of familiar words on the sign. If the attention or target value has been properly designed into the sign, the motorist will have seen it before he is able to read it, and Eq. 3, with 2 seconds as an absolute minimum will guarantee him time to read the sign twice unless something distracts him or obstructs his vision."

This limit of three to four familiar words appears to have been accepted and included in the current design standards. While there has been considerable research in other aspects of sign design (e.g., reading time, legibility distances, etc.), the literature review did not identify any other research that provided data on maximum amount of information that can be included on a sign(s) based on the limitations of driver information processing capacity.

PEDESTRIAN WALKING SPEED

Pedestrian walking speed in roadway crossing applications can be defined as the distance travelled by the pedestrian (e.g., curb-to-curb) divided by the time elapsed between the decision to proceed across the roadway and the arrival at the destination point (e.g., far curb).

Components of Characteristic

The process which a pedestrian undergoes in crossing a roadway can be divided into two distinct components: sidewalk queue time and walking time. Once a pedestrian has perceived that it is safe and legal to cross the street, the pedestrian must move from the original observation point to the street itself. In most cases, the pedestrian has stood at the curb of the street prior to crossing the street and the time taken to leave the curb is negligible. However, in instances where large pedestrian volumes are present, pedestrians will form a queue at the curb and thus the pedestrian must also traverse part of the sidewalk prior to leaving the curb. This amount of time taken for this maneuver is a function of both the number of pedestrians present (and thus the density) and the in-motion walking speed of pedestrians in advance of the other pedestrians. Means of accommodating this queuing problem are cited in a report by Abrams and Smith (72).

The second component of a pedestrian street crossing is the actual walking time between the curb and the destination point. Several studies have been conducted to measure this walking speed. As Institute of Transportation Engineers Technical Committee report (78) estimated a mean walking speed for street crossing to be 3.7 fps (1.13 m/s). A study by Hoel (79) estimated a mean speed of 4.72 fps (1.44 m/s). In

1968 Weiner (80) estimated a mean speed of 4.22 fps (1.29m/s) and a 1967 Swedish study by Sjostedt (81) estimated a mean speed of 4.5 fps (1.4m/s). The wide disparity between these four estimated mean values is indicative of the complexity of the walking speed component Walking speed is a function of a variety of factors including pedestrian density, age or other impairment of the pedestrian, sex of the pedestrian, street width, and trip purpose.

Factors Affecting Walking Speed

Pedestrian Density--In general, walking speeds decrease as pedestrian density increases. Hoel estimated 10 percent of individual, unconstrained pedestrain crossings to have walking speeds less than 4.0 fps (1.2 m/s). Abrams and Smith took Hoel's relationship and factored into it the constraint of increased pedestrian volumes to develop a direct mathematical formula which they then validated with field counts. This formula estimates that an intersection with a 2-way pedestrian volume of 10 per cycle will result in 33 percent of the signal cycles having pedestrian platoons with walking speeds less than 4.0 fps (1.2 m/s). For 20 pedestrians per cycle, the percentage rises to 57; and for 30 pedestrians per cycle, the percentage exceeds 80. Weiner made the observation that even in an unconstrained condition, walking speeds tend to decrease for pedestrians accompanying other pedestrians. The Weiner research indicates that average male walking speeds drop from 4.22 fps (1.29 m/s) when walking alone to 3.83 fps (1.17 m/s) when not alone, for females the drop is from 3.70 fps to 3.63 fps (1.13 m/s to 1.11 m/s).

Age of Pedestrian--The distribution of a pedestrian platoon's walking speeds fluctuate as a function of the platoon's age distribution. Sleight (82) developed a cumulative walking speed distribution for three distinct age groups: the elderly, adults, and children. Sleight found that in unconstrained movement, the average adult and the elderly crossed the street at approximately 4.5 fps (1.4 m/s) and that the average child crossed at 5.3 fps (1.6 m/s).

Sex of Pedestrian--ITE Committee 5-R reported an average street crossing speed for males at 3.8 fps (1.2 m/s) and for females at 3.3 fps (1.0 m/s). As cited above, Weiner reported average speeds of 4.22 fps (1.29 m/s) for males and 3.70 fps (1.13 m/s) for females. Each report estimates that female walking speeds are 12-13 percent less than for males.

Street Width--Street width affects the walking speed of a pedestrian. Bruce (83) observed that walking speeds will vary as a function of the street width (i.e., as street width increased, walking speed likewise increases). The Bruce data counted a range of average speeds from 4.8 fps (1.5 m/s) for a 132 foot (40/m) street to 4.1 fps (1.2 m/s) for a 60 foot (18 m) street to 3.6 fps (1.1 m/s) for a 42 foot (13 m) street.

Trip Purpose--Pushkarev (84) assembled data in the book Urban Space for Pedestrians which describes the effect of trip purpose on walking speeds. For instance, walking speeds for commuters are typically greater than for any other type of pedestrian. Students and shoppers generally walk slightly slower (except at very low densities, when student walking speeds increase significantly).

The distribution of walking speeds found at any intersection is tied directly to each of the above five factors--pedestrian density, age or other physical impairments of the pedestrians, sex distribution of the pedestrians, street width, and the trip purposes of the pedestrians.

Intersection walking speed distributions are completely site-dependent. Perhaps the only way to adequately portray these distributions is to present ranges of values for the 50th, 85th, and 95th percentiles. These ranges will be developed from three intersection scenarios: heavy elderly concentration with no pedestrian density constraints; "average" CBD distribution by age group and no pedestrian density constraints; and "average" distribution constrained by heavy pedestrian volumes.

- A high concentration of elderly pedestrians, according to Sleight (82) would yield a walking speed distribution as follows:

Percentile	Speed fps	Speed m/s
50	4.5	1.22
85	3.4	1.04
95	3.0	0.91

- Based on statistics reported by Bruce (83), an "average" CBD distribution of walking speeds uncontrained by pedestrian densities would be as follows:

Percentile	Speed fps	Speed m/s
50	4.2	1.28
85	3.6	1.10
95	3.1	0.94

- Based on Abrams and Smith (72) field studies dealing with congested intersections, the following distribution appears to be realistic:

Percentile	Speed fps	Speed m/s
50	3.9	1.19
85	3.4	1.04
95	3.0	0.91

Thus, in general, the distribution of walking speeds at intersections falls within the following ranges:

50th percentile is 3.9-4.5 fps (1.2-1.4 m/s)

85th percentile is 3.4-3.6 fps (1.0-1.1 m/s)

95th percentile is 3.0-3.1 fps (0.91-0.14 m/s).

IV. SENSITIVITY ANALYSIS AND CRITIQUE OF SPECIFICATION

Having discussed the relevant geometric design and traffic operations standards, and identified the current specifications and range of values for the relevant driver and pedestrian characteristics, this chapter will:

- present the results of the sensitivity analysis which identifies those standards that are sensitive to a change in the characteristic specification;
- indicate, where possible, what percent of the driving or pedestrian population is being excluded by the current specification and what might be the impact of this exclusion; and
- critique the current specification in light of what is now known about the characteristic and how it affects the standard.

The sensitivity analyses which follow employ the basic approach of determining the change in the standard value which corresponds to a specified change in the driver characteristic value. In its simplest form, the sensitivity is the ratio,

$$\frac{\text{Unit Change in Standard}}{\text{Unit Change in Driver Characteristic}}.$$

In certain circumstances, a better understanding of the relationship between the standard and the driver characteristic can be presented by calculating the ratio,

$$\frac{\text{Percent Change in Standard}}{\text{Unit Change in Driver Characteristic}}$$

or

$$\frac{\text{Percent Change in Standard}}{\text{Percent Change in Driver Characteristic}}.$$

In all of the sensitivity analyses, the base from which these sensitivity analyses are calculated is the current specification value for the driver characteristic. A unit change from the specification is chosen (e.g., 1 second) and the resultant driver characteristic value is used to calculate a revised standard value, or in equation form the sensitivity is

$$\frac{\text{Standard}_2 - \text{Standard}_1}{\text{Driver Charac}_2 - \text{Driver Charac}_1}$$

This computation yields a value for the incremental sensitivity of the standard to the driver characteristic. For some standards, those involving exponential or trigonometric functions in the standard formulation, the sensitivity changes as the driver characteristic values change even if the unit change in the driver characteristic is held constant. In other words, the instantaneous sensitivity rate is a function of the driver characteristic value and does not equal the incremental rate. The instantaneous rate is computed by taking the partial derivative of the standard formulation with respect to the driver characteristic.

For several standards, the population distribution for the driver characteristic remains somewhat in doubt, as documented in Chapter III. For these standards, the sensitivity analysis involves varying the standard values by certain percentages (e.g., 5, 10, 20 percent) and computing the required change in the driver characteristic to accommodate these variations. In that way, the analysis shows if a significant change in the standard requires only a small change in the driver characteristic. Or conversely, if a significant change in the driver characteristic yields only a small change in the standard.

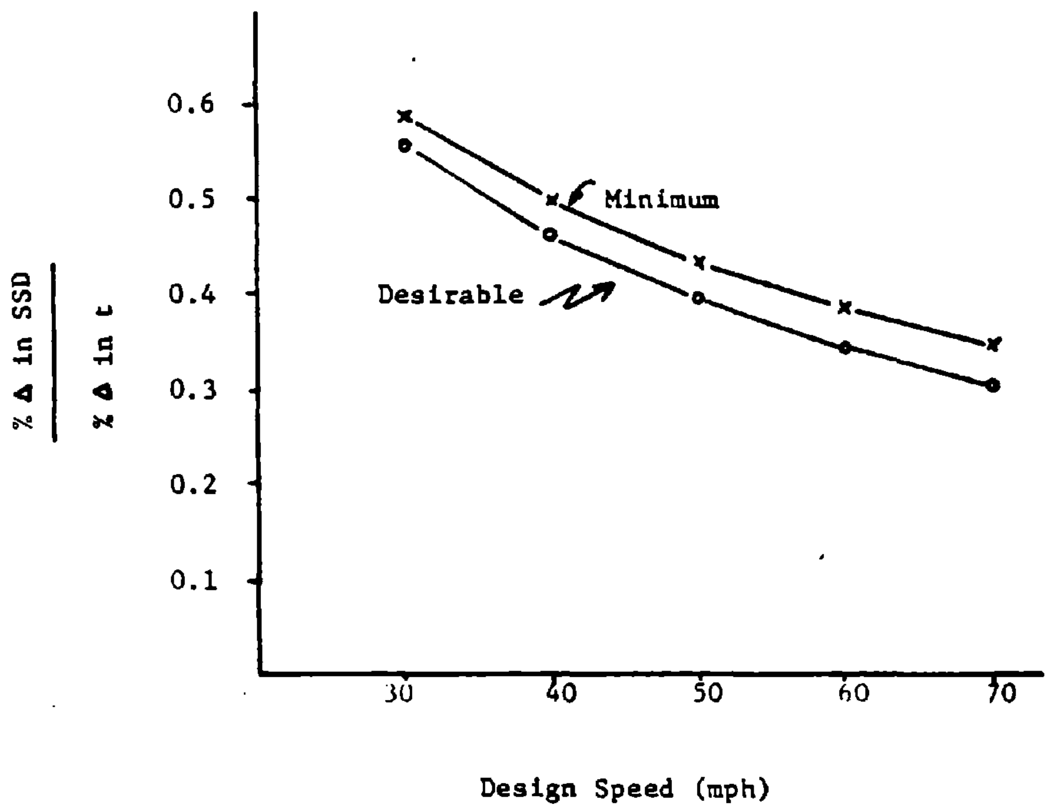
STOPPING SIGHT DISTANCE

Sensitivity Analysis

Stopping sight distance is determined from equation (1). The instantaneous and incremental change in stopping sight distance (SSD) with respect to an instantaneous (or incremental) percentage change in the driver characteristic, perception-brake reaction time (P), can be expressed as follows:

$$\frac{\frac{\partial(SSD)}{SSD}}{\frac{\partial(P)}{P}} = \frac{1.47\ PV}{1.47\ PV + \frac{V^2}{30(f+g)}} \qquad (26)$$

Application of equation (26) for both minimum and desirable stopping sigh distances yields the "sensitivity indices" which are plotted in Figure 6. At the design speed of 30 mph (48 km/h) where there is no grade, a one percent change in the perception-brake reaction time will yield a 0.580 percent change and a 0.563 percent change in the minimum and desirable stopping sight distances, respectively. This percent change decreases with increasing design speed. Another observation is that the minimum values are more sensitive to a change in



The SI (metric) conversion is 1 mph = 1.61 km/h

FIGURE 6 --Sensitivity Indices Related to Design Speeds for Minimum and Desirable Stopping Sight Distance

the perception-brake reaction time than are the desirable values because of the shorter braking distances associated with the minimum values.

Longer stopping sight distances are required on downgrades (because of the longer braking distances); therefore, an increase in the perception-brake reaction time would have less effect than on level grade. Conversely, for upgrades, less stopping sight distance is required and, therefore, the perception-brake reaction component has more affect on the overall stopping sight distance. Consequently, stopping sight distance is more sensitive to an increase in perception-brake reaction time for upgrades.

Referring back to Table 10, several values of perception-brake reaction time were suggested based on the literature synthesis. The values of 2.3, 2.8 and 3.2 seconds, which are the 85th percentile values for each of the three different assumptions, will be used for the sensitivity analysis. These values were selected because they at least provide a reasonable range of values that bracket the 2.5 second specification.

Table 13 displays the new minimum and desirable stopping sight distances and the percent changes from the current calculated stopping sight distances for the three values of perception-brake reaction time.

If 3.2 seconds was to be used as the specification value for perception-brake reaction time then the calculated minimum stopping sight distance would be nearly 16 percent higher than the current value at 30 mph (48 km/h) and nearly 10 percent higher at 70 mph (113 km/h). The 3.2 seconds represents the estimated 85th percentile value of driver perception-brake reaction time under the worst case assumption, i.e., the driver is not fixating down the travel lane and requires some decision time before activating the brake.

The question as to whether or not stopping sight-distance is sensitive to a change in the specification for the perception-brake reaction time can not be answered simply. About all that can be said is that a change in the perception-brake reaction time results in a measurable change in the stopping sight distance becoming less significant as design speeds increase. This finding raises the question as to whether or not perception-brake reaction time varies with speed. The brake reaction time probably doesn't, but it is possible that the perception-decision component might vary with speed. Hypotheses could be rationalized for either increasing or decreasing time with changes in speed. The literature does not provide much evidence for either case.

Exclusion Analysis

The current specification for perception-brake reaction time is 2.5 seconds. It apparently does not represent any specific percentile value of the driving population but was considered by AASHTO (1) to be "large enough to include the time taken by nearly all drivers under most highway conditions". Based on the available literature on perception and brake reaction time, it could be argued that the 2.5 seconds may not be adequate to compensate for "nearly all drivers". As shown previously in Table 10, a conservative assumption, the driver is looking down the travel lane still yields an estimated 85th percentile value of 2.8 seconds for perception-brake reaction time. The current specification of 2.5 seconds would accommodate an estimated 75 percent of the driving population under this assumption.

All of the percentile values in Table 10 are to be considered estimates. They were determined by adding values, in some instances estimates themselves, of discrete components of the perception-brake reaction time. It can not be stated with certainty that these values do represent the true distribution of the driving public, as they were based on relatively small sample sizes and less than actual driving conditions.

Perception-brake reaction varies among the driving population and varies for any one driver depending upon the situation. Consequently it is difficult to state categorically that a certain percentage of drivers are being excluded with the 2.5 second specification. It can be safely assumed that the current specification does not accommodate all drivers for all situations. Our estimate is that about 25 percent of the drivers may be excluded at some time by adoption of the 2.5 second specification. It can not be stated what group of drivers are being excluded but the elderly, with their slower reaction times, are likely predominant in the excluded group. Whether or not the excluded group compensates in any way by reducing travel speed is unknown and should not be assumed without evidence.

TABLE 13--Stopping Sight Distances and Percent Change from Current Computed Distances for Three Values of Brake Reaction Time

Design Speed mph	Assumed Speed mph	f	Perception-Brake Reaction Time (sec) 2.3 min	2.3 des	2.8 min	2.8 des	3.2 min	3.2 des
30	28	0.35	169 / -5.1	187 / -4.5	190 / 6.7	209 / 6.6	206 / 15.7	227 / 15.8
40	36	0.32	257 / -3.7	302 / -3.8	283 / 6.0	331 / 5.4	304 / 13.9	355 / 13.0
50	44	0.30	364 / -3.2	447 / -3.0	396 / 5.3	484 / 5.0	422 / 12.2	513 / 11.3
60	52	0.29	487 / -3.0	617 / -2.7	525 / 4.6	661 / 4.3	555 / 10.6	696 / 9.8
70	58	0.28	596 / -2.8	820 / -2.4	639 / 4.2	871 / 3.7	673 / 9.8	913 / 8.6

The SI (metric) conversions are 1 mph = 1.61 km/h and 1 ft = 0.3 m.

Upper value: Stopping sight distance (ft)

Lower value: Percent change from current computed SSD.

Equally undeterminable is what effect takes place by not designing for those motorists with perception-brake reaction time longer than 2.5 seconds. Logic would lead to the conclusion that those excluded motorists would be more susceptible to accidents involving stopping situations and limited sight distance. A longer time to perceive and react to a stopping situation leaves a shorter distance for braking.

Unfortunately not very much is known regarding the relationship of sight distance and accidents except that, in general, the greater the minimum sight distance the safer the highway becomes. A 1975 study by Gupta and Jain (85) attempted to relate accidents to roadway width, horizontal curvature, vertical clearance and sight distance restrictions. They found that only 5 percent of the variation in accident rate was explained by these variables. Other similar studies have not been successful in isolating the relationship of sight distance to accidents. Without such a relationship it cannot be determined what the effect would be on safety if a longer minimum sight distance were adopted as a result of a higher driver perception-brake reaction time specification value.

Critique of Specification

The current 2.5 second specification for perception-brake reaction time has been used for stopping sight distance for many years. It has been argued that this value is too low by others (Glennon, (86)) and estimates of the characteristic developed for this study can support this claim. It is estimated that the 2.5 seconds may be applicable to only 75-85 percent of the driving population. On the other hand, there is no strong evidence that the current minimum and desirable stopping sight distances, which are based on 2.5 seconds, are inadequate or at least unacceptable on the ground of safety. As stated before, studies of accidents related to sight distance restrictions have not shown any strong correlations.

There would appear to be no justification for lowering the 2.5 second specification. Most of the estimates for perception-brake reaction time (see Table 11) are indeed higher than the current specification. Any lowering of the specification would place more drivers in situations where they would not have enough time to perceive and react to a stopping situation. There might be justification, however, for increasing the specification if the values estimated here are considered reliable. (It should be noted here that the concept of decision sight distance essentially does this by providing longer premaneuver times). However, before this is considered, further empirical research is needed to derive a more reliable distribution of perception-brake reaction time for the driving population. This research would require both laboratory and field studies on large samples of the driving public. The test procedures should be developed so that the reaction times for the unalerted driver could be measured.

PASSING SIGHT DISTANCE

Sensitivity Analysis

As noted previously, it is difficult to isolate the driver characteristic in the AASHTO standards for passing sight distance. Nevertheless, it might be useful to evaluate the sensitivity of the AASHTO design standards to changes in what is termed the reaction time distance, d_1 which is expressed by equation (2). The distance d_1 is added to distances d_2, d_3, and d_4 to arrive at the design sight distance value.

Table 14 lists the total computed sight distance values for varying values of initial maneuver time for two speed groups. For instance, for the 30-40 mph (48-64 km/h) speed group, a 50 percent increase in initial maneuver time over the current specification yields only a 7.9 percent increase in total computed passing sight distance. The effect of a 50 percent increase ranges between a 7.9 and 8.4 percent increase. These relatively low yields from substantial changes in the specification value would indicate that the AASHTO standard for passing sight distance is relatively insensitive to changes in the initial maneuver time (which, in turn, is indirectly a function of the driver characteristic, perception-reaction time).

Exclusion Analysis

The driver characteristic perception-reaction time is cited by AASHTO as being an element of the sight distance standard. However a driver at the beginning of the passing maneuver is not provided sufficient sight distance to be assured of a safely completed pass. Rather, the driver is not assured of a safe pass until the "critical position" is reached and no opposing vehicle is sighted. Therefore, variations in a driver's perception-reaction time will not affect the driver's ability

TABLE 14--Effect of Changes in Initial Maneuver Time Necessary for Passing Sight Distance

Percent Change in Initial Maneuver Time	COMPUTED SIGHT DISTANCE, ft (Percent Change in Sight Distance) SPEED GROUP 30-40 mph		SPEED GROUP 60-70 mph	
-50%	962 ft	(-7.3%)	2190 ft	(-7.9%)
-40	977	(-5.9%)	2227	(-6.3%)
-30	992	(-4.4%)	2264	(-4.8%)
-20	1007	(-3.0%)	2302	(-3.2%)
-10	1022	(-1.5%)	2340	(-1.6%)
- 5	1030	(-0.8%)	2360	(-0.8%)
0	1038	(0)	2378	(0)
5	1046	(0.8%)	2397	(0.8%)
10	1054	(1.5%)	2417	(1.6%)
20	1070	(3.1%)	2457	(3.3%)
30	1086	(4.6%)	2497	(5.0%)
40	1103	(6.3%)	2537	(6.7%)
50	1120	(7.9%)	2578	(8.4%)

Notes: 1) Values are computed based on the AASHTO formulation and specifications: and assumes that the "v-m" value equals 10 mph (16 kph).
2) Values shown for a zero percent change may be different from the AASHTO design standards due to rounding.
3) Percentages are computed from the calculated values.
4) The SI (metric) conversions are 1 mph - 1.61 km/h and 1 ft = 0.3 m.

to safely initiate and complete a passing maneuver. One possible exception to this statement must be mentioned--in the case of marking the beginning of a passing zone, a driver with a short perception-reaction time will be capable of initiating the "pass-or-no-pass" decision process earlier in the passing zone than the driver with a slower perception-reaction time. Nevertheless, once this decision process is initiated, the "slow" and "fast" driver are on equal footing. Therefore, no drivers are treated unequally by the current passing sight distance standards because of their particular driver characteristics.

At the "critical position", the driver has two options: to complete the passing maneuver or to decelerate and pull back behind the vehicle that was to be passed. During the time that the driver of the passing vehicle is perceiving and initiating reaction to an opposing vehicle, the passing vehicle might continue to travel at the passing speed. Therefore, if the driver decides to complete the passing maneuver, the perception-reaction time does not contribute additional time (or distance) to the total time (or distance) needed to complete the passing maneuver. However, if the passing driver decides to abort the passing maneuver, the perception-reaction time <u>does</u> add to the total time and distance needed for the "pass abort" maneuver. Therefore, drivers with slower perception-reaction time might be excluded by the current AASHTO standard from executing a passing maneuver in the prescribed, safe manner. Instead these drivers must either accelerate to pass the slower vehicle at a greater than 10 mph (16km/h) differential, reduce the clearance between the passing and passed vehicle or between the passing and opposing vehicle, or abort potential pass completions more often than drivers with lower perception-reaction times.

Critique of Specification

The current specification value for the initial maneuver time is based on field observations and so is generally representative of the driving population. The primary fault of the field data used to develop the current specification is that it may now be outdated. The most recent appropriate passing data was collected by Weaver and Glennon (87) and reported in 1971. The AASHTO standard is based on data documented by Prisk (6) and Norman (68) prior to 1960.

DECISION SIGHT DISTANCE

Sensitivity Analysis

Decision sight distance is expressed by equation (4). The instantaneous and incremental percentage change in decision sight distance with respect to an instantaneous (or incremental) percentage change in the driver characteristic, pre-maneuver time (which equals the sum of detection, recognition, decision, and response initiation time), is expressed as follows:

$$\frac{\frac{d(DSD)}{DSD}}{\frac{d(t_{dc})}{t_{dc}}} = \frac{t_{dc}}{t_{dc} + t_5} \qquad (27)$$

where: DSD = decision sight distance ft (m)

t_{dc} = pre-maneuver time, sec

t_1 = detection time, sec

t_2 = recognition time, sec

t_3 = decision time, sec

t_4 = response time, sec

t_5 = maneuver time, sec

Application of formula (27) results in the sensitivity indices as shown in Table 15. For each design speed two values are shown, a low value which is based on the low end of the recommended range of the pre-maneuver times and a high value which is based on the high end of the range.

TABLE 15--Sensitivity Indices for Decision Sight Distance with Respect to Change in Driver Characteristic Specification

Design Speed		% Δ in DSD With 1% Δ in t_{dc}	
mph	km/h	Low	High
30	48	0.56	0.68
40	64	0.56	0.68
50	80	0.56	0.68
60	97	0.60	0.69
70	113	0.63	0.72

The SI (metric) conversion is 1 mph = 1.61 km/h.

These index values indicate that decision sight distance is sensitive to a change in the collective detection-recognition-decision-response driver characteristic. If, for example, the driver characteristic specification of 6.7 seconds (the low value for design speed of 60 mph (113 km/h) was changed by ±10 percent (0.67 sec) the resulting decision sight distance value would change ±6.0 percent. For the high value at the same speed the change would be 6.9 percent.

Exclusion Analysis

The values used for the detection-recognition-decision-response driver characteristic do not relate to any specific percentile of the driving population. However, it is likely that they apply to at least the 85th or even higher percentile, given the way they were developed. Initially they were estimated by summing the mean or higher values for the discrete components of the process. This procedure likely results in a conservative value, since it assumes that there is no time sharing of the four tasks, which does occur. Subsequently they were "validated" by a field experiment using test subjects. While the sample of drivers was very small (n=19) it did include a full range of age. Consequently, it can be reasonably assumed that very few motorists are excluded by the values used for driver characteristic specification.

Critique of Specification

There appears to be no reason to modify the driver characteristic values used for decision sight distance, especially since the low-high ranges suggested result in a wide range of distances. Increasing the specifications would result in a overly conservative standard. Providing sight distances "more than enough" for the task does not necessarily make it a safer or more efficient design. It has been observed that motorists will oftentimes not take advantage of the extra sight distance. For example, they will change lanes or exit "at the last moment" even though they could see the need for it a long distance away.

What is suggested for this standard are better guidelines for the use of decision sight distance and specifically what values between the recommended limits are appropriate for various conditions. Since the low-high range is based on ranges of the driver characteristics, further research may be warranted to define these values more precisely.

INTERSECTION SIGHT DISTANCE - CASE I ENABLING VEHICLES TO ADJUST SPEED

Sensitivity Analysis

The necessary distance from an intersection (which a vehicle must be in order for the driver to safely sight an approaching vehicle) is a function of the velocity of the driver's vehicle and of the driver's perception-reaction time as expressed by equation (5). Thus for each incremental change in the design value for perception-reaction time, there is a corresponding incremental unit change in the necessary intersection sight distance. The formulation is as follows:

$$\frac{\Delta D}{\Delta t} = 1.47\ (V) \qquad (28)$$

This sensitivity varies from a rate of 29.3 feet per second change in PRT (46.9 m/s) at a design velocity of 20 mph (32 km/h) to a rate of 103 feet per second (164 m/s) at a design velocity of 70 mph (113 km/h). The <u>percentage</u> change (rather than <u>unit</u> change) in the necessary intersection sight distance per unit change in perception-reaction time is computed using the following formula:

$$\frac{\%\ \Delta \text{ in } D}{\Delta t} = \frac{1}{t_1} \qquad (29)$$

where: t_1 = initial perception-reaction time, sec

At the specification for perception-reaction time, 3.0 seconds, the instantaneous percentage change in necessary sight distance per one second change in perception-reaction time is 33.3 percent/second.

To take this type of analysis one step further, the percentage change in necessary sight distance per one percent change in perception-reaction time is 1; in other words, necessary sight distance and perception-reaction time vary at the same percentage rate. For example, an increase in perception-reaction time from the specification of 3.0 seconds to the estimated 85th percentile value of 3.4 seconds, an increase of 13.3 percent, would increase the <u>calculated</u> necessary sight distance by a corresponding 13.3 percent.

The formulation for Case I Intersection Sight Distance provides the driver of a vehicle time to perceive and recognize the approaching vehicle, to decide what type of evasive action (if any) is necessary, and to move his/her foot to either the accelerator or brake pedal. The formulation does not provide any time for the evasive action to be taken. In other words, the vehicles collide at the inter-

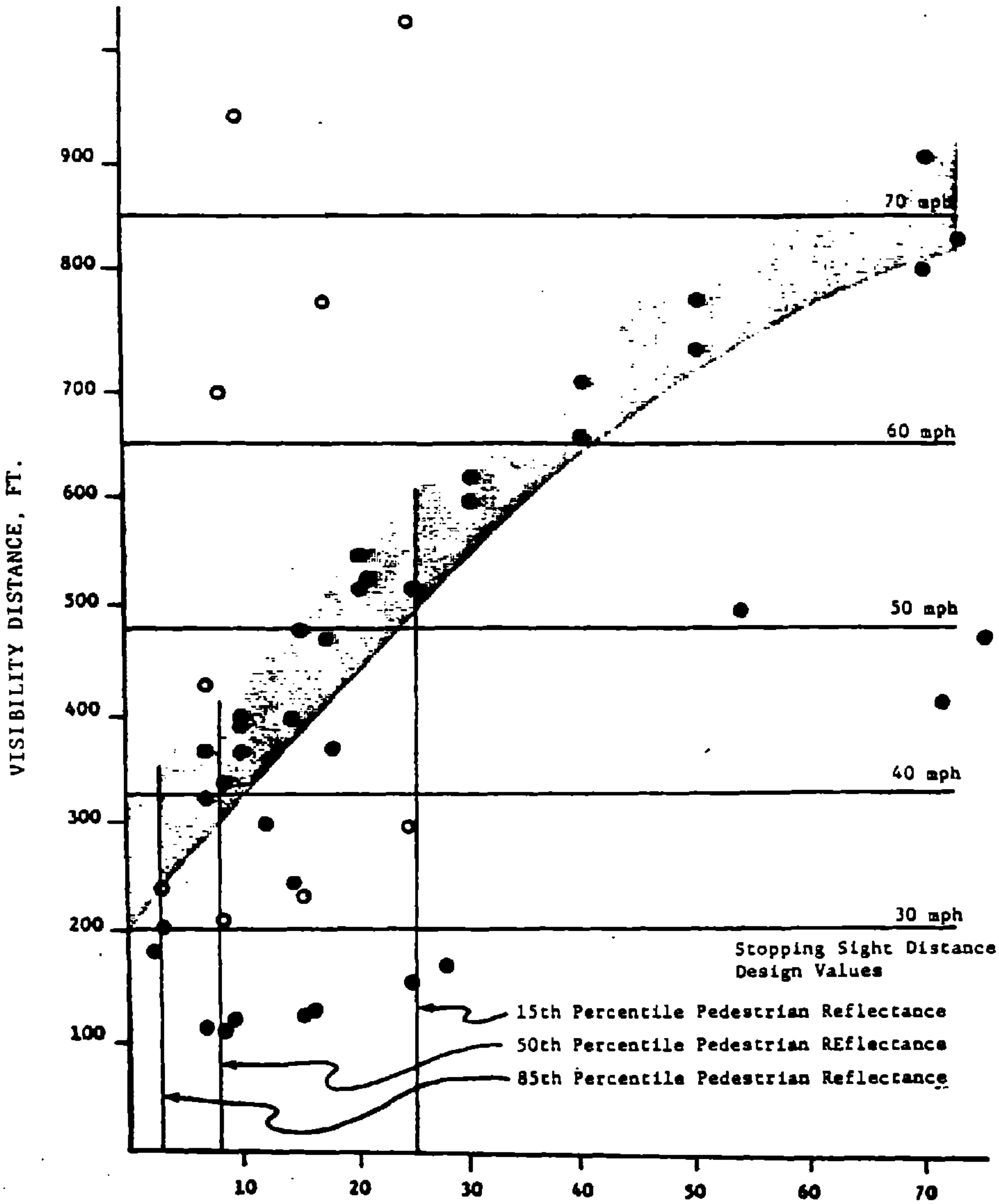


FIGURE 11--Approximate Maximum Visibility Distances to Pedestrians During Nighttime Conditions Based on Actual Pedestrian Reflectance Distributions

The SI (metric) conversions are 1 ft = 0.3 m.

section before either driver has had time to change the vehicle speed. Obviously, a more desirable sight distance would allow the drivers of the vehicles to be able to take adequate evasive action. Considering this concept, a proposed formulation is as follows:

$$D_A = 1.47V_AP + \frac{WV_A}{V_B} - \frac{dW^2}{2.93V_B^2} \qquad (30)$$

$$\left(D_A = \frac{V_AP}{3.6} + \frac{WV_A}{V_B} - \frac{1.8dW^2}{V_B^2}\right)$$

where: D_A = minimum intersection sight distance for vehicle A, ft (m)

V_A = design speed for vehicle A, mph (km/h)

P = perception-reaction time, sec

W = width or roadway on which vehicle A is traveling, ft ft (m)

V_B = design speed for vehicle B, the conflicting vehicle, mph (km/h)

d = deceleration rate of vehicle A, mphps (km/h/s)

Table 16 lists the minimum allowable intersection sight distances for intersections with various approach speeds, as computed with the formula presented above. Each of the values, as expected, exceeds the AASHTO design standard for Case I Intersection Sight Distance. For each design speed, the minimum allowable sight distance decreases as the approach speed of the "conflicting" vehicle increases. In other terms, as the speed of the conflicting vehicle increases, that vehicle passes through the intersection faster and thus the design vehicle does not need to decelerate for as long a period of time nor for as great a distance. It should also be noted from Table 16 that an increase in perception-reaction time from the specification value, 3.0 seconds, to the estimated 85th percentile value, 3.4 seconds, results in over a 10 percent increase in calculated minimum intersection sight distance.

Figure 7 illustrates the effect of providing the AASHTO (5) Case I Intersection Sight Distances on minimum allowable perception-reaction time. Based on the proposed formulation described above, all values are below the specification value of 3.0 seconds and well below the estimated 85th percentile value of 3.4 seconds. The computed values range from as low as 2.19 seconds to as high as 2.85 seconds.

Exclusion Analysis

As shown in Table 11, it is estimated that one-half of the driving population has a perception-reaction time greater than 2.6 seconds when confronted with the traffic conditions presented in Case I. And as shown in Figure 7, the perception-reaction time value of 2.6 seconds falls somewhere in the middle of the range of maximum allowable perception-reaction times that are based on the proposed revised methodology for computing Case I Intersection Sight Distances. Therefore, it is estimated that approximately 50 percent of the driving population cannot perform to meet the AASHTO (5) standards.

Even if the AASHTO formulation is used as the basis by which population exclusion is estimated, a high percentage results. The specification value, 3.0 seconds, corresponds to approximately the 65th percentile driver; thus over one-third of all drivers cannot even satisfy this criterion.

INTERSECTION SIGHT DISTANCE - CASE II ENABLING VEHICLES TO STOP

Sensitivity Analysis

As was noted previously, the Case II Intersection Sight Distance formulation is merely an application of the stopping sight distance formulation. A detailed sensitivity analysis of this latter formulation is presented earlier in this chapter under Stopping Sight Distance. The reader is referred to that section for the complete analysis of the interrelationship between stopping sight distance and the driver characteristic, perception-brake reaction time.

The 50th, 85th, and 95th percentile values for Case II perception-reaction time are estimated to be 2.6, 3.4, and 4.0 seconds, respectively. The current specification value is 2.5 seconds.

Table 17 displays revised minimum and desirable stopping sight distances and the percent changes from the current rounded stopping sight distances for the four values of perception-brake reaction time. For example, if the value of 3.4 seconds was selected as the new specification, the calculated desirable stopping sight distance would be 18 percent higher than the current standard value at 30 mph (48 km/h) and nearly 10 percent higher at 70 mph (113 km/h).

TABLE 16--Actual Sight Distance Required for Case I Intersection Sight Distance Based on Suggested Revised Formulation

Design Speed of Vehicle Approaching An Uncontrolled Intersection, mph (3)	Minimum Allowable Intersection Sight Distance (@PRT = 3.0s/@PRT =3.4s) feet (1) — Approach Speed of "Conflict" Vehicle, mph (2): 20	30	40	50	60	70	Design Intersection Sight Distance (4) feet
20	107 / 119	102 / 114	99 / 110	97 / 109	95 / 107	94 / 106	90
30	163 / 181	154 / 171	149 / 166	146 / 163	143 / 161	142 / 159	130
40	219 / 243	206 / 229	199 / 222	194 / 218	191 / 215	189 / 213	180
50	275 / 304	258 / 287	249 / 278	243 / 272	239 / 269	237 / 266	220
60	331 / 366	310 / 345	299 / 334	292 / 327	287 / 323	284 / 319	260
70	387 / 428	362 / 403	349 / 390	341 / 382	335 / 376	332 / 373	310

349 — Minimum Allowable Sight Distance @PRT = 3.0s

390 — Minimum Allowable Sight Distance @PRT = 3.4s

NOTES:

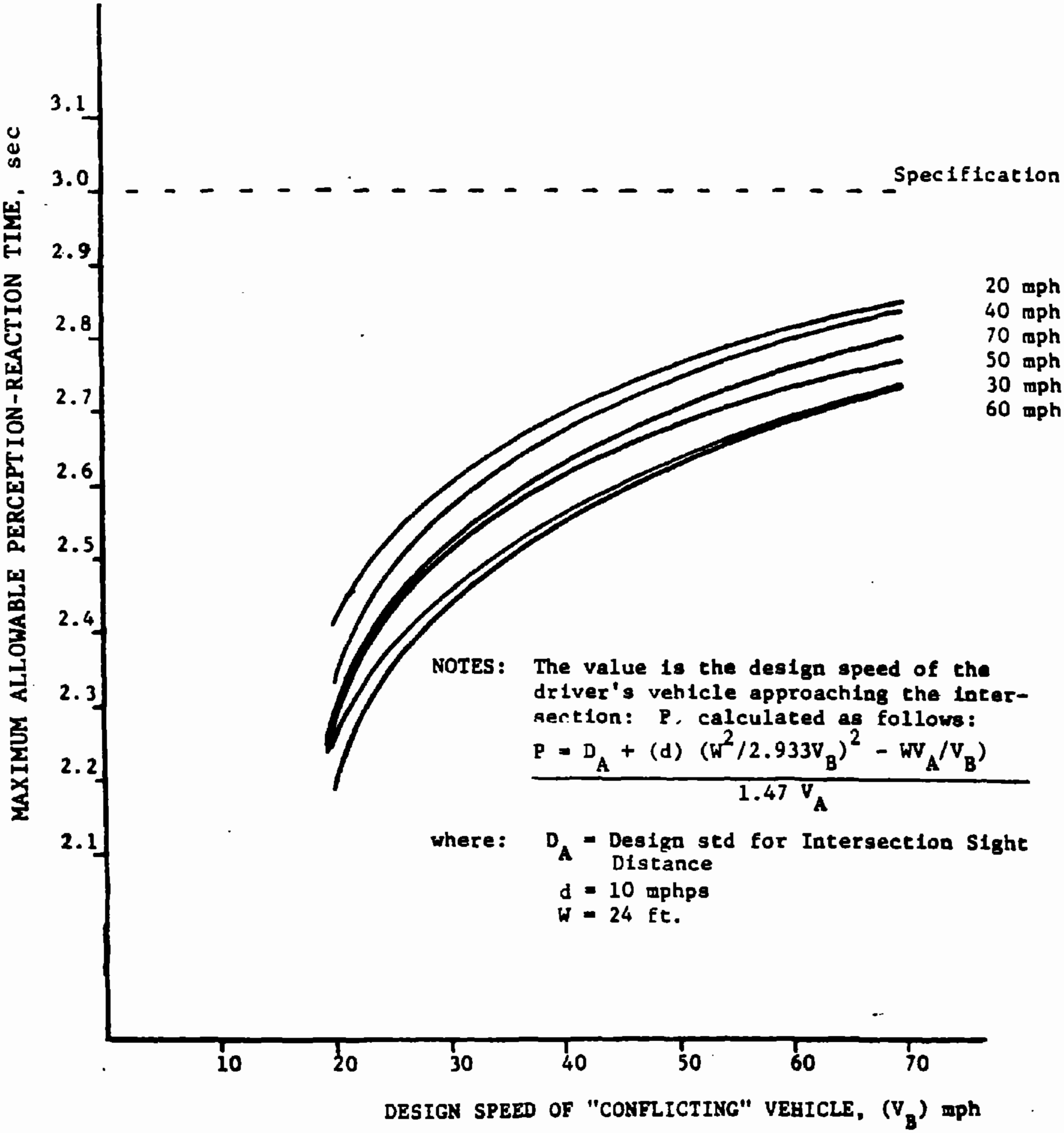
(1) Values calculated based on methodology presented in text; values expressed in feet; assumes roadway width is 24 feet and deceleration rate is 10 mphps.

(2) Speed of vehicle approaching intersection which driver is decelerating to avoid.

(3) Speed of vehicle approaching intersection which will be decelerated.

(4) Standard value recommended by AASHTO (1).

The SI (metric) conversions are 1 mph = 1.61 km/h and 1 ft = 0.3 m.



The SI (metric) conversions are 1 mph = 1.61 km/h and 1 ft = 0.3 m.

FIGURE 7--Maximum Allowable Perception-Reaction Time For Case I Intersection Sight Distance Design Values Based on the Revised Formulation

TABLE 17 --Computed Case II Intersection Sight Distances Based on Various Values of Perception-Brake Reaction Time

Design Speed, mph	Design SSD, ft[1]	COMPUTED SSD, ft/(PERCENT INCREASE ABOVE STANDARD) @RT = 2.5 S (Specification)	@RT = 2.6 S (50th %ile)	@RT = 3.4 S (85th %ile)	@RT = 4.0 S (95th %ile)
30	200 (min)	177 (-12)	181 (-9.5)	214 (7.0)	239 (20)
	200 (des)	196 (-2.0)	200 (0)	235 (18)	262 (31)
40	275 (min)	267 (-2.9)	272 (-1.1)	315 (15)	346 (26)
	325 (des)	313 (-3.7)	319 (-1.8)	366 (13)	401 (23)
50	375 (min)	376 (0.3)	383 (2.1)	435 (16)	473 (26)
	475 (des)	461 (-2.9)	468 (-1.5)	527 (11)	571 (20)
60	525 (min)	501 (-4.6)	509 (3.0)	570 (8.6)	616 (17)
	650 (des)	634 (-2.5)	643 (-1.1)	713 (9.7)	766 (18)
70	625 (min)	613 (-1.9)	622 (-0.5)	690 (10)	741 (19)
	850 (des)	840 (-1.2)	850 (0)	932 (9.6)	994 (17)

NOTES: (1) The two values given are the "minimum" and "desirable" values for Stopping Sight Distance; Source: AASHTO (1)

(2) The SI (metric) conversions are 1 mph = 1.61 km/h and 1 ft = 0.3 m.

Exclusion Analysis

The current specification for perception-brake reaction time is 2.5 seconds, which is based on a driver of a vehicle sighting and stopping for a stationary object in the roadway. However as was noted previously, the driver in this situation (Case II Intersection Sight Distance) is required to perform a more complex set of actions. The estimated values listed earlier in Table 11 represent more accurately the driver characteristic involved in this particular situation than does the 2.5 second value.

It should be emphasized that the values in Table 11 are to be considered estimates. They were determined by adding values, in some instances estimates themselves, of discrete components of the perception-brake reaction time. It cannot be stated with certainty that these values do represent the true distribution of the driving public, as they were based on relatively small sample sizes and less than actual driving conditions.

Perception-brake reaction varies among the driving population and varies for any one driver depending upon the situation. Consequently, it is difficult to state categorically that a certain percentage of drivers are being excluded with the 2.5 second specification. It can be safely assumed that the current specification does not accommodate all drivers for all situations. It is estimated that approximately 50 percent of the driving population is essentially excluded by adoption of the 2.5 seconds specification. It cannot be stated what specific group of drivers are being excluded but the elderly, with their slower reaction times, are likely predominant in the excluded group.

Critique of Specification

As has been noted several times above, Case II Intersection Sight Distance standards based on the current 2.5 seconds specification for driver perception-brake reaction time accommodate only an estimated one-half of the drivers approaching uncontrolled intersections. Despite what would appear from this analysis to be a seriously hazardous set of standards, very little statistical evidence has been developed or presented to substantiate this hypothesis. Most of the applicable research work found in a literature search dealt with the safety impact of providing less than the AASHTO guidelines for intersection sight distance. It could be argued (based on the estimation that the specification value of 2.5 seconds is low) that these studies are comparing two like set of conditions, each of which is hazardous. A more to the point analysis would establish estimated accident rates for incremental intersection sight distances, including those in excess of the AASHTO design standards. An analysis of that sort may provide a clearer picture of the appropriate threshold values that economically optimize intersection safety.

The AASHTO (5) definition of the conditions describing Case II - Enabling Vehicles to Stop states that the "operator of a vehicle on either highway must be able to see the intersection and the intersecting highway in sufficient time to stop the vehicle before reaching the intersection." The three different relative approach patterns for two vehicles at an uncontrolled intersection--either vehicle A arrives first, or vehicle B, or they arrive simultaneously. According to the above definition, Case II sight distance should enable a driver confronted with the latter situation (i.e., vehicles to collide) to bring the vehicle to a complete stop before reaching the intersection. The AASHTO standards for stopping sight distance no not in some cases provide this adequate sight distance. Instead, the AASHTO standards will sometimes place the driver in a situation where a speed adjustment must be made to avoid a collision but where stopping distance is not available; in other words, Case I--Enabling Vehicles to Adjust Speed applies.

INTERSECTION SIGHT DISTANCE - CASE III ENABLING STOPPED VEHICLES TO CROSS MAJOR STREETS

Sensitivity Analysis

The most direct measure of sensitivity between the design standard (Case III Intersection Sight Distance) and the driver characteristic (perception-reaction time) is the partial derivative of intersection sight distance with respect to perception-reaction time:

$$\frac{\partial(D)}{\partial(J)} = 1.47V \qquad (31)$$

(refer to equation (6))

For example, if the velocity of the approaching vehicles is 60 mph (96 km/h) a one second change in perception-reaction time necessitates an 88 feet (27m) increase in sight distance. The percentage change in the D value for a unit change in J is computed with the following formula:

$$\frac{\partial(D)}{\partial(J)} \cdot \frac{1}{D} = \frac{1}{J + t_a} \qquad (32)$$

These values range from approximately 0.14 percent/second for passenger vehicles to 0.08 percent/second for WB-50 vehicles. The percentage change in J is D value for a percentage change in J is computed with the following formula:

$$\frac{\partial (D)}{\partial (J)} \frac{J}{D} = \frac{J}{J + t_a} \qquad (33)$$

These values range from approximately 0.29 percent/percent for passenger vehicles if perception-reaction time is 2.0 seconds to 0.16 percent/percent for WB-50 vehicles if perception-reaction time is 2.0 seconds. Therefore, the computed value for Case III Intersection Sight Distance changes at a much slower rate than does perception-reaction time. Or, in other words, Case III Intersection Sight Distance is not highly sensitive to perception-reaction time, especially in comparison with its sensitivity to vehicle acceleration.

Table 18 lists the computed changes in intersection sight distance that result from incremental increases and decreases in perception-reaction time. Note that although the absolute values for intersection sight distance will vary depending on the vehicle approach speed, the percentage change in D remains the same. The analysis presented in Table 18 is particularly useful in evaluating the sensitivity of Case III Intersection Sight Distance to the driver characteristic, J, because it is not currently possible to develop from existing research a definitive population distribution of driver characteristic values. Instead, only an approximate range of 1.4 to 2.8 seconds for the general driving population can be established. Both of these values are evaluated for their sensitivity in Table 18, as well as incrementally-stepped values in between.

Exclusion Analysis

Based on current available research, an accurate population distribution cannot be developed for the driver characteristic, perception-reaction time. For that reason, it is not possible to estimate what segment of the driving population is excluded by the Case III Intersection Sight Distance design standards. However, it can be safely stated that the current specification is neither much too high nor much too low for the general driving population.

Equally undeterminable is what effect takes place by not designing for those motorists with perception-reaction times longer than the specification, 2.0 seconds. As is stated previously under Case III--Intersection Sight Distance, a direct quantifiable relationship between sight distance and accidents has not developed. Logic, of course, would lead to the conclusion that "excluded" motorists would be more susceptible to accidents involving limited sight distance at intersections.

Critique of Specification

The specification value for the driver characteristic, perception-reaction time, appears to represent an adequate percentage of the driving population. The synthesis of current, applicable research suggests a range of values for the general driving population which brackets the specification value of 2.0 seconds. The estimated 85th percentile value of 2.2 seconds causes a less than 4 percent increase in the computed sight distance required for passenger vehicles and a just over 1 percent increase in the distance required for WB-50 vehicles.

INTERSECTION SIGHT DISTANCE - CASE IV & V ENABLING A STOPPED VEHICLE TO TURN LEFT ONTO A TWO-LANE HIGHWAY

As indicated in Chapter II, there are two new cases for intersection sight distance design. Referring back to Figure 1, it can be seen that both cases (curve C is Case IV and curve D is Case V) require considerably longer sight distance than indicated by curve B (Case III) for a given speed. The longer distance result from the longer time it takes to turn left and for Case V the distance required for the turning vehicle to reach the speed of the vehicle approaching from the right. These additional distances are not related to a driver characteristic and, therefore, insensitive to a change in the driver characteristic, J, the perception-reaction time. Since it was shown that the sight distance required for Case III varies only slightly with a significant change in the driver characteristic then it follows that Case IV and V would be even less sensitive.

RAILROAD-HIGHWAY GRADE CROSSING SIGHT DISTANCE - CASE I CORNER SIGHT DISTANCE TRIANGLE

Sensitivity Analysis

The sensitivity analysis for this standard can be approached in the same way as for stopping sight distance. The first analysis is to determine the "sensitivity index" (i.e., the percent change in the standard resulting from a one percent change in the specification). This value

TABLE 18--Changes in Case III Intersection Sight Distance as a Function of Changes in Perception-Reaction Time

DESIGN VEHICLE	PERCENT CHANGE FROM SPECIFICATION FOR J	NEW J,(1) sec.	NEW D,(2) ft	CHANGE IN D,(3) FROM STANDARD, ft	% CHANGE IN(3) D FROM STANDARD
Passenger Car (P)	50	3.0	590	90	16
	30	2.6	560	50	10
	20	2.4	540	30	5.9
	10	2.2	530	20	3.9
	5	2.1	520	10	2.0
	0	2.0	510	0	0
	-5	1.9	500	-10	-2.0
	-10	1.8	500	-10	-2.0
	-20	1.6	480	-30	-5.9
	-30	1.4	470	-40	-7.8
	-50	1.0	440	-80	-14
Single Unit Truck (SU)	50	3.0	760	70	10
	30	2.6	730	40	5.8
	20	2.4	720	30	4.3
	10	2.2	700	10	1.4
	5	2.1	700	10	1.4
	0	2.0	690	0	0
	-5	1.9	680	-10	-1.4
	-10	1.8	670	-20	-2.9
	-20	1.6	660	-30	-4.3
	-30	1.4	650	-40	-5.8
	-50	1.0	620	-70	-10
50 ft. Wheelbase Truck (WB-50).	50	3.0	980	70	7.7
	30	2.6	950	40	4.4
	20	2.4	940	30	3.3
	10	2.2	920	10	1.1
	5	2.1	920	10	1.1
	0	2.0	910	0	0
	-5	1.9	900	-10	-1.1
	-10	1.8	890	-20	-2.2
	-20	1.6	880	-30	-3.3
	-30	1.4	870	-40	-4.4
	-50	1.0	840	-70	-7.7

NOTES: (1) Computed by making listed percentage change to specification for J, 2.0 sec.

(2) Computed using the formula "$D = 1.47V (J + t_a)$;" assume V = 50 mph and the width of the main highway is 24 ft Values are rounded.

(3) Uneven steps in values are due to rounding in the computation of D.

(4) The SI (metric) conversions are 1 mph = 1.61km/h and 1 ft - 0.3 m.

is computed by taking the partial derivative of the standard with respect to the driver characteristic, dividing by the standard, and then multiplying by the driver characteristic.

Tables 19 and 20 lists, for various highway speeds, the "sensitivity indices" for distances along the highway, D_H, and along the track, D_R, respectively. Similar to stopping sight distance two speeds are used--design speed for desirable values and assumed speed for minimum values. The sensitivity indices indicate the percent change in the sight distance resulting from a one percent change in the specification for the perception-brake reaction time. For example, in Table 19 it can be seen that a design speed of 70 mph (113 km/h) the sight distance down the highway will increase 0.30 percent for a one percent increase in perception-brake reaction time. Similarly, in Table 20, it can be seen that at a design speed of 70 mph (113 km/h) the distance down the track will increase 0.27 percent for a one percent increase in perceptoin-brake reaction time. Train speed does not affect the sensitivity index; hence, it would be the same for all values of train speed for a given highway speed.

Since the equations (7) and (8) used to develop these values are nearly the same as for Stopping Sight Distance the sensitivity indices are similar. However, the additional 25 feet (7.6 m) in equation (7) and the additional 100 feet (30.5 m) in equation (8) has the effect of minimizing the sensitivity of the sight distance due to a change in the perception-brake reaction time.

The sensitivity indices were used to develop the plots in Figures 8 and 9 which show the percent change in the current distance for the 50th, 85th and 95th percentile estimated values of perception-reaction time for distance along the highway and distance along the track, respectively. The sensitivity indices used for the plots were those for "desirable" distances shown in Tables 19 and 20.

From the plots, then, it can be determined what the new distance would be for any of three percentile values of perception-reaction time. From Figure 8, for example, it can be seen that for a design vehicle speed of 60 mph (97 km/h) and an 85th percentile perception-reaction time of 3.5 seconds the distance along the highway would incease 13.4 percent. From Figure 9 for the same design vehicle speed and some perception-reaction time, the distance along the track would increase 12.0 percent. For a given design vehicle speed, the changes are constant for all train speeds.

Exclusion Analysis and Critique of Specification

The current specification of the perception-reaction time is 2.5 seconds, which as already noted, is the same for stopping sight distance. But for reasons already discussed, the perception-reaction process involved in a passive-controlled railroad-highway grade crossing is much more demanding than that for stopping sight distance. Consequently, it can be expected that perception-reaction time would be higher than that for a stopping distance.

By adding time values determined from the literature for discrete elements of the perception-reaction process, estimated values were determined for the 50th, 85th, and 95th percentile perception-reaction time. With the current specification set at only 2.5 seconds, the standard would only accommodate slightly over 50 percent of the drivers, since the estimated 50th percentile value is 2.3 seconds.

However, this does not necessarily mean that close to 50 percent of the drivers cannot perform within the specification of 2.5 seconds.

It may be that the estimated values are on the conservative side. This is because it has been assumed that a person who performs at an 85 percent level for the detection process would also perform at that level for all elements of the process which may not be the case. Also, it has been assumed that all elements of the process are sequential without any time sharing of subtasks, which may be too conservative an assumption.

One way to test the appropriateness of the 2.5 second perception-reaction time specification would be to examine the relationship of available sight distance to train-vehicle accidents. If the 2.5 seconds were too low for a large percentage of the drivers, then presumably, an accident analysis might reveal a positive correlation. Unfortunately the literature is silent on this aspect.

TABLE 19--Sensitivity Indices for Distance Along the Highway With Respect to a Change in Perception-Reaction time

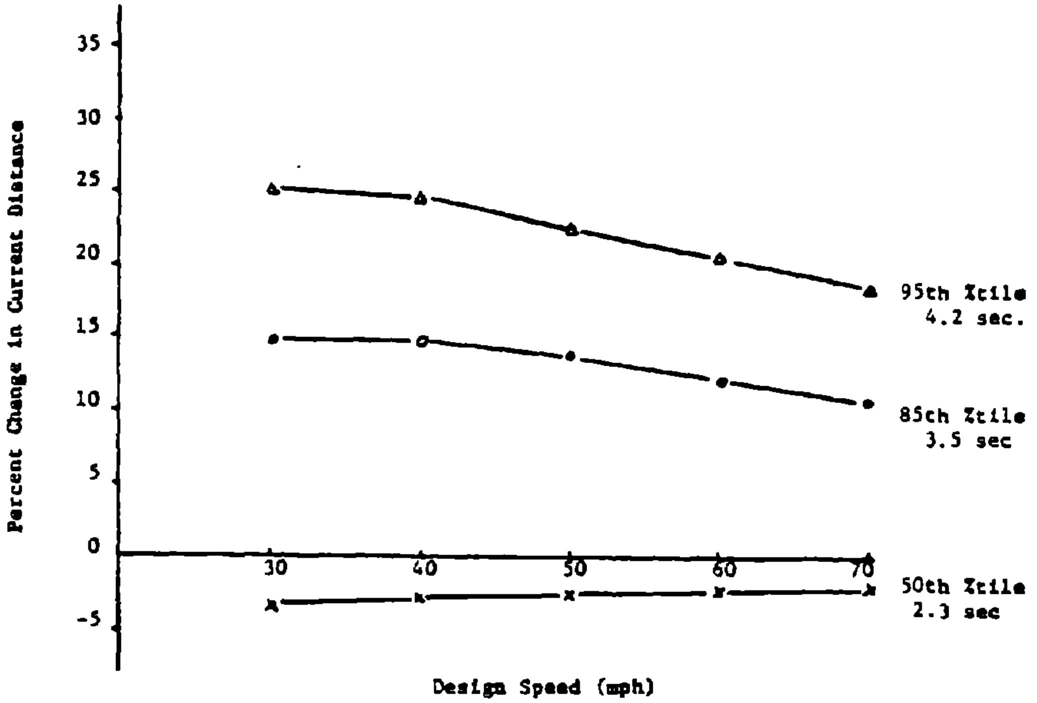
Design Speed (mph)	Assumed Speed (mph)	Friction Value (f)	% Δ in D_H With 1 % Δ in t minimum	desirable
30	28	0.35	0.51	0.50
40	36	0.32	0.45	0.43
50	44	0.30	0.40	0.38
60	52	0.29	0.36	0.33
70	58	0.28	0.33	0.30

1 mph = 1.61 km/h

TABLE 20--Sensitivity Indices for Distance Along the Track With Respect to a Change in Perception-Reaction Time

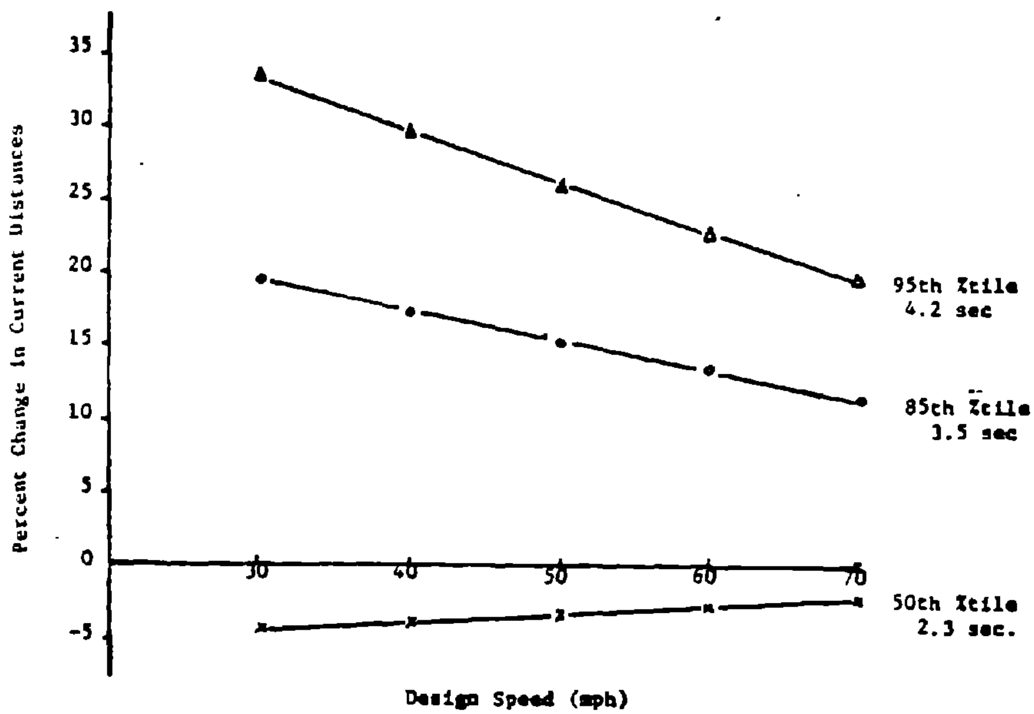
Design Speed (mph)	Assumed Speed (mph)	Friction Value f	% Δ in D_R with 1% Δ in t minimum	desirable
30	28	0.35	0.37	0.37
40	36	0.32	0.36	0.36
50	44	0.30	0.34	0.33
60	52	0.29	0.32	0.30
70	58	0.28	0.30	0.27

1 mph = 1.61 km/h



SI (metric) conversion is 1 mph = 1.61 km/h

FIGURE 8--Percent Change in Current Computed Distance Along the Track for Three Values Perception-Reaction Time



SI (metric) conversion is 1 mph = 1.61 km/h

FIGURE 9 --Percent Change in Current Distance Along the Highway for Three Values of Perception-Reaction Time

RAILROAD-HIGHWAY GRADE CROSSING SIGHT DISTANCE - CASE II SIGHT DISTANCE ALONG RAILROAD FOR STOPPED VEHICLES TO CROSS IN FRONT OF TRAIN

Sensitivity Analysis

Equation (9), presented earlier, is used to determine the required sight distance along the railroad for a vehicle at a stopped position to cross in front of a train. When the partial derivative of the standard, i.e., the distance D_T, with respect to the driver characteristic, J, is divided by D and then multiplied by J, the resulting sensitivity index of 0.12 results. This is a constant value which applies to all values of train speed (V_T). It means that if the driver characteristic specification of 2.0 second is changed by 1 percent the sight distance will change by 0.12 percent.

The current specification for the driver characteristic is 2.0 seconds. In Chapter III, this perception-reaction time characteristic was discussed with the resulting conclusion that the range was from 1.4 to 2.4 seconds with the lower value applicable to the 50th percentile of the driving population and the higher value applicable to at least the 85th percentile.

Shown in Table 21 are changes in the sight distance requirements that would result if the current specification of 2.0 seconds. If 1.4 seconds were to be used all the distances would be reduced by 3.6 percent and if 2.4 seconds were to be used the distances would be increased by 2.4 percent. These are relatively small changes compared to the current standard.

Exclusion Analysis and Critique of Specification

Prior to the preparation of the new edition of the Traffic Control Devices Handbook (17), a driver characteristic had not been considered in this standard. With the adoption of a 2.0 second specification for the perception-reaction time, the required sight distances have been increased substantially.

No empirical data could be identified in the literature that indicates what the actual perception-reaction time is for this situation. However, a range of 1.4 to 2.4 seconds has been determined analytically. It would seem, therefore, that the 2.0 seconds is a reasonable value for this standard and does not exclude very many drivers. In fact, it could be reasoned that the 1.4 second value may be more appropriate because the standard is based on large trucks, and therefore, truck drivers. These drivers are typically more skilled and experienced and would perform better than most with regard to perception-reaction time.

In any event, the standard is relatively insensitive to a change in the driver characteristic. Because of this and the fact that the current 2.0 second specification is reasonable, there is no compelling reason to change the specification or define it more accurately through empirical research.

TABLE 21--Changes in Sight Distances at Railroad-Highway Grade Crossings for Stopped Vehicles to Cross in Front of Train with Changes in Driver Characteristic Specification

Train Speed mph	Sight Distance (ft) Current Standard W/PRT = 2.0 sec	Change in Distance (ft) with PRT Time of 1.4 sec	2.4 sec
10	240	- 9	+ 6
20	480	-17	+12
30	720	-26	+17
40	960	-35	+23
50	1,200	-43	+29
60	1,440	-52	+35
70	1,680	-60	+40
80	1,920	-69	+46
90	2,160	-78	+52

The SI (metric) conversion is 1 mph = 1.61 km/h.

CREST VERTICAL CURVE LENGTH

Sensitivity Analysis

In order to maintain a particular sight distance on a crest vertical curve when the driver eye height is lowered, a longer (flatter) curve must be provided. If eye height is decreased by one percent, longer vertical curve must be provided. A decrease in eye height to the estimated 85th percentile value of 41.1 inches (1.04 m) results in a 1.58 percent increase in curve length; a decrease to the estimated 95th percentile value necessitates a 3.22 percent increase in curve length. In order to effect a 5 percent increase in vertical curve length where S is less than L, eye height must be decreased from the 42 inch (1.07 m)--which is only 0.4 inches (10 mm) less than the suggested 95th percentile value for driver eye height.

For crest vertical curves with sight distance less than curve length, the maximum allowable perception-brake reaction time and distance are all greater than the 2.5 second specification if the driver's eye height is 42.0 inches (1.07 m). For drivers with an eye height of 40.2 inches (1.02 m)--the estimated 95th percentile value--additional distance to stop is needed than is available on the crest vertical curve. However, this additional distance amounts to no more than a two percent overrun. The computed values are listed in Table 22.

The relatively large increases in K values necessitated by a 2.8 second perception-brake reaction time are much more sizeable than those caused by changes in driver eye height.

Table 22 also lists the sight deficiencies caused by perception-reaction time of 2.8 and 3.2 seconds. For example, a driver whose perception-brake reaction time is 2.8 seconds on a crest vertical curve designed for V = 60 mph (97 km/h) will come to a stop 21 feet (6.4 m) beyond the sight distance of 503 feet (153 m). If the driver's perception-brake reaction time is 3.2 seconds on the same vertical curve, the vehicle will stop 52 feet (16 m) beyond the "sighted" object.

The AASHTO design values for minimum crest vertical curve length are expressed in the units of a "K" value which is simply the ratio of curve length to the algebraic difference in grades (expressed in feet). In equation form, K equals L/A.

Table 23 lists the computed K values needed in order to provide adequate stopping sight distance on crest vertical curves as a function of two driver eye heights and two perception-brake reaction times. These computed values are then compared to the actual design K values.

TABLE 22--Stopping Sight Distance Deficiencies on Crest Vertical Curves at AASHTO Design Values(1)

V (mph)	Computed Sight(2) Distance (ft) Provided by Design Values	Additional Distance (ft)(3) Needed for Vehicle to Stop @ RT=2.5/2.8/3.2 sec	@ He=4.1"/40.2"
30	200	0/9/27	0/0
40	282	0/1/22	0/0
	326	0/5/28	0/0
50	382	0/14/40	0/0
	461	0/22/51	4/7
60	503	0/21/52	3/7
	642	0/18/53	0/2
70	621	0/18/52	0/2
	842	0/24/65	0/6

Notes: (1) Source: AASHTO (1)
(2) Assumes He = 41 in (1.07 m) and Ho = 6 in (0.15 m)
(3) The distance the stopping vehicle will travel beyond the "sighted" object in the roadway; the "additional distance" calculations for varying reaction times assumes He = 42 in (1.07 m); the "additional distance" calculations for varying eye heights assumes RT = 2.5 sec.

The SI (metric) conversions are 1 mph = 1.61 km/h and 1 ft = 0.3 m.

TABLE 23--Design Crest Vertical Curve "K" Values for Various Driver Eye Heights and Perception-Brake Reaction Time

Design Speed (mph)	Design K (1)	Calculated "K" Value/(Percent Increase Above Standard)							
		@ He = 41.1 in. (2) (85th %ile)		@ He = 40.2 in. (2) (95th %ile)		@ RT = 2.8 sec (3)		@ RT = 3.2 sec (3)	
30 mph	30 (min)	24.1	(-20)	24.5	(-18)	27.1	(-9.8)	32.0	(6.7)
	30 (des)	29.3	(-2.3)	29.7	(-1.0)	32.8	(9.4)	38.6	(29)
40	60 (min)	54.4	(-9.3)	55.3	(-7.8)	60.2	(0.3)	69.5	(16)
	80 (des)	75.1	(-6.1)	76.3	(-4.6)	82.4	(3.0)	94.5	(18)
50	110 (min)	109	(-0.9)	110	(0)	118	(7.3)	134	(22)
	160 (des)	163	(1.9)	165	(3.1)	176	(10)	198	(24)
60	190 (min)	192	(1.1)	195	(2.6)	207	(8.9)	232	(22)
	310 (des)	307	(-1.0)	312	(0.6)	328	(5.8)	364	(17)
70	290 (min)	287	(-1.0)	292	(0.7)	307	(5.9)	340	(17)
	540 (des)	539	(-0.2)	548	(1.5)	570	(5.6)	626	(16)

NOTES: (1) The two values are the "minimum" and "desirable" design values for K (K is defined as Curve Length, in feet, divided by algebraic difference in grades, in percent)

(2) Assumes RT = 2.5 seconds

(3) Assumes He = 42.0 inches

The SI (metric) conversions are 1 mph = 1.61 km/h and K(ft/pct) = 0.3 K(m/pct).

All of the above calculations have been based on the assumption that the "g" value--grade--has been equal to zero. Our research indicates that the literature fails to define the use of the "g" value for roadway surfaces whose grades change during the braking action. In the case of crest vertical curves for instance, it is unknown whether to use the "g" value at the instant of brake application, the "g" value at the stopping point, or (more probably) somewhere in between. The impacts of the "g" value used in crest vertical curve length calculations can be quite significant. If a grade of -4 percent is assumed, none of the K (minimum) or K (desirable) design values allow perception-brake reaction times as high as the specification, 2.5 seconds. Even at a grade of -2 percent, all perception-brake reaction times (except those for a 40 mph (64 km/h) design speed) likewise fall below 2.5 seconds.

If perception-brake reaction time is assumed to be 2.5 seconds and driver eye height to be 42 inches (1.07 m) a grade of -4 percent would require a 25 percent increase in K (desirable) values for a design speed of 70 mph (113 km/h) in order to provide for adequate stopping sight distance.

Exclusion Analysis and Critique of Specification

Throughout the preceding sensitivity analysis it became evident that auto driver eye height, within its current range of values, has relatively little impact on crest vertical curve characteristics. Decreases in driver eye height to the 95th percentile value result in sight distance reductions in most cases of approximately 1.5 percent of the current standard. Therefore, the disproportionately high percentage of female drivers who comprise the "shortest" drivers are not significantly excluded by current standards for crest vertical curve characteristics.

As was shown earlier in Table 22, any driver with a perception-brake reaction time of 2.8 seconds is excluded by the current crest vertical curve standard. Thus fully 15 percent of the driving population may be excluded. If the specification for perception-brake reaction time is ever increased to 2.8 seconds, nearly all K design values for crest vertical curves will need to be adjusted upward. However, as was noted previously, safety studies have not drawn a direct correlation between traffic safety and inadequate stopping sight distances on crest vertical curves. Therefore, it would be premature to consider adjusting the current design K values to account for the 2.8 seconds. This should take place only after further empirical research indicates either that 2.5 seconds does not satisfy the 85th percentile driver or that sight distances based on current K values affect traffic safety adversely.

As for drivers with eye heights of less than 42 inches (1.07m), they are likewise excluded by the current standard, although the result is much less severe both because only two design speeds are affected and because the sight distance shortfall is relatively insignificant.

SAG VERTICAL CURVE LENGTH

Sensitivity Analysis

The sensitivity of sag vertical curve geometrics to unit changes in the driver characteristic perception-brake reaction time can be obtained by taking the derivative of the "headlight sight distance" formulas, equations (14) and (15). On sag vertical curves where sight distance is less than the curve length, the "threshold" value for K changes as a function of P (perception-reaction time) at a rate that varies from 3.2 percent per one-tenth second change in P on 30 mph (48 km/h) design speed curves to 1.4 percent per one-tenth second change in P on 70 mph (113 km/h) design speed curves.

For sag vertical curves designed at K (minimum) and K (desirable) the maximum allowable perception-brake reaction time for a vehicle to stop in sufficient time and distance is at least 2.5 seconds, the specification value, for each of the design speeds. The maximum allowable perception-brake reaction time values for each K (minimum) and K (desirable) design standard are listed in Table 24. The table also lists the distance which a vehicle will overshoot the "sighted" object due to a deficiency in sight distance based on a range of perception-brake reaction times. The negative values in the table indicate the vehicle will come to a stop in front of the "sighted" object. Thus a driver with a perception-brake reaction time of 2.5 seconds will be able to stop a vehicle within a distance at least 9 feet (2.7 m) less than the sight distance on any design curve. However, a driver on a 50 mph (80 km/h) design curve who has a perception-brake reaction time of 2.8 seconds will overshoot an object sighted at the maximum available sight distance, 477 feet (145 m), by 6 feet (1.8 m)--or, in other terms, by 1.3 percent.

TABLE 24 --Stopping Sight Distance Deficiencies on Sag Vertical Curves at Design "K" Values

V	Design[(1)] K (ft/pct)	Sight Distance[(2)] Provided by K	Additional Distance[(3)] Needed for Vehicle to Stop (@RT = 2.5/2.8/3.2 sec)	RT[(4)] @ Design K
30 mph	40 (min)	215 ft	-38/-25/-9 ft	3.41 sec
	40 (des)	215	-19/-6/12	2.93
40	60 (min)	292	-25/-9/12	2.98
	70 (des)	330	-17/ 1/24	2.78
50	90 (min)	404	-28/-8/18	2.93
	110 (des)	477	-16/ 6/35	2.72
60	120 (min)	513	-12/11/42	2.66
	160 (des)	657	-23/ 3/38	2.77
70	150 (min)	622	- 9/17/51	2.60
	220 (des)	871	-31/ 0/41	2.80

Notes: (1) Source: AASHTO (5)

(2) Assumes headlight height is 2.0 ft (0.6m), the specification

(3) The distance the stopping vehicle will travel beyond the "sighted" object in the roadway; a negative value indicates the vehicle will stop in front of the object.

(4) Values given are the maximum allowable perception-brake reaction times in order for vehicles to stop on sag vertical curves with the geometric characteristics called for in the design "K" values

The SI (metric) conversions are 1 mph = 1.61 km/h, 1 ft = 0.3 m, and 1 K(ft/pct) = 0.3 K(m/pct).

Table 25 lists the computed K values needed in order to provide adequate stopping sight distance on sag vertical curves. It is evident from this table that even at the specification for perception-brake reaction time, 2.5 seconds, several design K values could be safely reduced by 5 units. Five of the current design standards for K would require slight increases if a perception-brake reaction time of 2.8 seconds is deemed a more accurate specification for the driver characteristic.

Exclusion Analysis and Critique of Specification

Estimates based on aggregated simulation results indicate that perception-brake reaction time values of 2.8 to 3.2 seconds may be more appropriate for the 85th percentile driver than the current 2.5 second specification value. This apparent lack of adequate sight distance in the standard formulation has not resulted in an unsafe condition evidenced by a large number of accidents. Perhaps this seemingly contradictory set of circumstances can be explained in either of two ways (or both). First, the sight distance deficiencies may indeed exist on sag vertical curves but they may not result in an appreciable safety hazard because the stopping action called for in the formulation is such a rare occurrence. In other words, the presence of a 6 inch (0.15m) object in the roadway usually requires the driver of a vehicle to only maneuver to avoid the object, not to bring the vehicle to a stop prior to the object. Therefore in most cases, it would seem that the sight distance needed on sag vertical curves is only the distance required for the driver to perceive and react to the object and for the vehicle to be maneuvered (not stopped) to avoid the object. The second possible explanation is that the standard formulation is incorrect. The several elements within the standard formulation which stand out as potential sources of errors include:

- The 1 degree vertical divergence of the headlight beam--if the actual divergence is larger than 1 degree, a greater distance can be viewed;
- Braking distance--the actual braking distance for vehicles (including trucks) may be less than what is estimated by the formula: and
- Size of object--the standard formulation for sag vertical curves calculates sight distance to the roadway itself; if a 6 inch (0.15m) object is to fall within the light beam, sight distance will actually be shorter than the value computed from the standard formulation.

LATERAL CLEARANCE TO SIGHT OBSTRUCTIONS ON HORIZONTAL CIRCULAR CURVES

Sensitivity Analysis

Each of the design standard values for horizontal circular curves (middle ordinate distance, degree of curvature, and radius of curve) can be expressed as a function of sight distance. The rest of this section focuses on stopping sight distance which in turn is a function of the driver characteristic perception-reaction time. Thus the sensitivity of the design standards to changes in the driver characteristic specification can be calculated. As noted earlier, provision of passing sight distance on horizontal circular curves is usually an impractical application and, therefore, will not be analyzed any further in this section.

The direct sensitivity of middle ordinate distance to changes in perception-reaction time for each design speed at its maximum design degree of curvature value can be calculated from equation (17). For curves with the minimum stopping sight distance, the instantaneous rate of change in the middle ordinate distance per one second change in perception-reaction time ranges between 7.66 and 8.67 feet/second (2.33 m/sec and 2.64 m/sec). For curves with the desirable stopping sight distance the values range from 9.03 to 13.0 feet/second (2.75 m/sec to 3.96 m/sec).

The percentage change in middle ordinate distance as a result of change in perception-reaction time ranges between 44 and 24 percent per one second change in perception-reaction time (refer to Table 26). Thus for a driver with a perception-reaction time of 3.2 seconds (upper limit of 85th percentile estimate) the percentage change in design values for middle ordinate distance needed in order for the driver to have sufficient stopping sight distance ranges between approximately 28 percent at 30 mph (48 km/h) and 15 percent at 70 mph (113 km/h). Both of these sensitivities are certainly significant. Efforts should be taken to verify a tighter range of driver characteristic values than the 2.3-3.2 second spread presented earlier.

Exclusion Analysis

In an earlier chapter it was noted that the actual coefficient of braking friction on horizontal curves is sometimes

TABLE 25--Design Sag Vertical Curve "K" Values for Various Perception-Brake Reaction Times

DESIGN SPEED (mph)	DESIGN K[1]	COMPUTED "K" VALUE[2]/(PERCENT INCREASE ABOVE STANDARD) @RT = 2.3 sec	@RT = 2.5 sec Specification	@RT = 2.8 sec	@RT = 3.2 sec
30	40 (min)	28.8 (-28)	30.8 (-23)	33.8 (-16)	37.9 (-5.2)
	40 (des)	33.1 (-17)	35.3 (-11)	38.6 (-3.5)	43.0 (7.5)
40	60 (min)	50.7 (-16)	53.4 (-11)	57.6 (-4.0)	63.1 (5.2)
	70 (des)	62.5 (-11)	65.6 (-6.3)	70.3 (0.4)	76.6 (9.4)
50	90 (min)	79.0 (-12)	82.5 (-8.3)	87.7 (-2.6)	94.8 (5.3)
	110 (des)	102 (-7.7)	106 (-3.6)	112 (1.8)	120 (8.8)
60	120 (min)	112 (-6.7)	117 (-2.5)	123 (2.5)	131 (9.2)
	160 (des)	149 (-6.9)	153 (-4.4)	161 (0.6)	171 (6.9)
70	150 (min)	143 (-4.7)	148 (-1.3)	155 (3.3)	164 (9.5)
	220 (des)	205 (-6.8)	211 (-4.1)	220 (0)	232 (5.5)

NOTES: (1) The two values given are the "minimum" and "desirable" values for K; source: AASHTO (1)

(2) Assumes sight distance is less than curve length.

The SI (metric) conversions are 1 mph = 1.61 km/h and 1 K(ft/pct) = 0.3 K(m/pct).

TABLE 26-Percentage Change in Middle Ordinate Distance Per One Second Change in Perception-Reaction Time

Design Speed MPH	"Unrounded"(1) Stopping Sight Distance, ft.	Degree of(2) Curvature	Instantaneous Percentage(3) Change in Middle Ordinate Distance Per One Second Change in Perception-Reaction Time
30 mph	196 ft	24.75	44.2%
40	267 (min)	13.25	39.2
	313 (des)	13.25	37.1
50	376 (min)	8.25	34.1
	461 (des)	8.25	31.5
60	501 (min)	5.25	30.3
	634 (des)	5.25	27.6
70	613 (min)	3.50	27.7
	840 (des)	3.50	24.3

Notes: (1) Unrounded AASHTO design values

(2) Maximum value (rounded) when superelevation is 0.10.

(3) Computed using the formula

$$(\partial m/\partial RT)(1/m) = (VD/7814)(\cot(SD/22920))$$

The SI (metric) conversions are 1 mph = 1.61 km/h, 1 ft = 0.3 m and 1 ft/sec = 0.3 m/sec.

less than the coefficient of braking friction observed on straight sections. Therefore, the vehicle braking distance is sometimes greater on horizontal curves than on straight sections. Because the AASHTO stopping sight distance design values are based on straight section braking, a driver with a perception-reaction time of 2.5 seconds (the specification) may not have sufficient braking distance on horizontal curves. Listed in Table 27 are the maximum allowable perception-reaction time values on horizontal curves designed to AASHTO stopping sight distance standards. The table shows that on horizontal curves designed to their maximum degree of curvature, none of the "desirable" design values nor the "minimum" design value for a 50 mph (80 km/h) design speed permit the specification for perception-reaction time, 2.5 seconds. Even if the lowest value of the estimated range of 85th percentile values (2.3 seconds) is taken, the "desirable" stopping sight

TABLE 27--Maximum Allowable Perception-Reaction Time on Horizontal Circular Curves

DESIGN SPEED, mph	DESIGN STOPPING SIGHT DISTANCE, ft (1)	DEGREE OF CURVATURE (2)	FRICTION FACTOR (3)		MAXIMUM ALLOWABLE PERCEPTION-REACTION TIME, sec (4)
			DESIGN	ACTUAL	
30	200	9.55	.35	.350	2.60
		24.75		.312	2.36
40	275 (min)	6.63	.32	.320	2.65
		13.25		.304	2.52
	325 (des)	5.37	.32	.320	2.70
		13.25		.284	2.34
50	375 (min)	4.44	.30	.300	2.48
		8.25		.287	2.33
	475 (des)	3.44	.30	.300	2.69
		8.25		.265	2.19
60	525 (min)	3.18	.29	.290	2.81
		5.25		.283	2.71
	650 (des)	2.39	.29	.290	2.68
		5.25		.264	2.22
70	625 (min)	2.55	.28	.280	2.64
		3.5		.278	2.61
	850 (des)	1.75	.28	.280	2.60
		3.5		.262	2.21

Notes: (1) AASHTO "rounded" design values

(2) For each design stopping sight distance, two values for degree of curvature are given. The top value corresponds to a curve where the actual friction factor equals the design friction factor; for smaller values, the friction factor (and thus the perception-reaction time) remains constant. The bottom value is the maximum permissable value when the superelevation is 0.10.

(3) Coefficient of friction available for braking.

(4) Maximum allowable perception-reaction time given the design stopping sight distance, the design speed, and actual coefficient of friction; computed by the formula:

$$RT = (S-V^2/30f)/(1.47V)$$

Perception-reaction time values are plotted in Figures 7 and 8.

The SI (metric) conversions are 1 mph = 1.61 km/h and 1 ft = 0.3 m.

distance for 50, 60, and 70 mph (80, 97, 113 km/h) design speeds are not sufficient. Therefore, significant portions of the driving population are being excluded by the current design standards.

Critique of Specification

The above calculations of maximum allowable perception-reaction times indicate that current design standards do not accommodate the current design specification for perception-reaction time. However, if the specification can be shown to be too high for the design driver, the current design standards may in fact be less inadequate, or even adequate. In order to make this assessment, it will be necessary to conduct further research to define a tighter range of estimated driver characteristic values. If this additional research confirms the validity of the current 2.5 second specification, the current design standards will need to be modified accordingly.

HORIZONTAL CURVATURE

Sensitivity Analysis

As discussed earlier the side function factor, f, which is used in equation (16) to determine maximum radius of curve, is being considered as the surrogate for the driver characteristic of driver comfort. From the basic equation (16) the sensitivity index can be determined from the following equation

$$\frac{\frac{\partial(R)}{\partial(f)}}{R}(f) = \frac{-f}{e+f} \qquad (34)$$

However, in this situation the partial derivative of R with respect to f is a function of f, and provides an instantaneous rate of change that may not hold true over the range in which f is varied. The values derivable from equation (34) would only be approximations. Alternatively, the sensitivity can be determined by substituting values of R + Δ R and f + Δ f in equation (16), where Δ implies an incremental increase or decrease and solve for ΔR. The resulting equation simplifies to:

$$R = \frac{-R\Delta f}{e+f+\Delta f} \qquad (35)$$

The equation above can be solved for various increments of Δ f, given values of e, f, and R.

The results of the application of this formula can be seen in Table 28. For the two most common maximum superelevation values 0.08 and 0.10 and for a low design speed of 30 mph (48 km/h) and a high speed of 70 mph (113 km/h). Changes in the minimum radius are determined for ±10 percent changes in the side friction factor. The radii for design values are those calculated from the basic horizontal curvature equation and are not the rounded values for design.

The results are also displayed in Figure 10 which shows the percent change in the minimum radius as a function of a percent change in the side friction factor. It is evident from the figure that:

- As f decreases for a given value of e, R increases non-linearly. That is, the radius, R, becomes more sensitive to a change in side friction factor, f, with increasing values of f.

- Conversely as f increases for a given value of e, R decreases non-linearly. That is, R becomes less sensitive to a change in f with ever increasing values of f.

- Sensitivity of R with respect to a change in f increases with decreasing values of e.

Consequently, the sensitivity of the standard, i.e., horizontal curvature as expressed by minimum radius, must be stated in terms of its value of design speed, superelevation and side friction factor. For example, in the case of V = 70 mph (113 km/h), e = 0.08 and f = 0.10, a ±10 percent change in the side friction factor, f, would result in a 5.2 percent decrease in the minimum radius (for +10 percent) and a 6.0 percent increase in the minimum radius for a -10 percent). An extreme change of ±50 percent would yield corresponding change of -21.7 percent and +38.5 percent. Even the lower values are appreciable changes and would have significant economic consequences when evaluated on the basis of construction costs.

Exclusion Analysis

The next question to resolve is how those motorist who are being excluded by the current driver characteristic specification are affected. Answering this question assumes that we know the driving population distribution of the characteristic and how the current specification relates to this distribution. Neither of these are valid assumptions.

As argued earlier, the driver characteristic involved here is the notion of driver comfort. Since this is not easily

TABLE 28--Changes in Minimum Radius for Changes in Side Friction for Two Value of e and v

Design Values	% Δ from f	New f Value	New Radius (ft)	% Δ from Standard	Design Values	% Δ from f	New f Value	New Radius (ft)	% Δ from Standard
e = 0.08	+50	0.240	188	-24.8	e = 0.10	+50	0.240	176	-23.8
R = 250 ft*	+40	0.224	197	-21.2	R = 231*	+40	0.224	185	-19.9
f = 0.16	+30	0.208	208	-16.8	f = 0.16	+30	0.208	195	-15.6
V = 30 mph	+20	0.192	221	-11.6	V = 30	+20	0.192	205	-11.3
	+10	0.176	234	- 6.4		+10	0.176	217	- 6.1
	-10	0.144	268	+ 7.2		-10	0.144	264	6.5
	-20	0.128	288	+15.2		-20	0.128	263	13.9
	-30	0.112	313	+25.2		-30	0.112	283	22.5
	-40	0.096	341	+36.4		-40	0.096	306	32.5
	-50	0.080	375	+50.0		-50	0.080	333	44.2
e = 0.08	+50	.15	1420	-21.7	e = 0.10	+50	.15	1307	-20.0
R = 1814*	+40	.14	1485	-18.1	R = 1633I	+40	.14	1361	-16.7
f = 0.10	+30	.13	1556	-14.2	f = 0.10	+30	.13	1420	-13.0
V = 70 mph	+20	.12	1633	-10.0	V = 70 mph	+20	.12	1485	- 9.1
	+10	.11	1719	- 5.2		+10	.11	1556	- 4.7
	-10	.09	1922	6.0		-10	.09	1719	5.3
	-20	.08	2042	12.6		-20	.08	1815	11.1
	-30	.07	2178	20.1		-30	.07	1922	17.7
	-40	.06	2333	28.6		-40	.06	2042	25.0
	-50	.05	2513	38.5		-50	.05	2178	33.4

*Radius are calculated values from equation $R = \dfrac{V^2}{15(e+f)}$

The SI (metric) conversions are 1 ft = 0.3 m and 1 mph = 1.61 km/h.

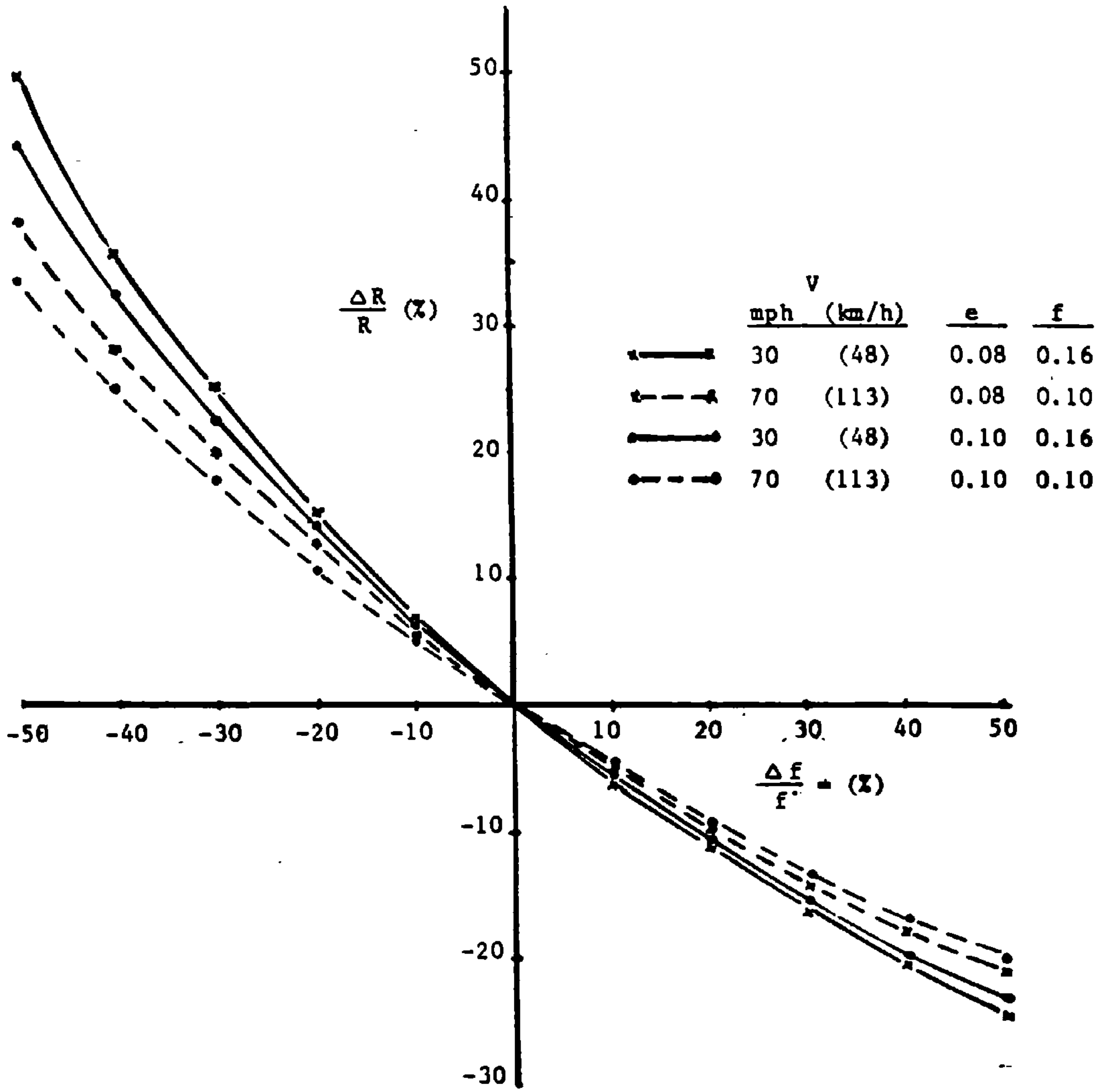


FIGURE 10--Percent Change in Minimum Radius as Function of Percent Change in Side Friction Factor

measured we have selected side friction factor, f, as its surrogate. The side friction factors that are specified in the horizontal curve formulae were, presumably, selected as those "...at which the centrifugal force is sufficient to cause the driver to experience a feeling of discomfort..." Whether these values are for all drivers, 85 percent, or the average driver is not known. Since this feeling of discomfort is likely to vary among drivers and for any one driver with different vehicles, it is difficult to state who, if any, are being excluded under the current specification.

Critique of Specification

For reasons explained earlier, side friction factor is being evaluated as the driver characteristic. The specifications for side friction factors vary from 0.10 to 0.17 depending upon design speed.

It is repeated here that the underlying assumption for selection of the current side friction factors is that they are related to the motorists threshold of discomfort. Since we do not know if these side friction factors values used are applicable for current vehicles, it cannot be stated whether the specifications are too stringent or too unconstrained. Arguments have been presented earlier for both cases.

It is apparent that horizontal curvature standards are sensitive to a change in the side friction factor. At least sufficiently so to warrant further research into evaluating the relationship of side friction factor, driver comfort and minimum radius. In this regard, it is recognized that FHWA has initiated a research project entitled "Side Friction for Superelevation on Horizontal Curves" which should resolve this issue.

SIGHT DISTANCE MEASURING CRITERIA

The sensitivity of sight distance to the driver characteristic, eye height, has been discussed previously under the heading Crest Vertical Curve Length. That sensitivity analysis deals strictly with the issue of whether or not a driver's physical line of sight to an object is obstructed. Of equal importance is whether or not the driver's visual capabilities enable the driver to actually perceive an object within the field of view ahead. The following analyses address this issue for both day and night conditions.

Daytime Visibility Conditions

A series of calculations were made for this study which estimate distances at which objects become visible to a driver of a vehicle. The calculations are based on two types of objects viewed against a pavement of average reflectance (0.15). The first object has reflectance of 0.60 which corresponds roughly to a whitish rock or a light grey coat. The second object has a reflectance of 0.20, something like a dark rock, a dark grey coat or a tire. The calculations ignore several real-world factors which introduce too many complications for a study of this scope, such as: atmospheric attenuation, inclement weather, and lighting geometry. The visual acuity distribution developed previously in this Chapter III was used in the calculations.

The following is a brief summary of how the visibility calculations can be applied in the case of two different sight distance standards--stopping and passing.

Table 29 lists the computed values at the AASHTO design stopping sight distances. The target sizes which can be detected by the median (50th percentile) driver range from 0.35 inches (9 mm) for a high-contrast target at 200 feet (61 m) to 4.04 inches (103 mm) for a low-contrast target at 850 feet (259m). For the 85th percentile driver, these values range from 0.39 inches (10 mm) to 4.98 inches (126 mm); and for the 95th percentile driver, the limits are 0.43 inches (11 mm) and 5.49 inches (139 mm). In other words, at the desirable design stopping sight distance for a 70 mph (112 km/h) design speed, 850 feet (259m), the 95th percentile driver can detect low-contrast targets which are 5.49 inches (139 mm) high or larger.

Passing sight distance standards are significantly greater than those for stopping sight distance. As was described in detail earlier under Passing Sight Distance, both the MUTCD (3) and AASHTO (5) recommend passing sight distance standards and these standards differ from each other. Table 30 lists the computed minimum detectable object heights at both the MUTCD and AASHTO passing sight distances. These values range to as high as 16.2 inches (0.41m) for the 70 mph (113 km/h) driver attempting to sight a low-contrast target at the AASHTO passing sight distance.

TABLE 29--Sizes of Objects Which Can be Sighted at the Design Stopping Sight Distances

Design Speed, mph	Design Stopping Sight Distance, ft	Diameter of Circular Object Which Can be Sighted at Stopping Sighting Distance by Various Percentile Drivers, in: High-Contrast Target			Low-Contrast Target		
		50th %ile	85th %ile	95th %ile	50th %ile	85th %ile	95th %ile
30	200	0.35	0.39	0.43	0.95	1.17	1.29
40	275 (min)	0.48	0.53	0.59	1.31	1.61	1.78
	325 (des)	0.56	0.65	0.69	1.54	1.90	2.10
50	400 (min)	0.69	0.78	0.85	1.90	2.34	2.58
	475 (des)	0.82	0.92	1.01	2.26	2.78	3.07
60	525 (min)	0.91	1.02	1.12	2.49	3.08	3.39
	650 (des)	1.12	1.26	1.38	3.09	3.81	4.20
70	625 (min)	1.08	1.21	1.33	2.97	3.66	4.04
	850 (des)	1.47	1.65	1.81	4.04	4.98	5.49

Note: The actual contract for the high-contrast target is 3.0 and for the low-contrast target is 0.3 where contrast = difference of the reflectance of the target and road divided by the reflectance of the road.

The SI (metric) conversions are 1 mph = 1.61 km/h, 1 ft = 0.3 m, and 1 in = 25.4 mm.

TABLE 30 --Sizes of Objects Which Can be Sighted at the Passing Sight Distances

Design Speed, mph	Passing Sight Distance, ft	Diameter of Circular Object Which Can be Sighted at Passing Sight Distance by Various Percentile Drivers, in					
		High-Contrast Target			Low-Contrast Target		
		50th %ile	85th %ile	95th %ile	50th %ile	85th %ile	95th %ile
MUTCD							
30	500	0.9	1.0	1.1	2.4	2.9	3.2
40	600	1.0	1.2	1.3	2.8	3.5	3.9
50	800	1.4	1.6	1.7	3.8	4.7	5.2
60	1000	1.7	1.9	2.1	4.8	5.9	6.5
70	1200	2.1	2.3	2.6	5.7	7.0	7.8
AASHTO							
30	1100	1.9	2.1	2.3	5.2	6.4	7.1
40	1500	2.6	2.9	3.2	7.1	8.8	9.7
50	1800	3.1	3.5	3.8	8.6	10.5	11.6
60	2100	3.6	4.1	4.5	10.0	12.3	13.6
70	2500	4.3	4.8	5.3	11.9	14.6	16.2

Note: The SI (metric) conversions are 1 mph = 1.61 km/h, 1 ft = 0.3 m, and 1 in = 25.4 mm.

Limited Visibility Conditions

Any design standard that involves driver vision will be affected by limited visibility conditions, i.e., nighttime1/. The degree of luminance of the object to be sighted and its contrast with its background. For example, a self-illuminated object such as an automobile with headlights turned on can usually be perceived at a greater distance during limited visibility conditions than a non-illuminated vehicle during the daylight. However, there are indications in the literature that other elements of the perception-reaction time driver characteristic may be adversely affected. For instance, judgment of the speed of an oncoming vehicle (auto or train) is made more difficult when the sole reference for this judgment is the movement of a headlight within an otherwise uniform dark background. This is because of the lack of other visual cues which would provide a reference point to judge the speed.

The analysis of the effect of limited visibility conditions on geometric design criteria does not cover standards whose object is self-illuminated because it is assumed that the current standards are sufficient. Instead, the analysis focuses on those geometric design standards whose objects are not self-illuminated such as pedestrian or debris in the roadway. Basically, these standards are all those which are a function of stopping sight distance.

A number of researchers have measured visibility distances to pedestrians during nighttime conditions using standard U.S. low beams, with no fixed ambient lighting, and with no oncoming glare. Despite these commonalities, the various test procedures used by the researchers introduced relatively significant differences in what the researchers were actually measuring. For example, in only two of the 14 research efforts reviewed were the drivers/observers "unalerted" to the possibility of a pedestrian appearing in the path of the vehicle. In each of the other 12 field tests, the drivers were aware that a pedestrian (or a simulated pedestrian) probably would appear and were thus "alerted" to that possibility. Field tests show that the average driver perceives an unexpected obstacle only half as far as from an expected obstacle (88).

The reported visibility distances from 14 separate research efforts are plotted versus pedestrian reflectance in Figure 11. The figure includes distances for both high and low beam observations.

Despite mixing the results of somewhat-dissimilar field tests, the plot does exhibit a definite trend for the upper values of "low beam" values. It should be noted that the "unalerted driver" tests conducted by Roper and Howard (88) fall within the shaded "approximate maximum range" of values. Granted, this approach for estimating maximum visibility distances falls short scientifically of a step-by-step modelling procedure which reflects all the driver, vehicle, and environment elements which affect the visibility distance. The Highway Safety Research Institute (89) and Ford Motor Company (90) have indeed developed quite refined models for estimating visibility distances. However, these models are calibrated from some of the field test results presented in the graph (rather than from theoretical or laboratory test results). In fact both models project values which fall within or just below the shaded maximum range.

Farber (90) measured typical reflectances for pedestrians and found that the median pedestrian has 8 percent reflectance, the 85th percentile has 2.5 percent reflectance, and the 15th percentile has 25 percent reflectance. These reflectances are plotted in Figure 12 along with the estimated "maximum range of visibility distances" and current design standards for stopping sight distance. Based on this comparison it appears that the 85th percentile pedestrian can be seen at best at approximately the 40 mph (64 km/h) stopping sight distance. Drivers travelling faster than 40 mph (64 km/h) will probably not be able to see the 85th percentile pedestrian in time enough to stop. In the case of the median pedestrian, sight distance is still not much higher than the 40 mph (64 km/h) design standard.

The presence of oncoming glare inhibits the driver's ability to see a target. Figure 13 is a plot of field test visibility distances in the presence of oncoming glare as reported in 10 research efforts. As expected, the "glare" visibility distances are lower than the "no glare" values. Not even at higher pedestrian reflectances is the 40 mph (64 km/h) design distance (325 feet, 99m) satisfied; and in the range of 2.5 percent reflectances (the 85th percentile value), 30 mph (48 km/h) travel speed may not even be slow enough in the presence of oncoming glare.

1/Visibility can also be limited by fog, rain, snow, etc. However, these conditions are not to be addressed here.

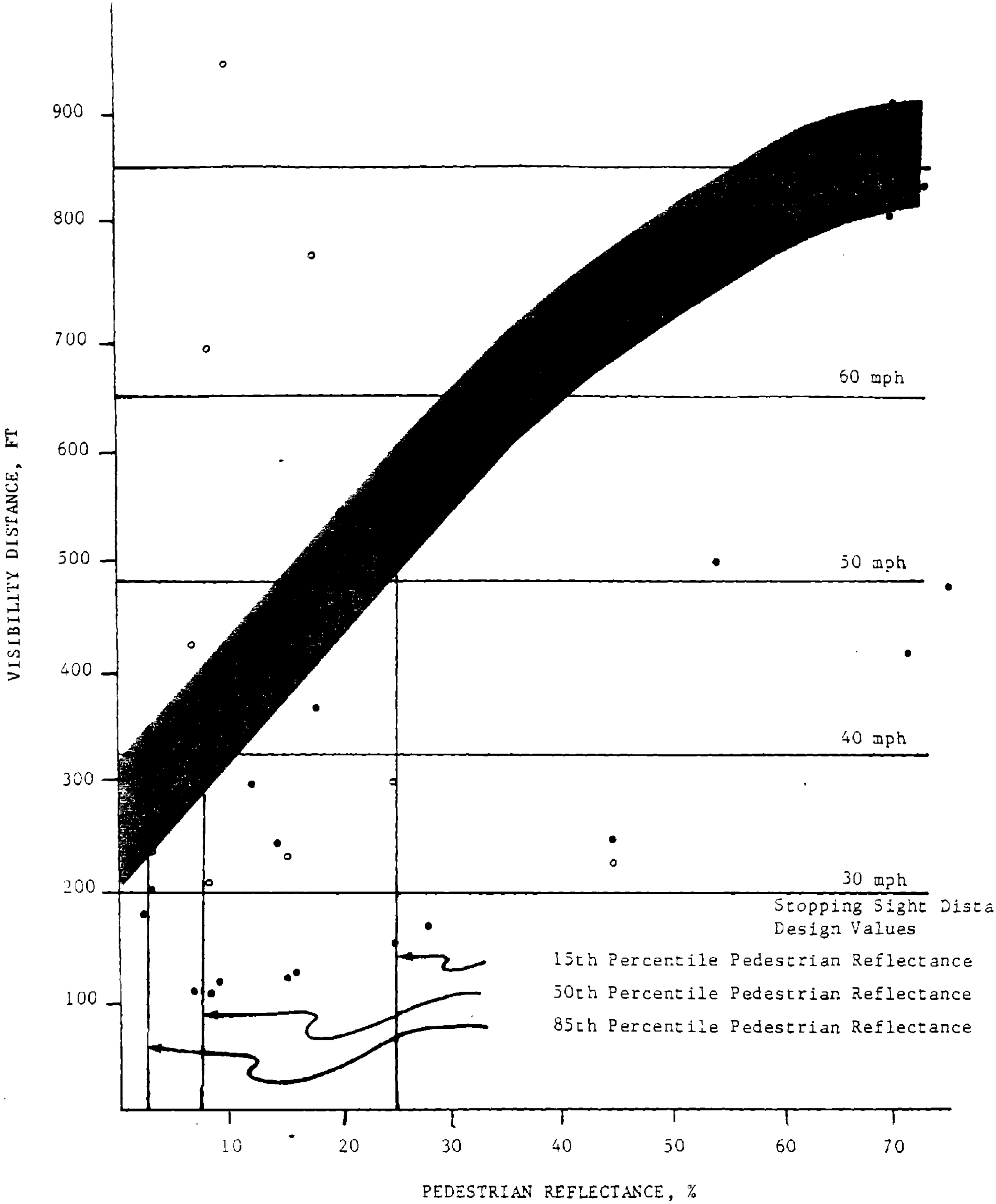


FIGURE 11--Approximate Maximum Visibility Distances to Pedestrians During Nighttime Conditions Based on Actual Pedestrian Reflectance Distribtuions

The SI (metric) conversions are 1 ft = 0.3 m.

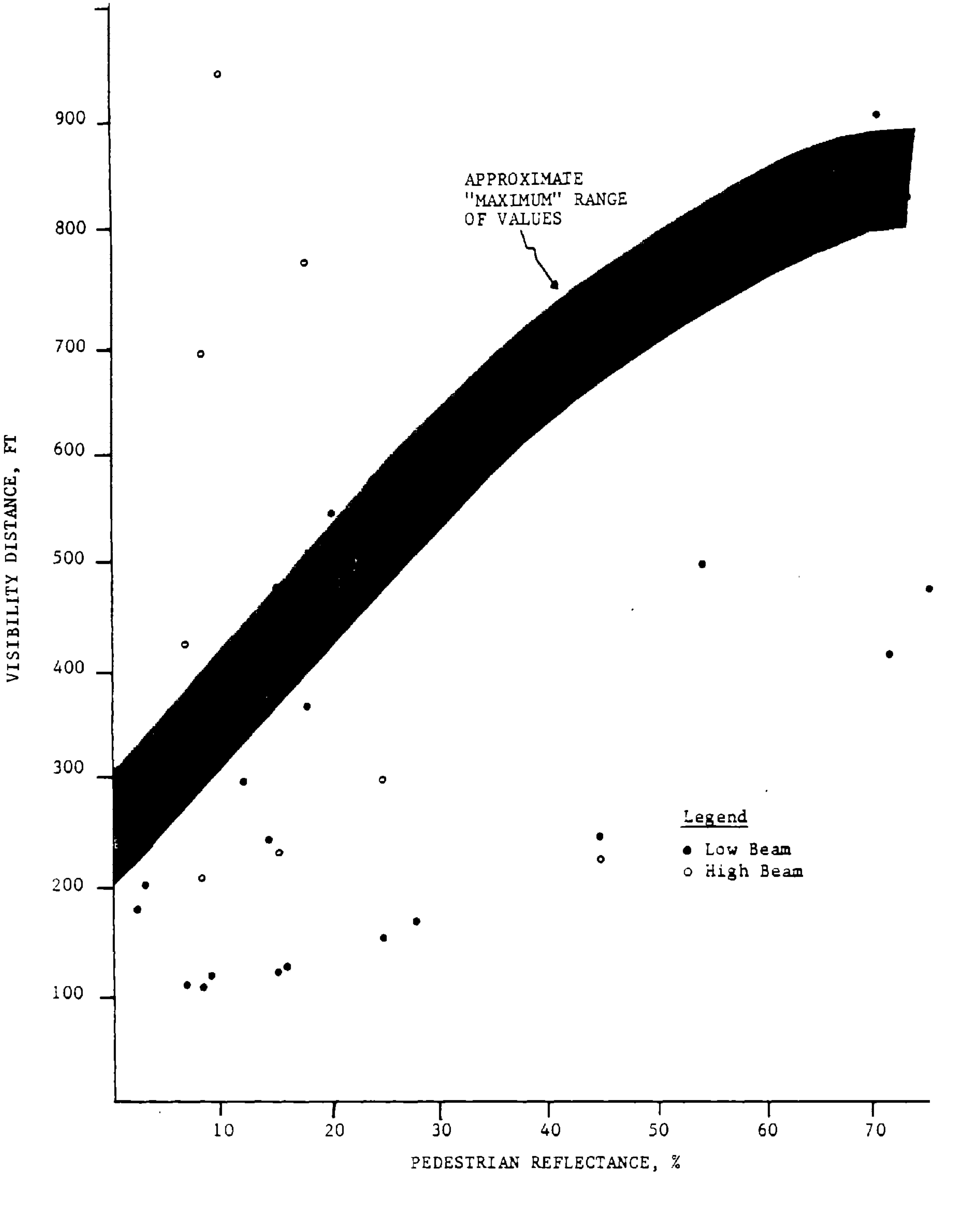


FIGURE 12--Maximum Visibility Distances to Pedestrians Under Nighttime Conditions--No Fixed Ambient Lighting, No Oncoming Glare

The SI (metric) conversions are 1 ft = 0.3 m.

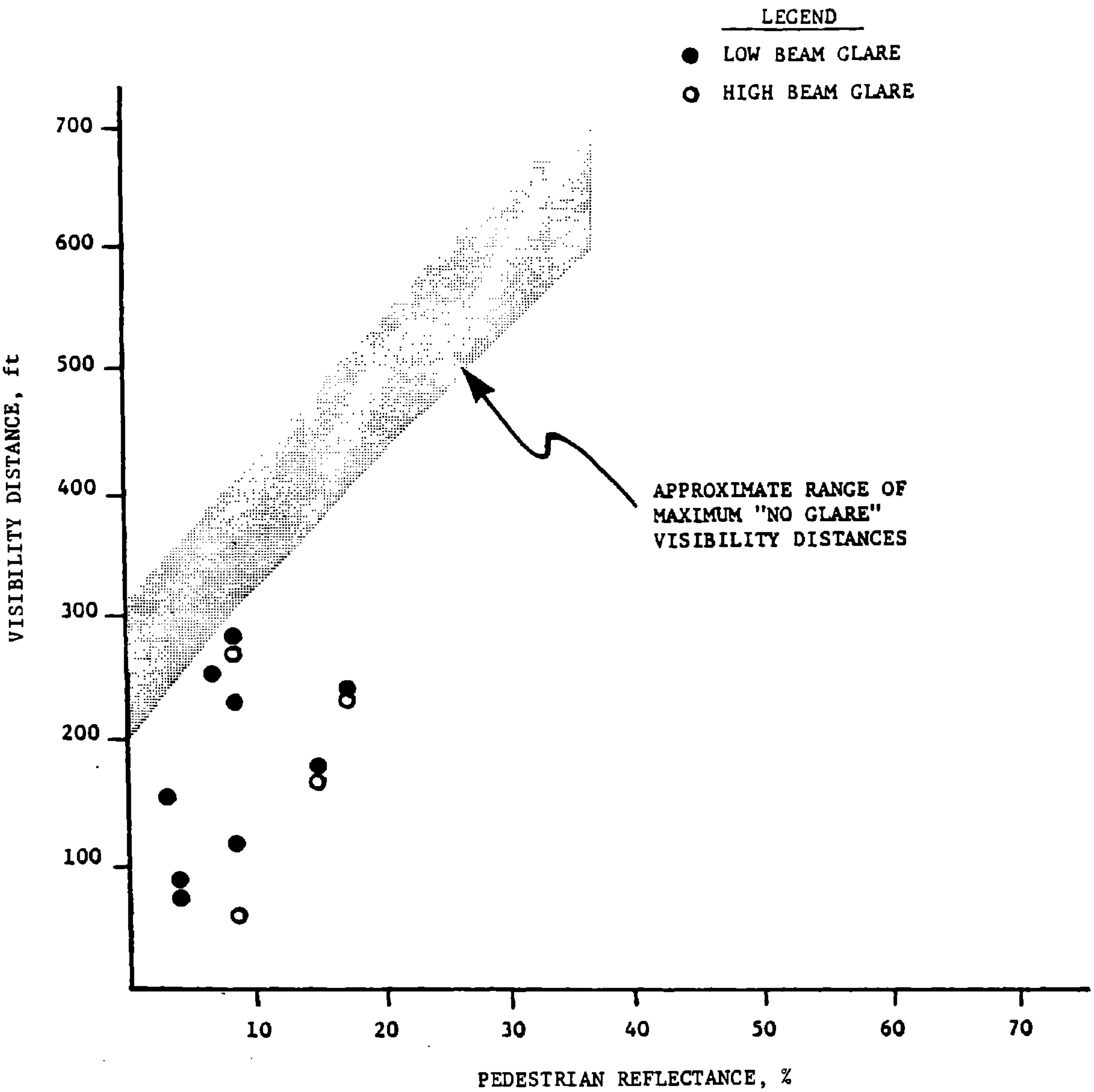


FIGURE 13 --Comparison of Observed "Oncoming Glare" Visibility Distances to Observed "No Glare" Visibility Distances

The SI (metric) conversions are 1 ft = 0.3 m.

In summary, review of over 28 research efforts which produced nighttime visibility distance measurements indicates as expected, that sight distance to non-self-illuminated objects is reduced at night from daylight sight distances. Most of the researched field tests represent optimum conditions--no oncoming glare; a driver/observer alerted to the vehicle's path during the test; flat, tangent roadway section; and a functionally-standard headlamp system. Despite these optimum conditions, the field tests suggest that it is not possible to adequately sight objects beyond approximately the 50 mph (80 km/h) desirable stopping sight distance standard. And in fact, once some real world factors (like glare, curves, misaimed headlamps) are introduced, speeds of 30 and 40 mph (48 and 64 km/h) become more and more often too fast for driver/vehicle capabilities.

There are basically three optional methods for improving nighttime visibility distances: change the environmental characteristics, change the vehicle characteristics, or change the driver characteristics.

One change to the environment which would enhance night visibility is the installation of fixed ambient illumination. Obviously, this would not be a cost-effective measure if applied universally but more could certainly be done and still be cost-effective. Another change to the night driving environment which would improve night visibility is a shift in the distribution of pedestrian reflectances to higher values. Pedestrians which are 25 percent reflective (light grey) can be seen approximatley 250-300 feet (75-91 m) further than a pedestrian wearing clothing at the current 85th percentile value of 2.5 percent. This represents an over 100 percent increase in visibility distance. A third means by which the environment could be changed to enhance night visibility is to design glare-reducing structures into highway facilities. On expressways, this action can be and has been accomplished by means of glare screens; however, on 2-lane or undivided arterials, glare reduction via changes in the highway system is a much difficult and costly endeavor.

The second major option for improving nighttime visibility distances is to change vehicle characteristics. One such method is to increase the headlamp intensity or redirect the headlamp aim to provide longer sight distance. The inherent problem to this approach is that it significantly increases glare degradation effects for opposing vehicles if the current headlamp system is used. An alternative approach is the polarized-headlamp system which underwent in-depth analysis a few years ago. However, full-scale practically eliminates any adverse effects from oncoming glare. However, full-scale implementation of a polarized headlamp system is a long way from reality due to stumbling blocks such as cost and transition-period safety concerns.

The third option for improving the safety of current nighttime visibility distances is to change the driver's characteristics. Basically, this can be accomplished by reducing driver perception-brake reaction time, perhaps by alerting drivers of the likelihood of an object (e.g., pedestrian) appearing in the path of the vehicle. The practicality of such a technique is, of course, quite limited in terms of warning motorists of all potential objects in the roadway. Another potential measure for improving nighttime safety is to reduce vehicle speeds to the point where vehicle stopping sight distance equals the actual nighttime visibility distance. However, only a limited amount of highway mileage has been observed to have day/night speed limits.

ADEQUATE GAP TIME FOR SCHOOL CROSSING TRAFFIC SIGNAL WARRANT

Sensitivity Analysis

The effect of a change in the pedestrian perception-reaction time on the adequate gap time is the subject of the first sensitivity analysis assuming all other factors are independent of the pedestrian perception-reaction time (e.g., they remain constant even if the specification of the perception-reaction time is changed), the sensitivity of the adequate time with respect to the pedestrian perception-reaction time is:

$$\frac{\text{Change in } G}{\text{Change in } t} = 1 \qquad (36)$$

where: G = Adequate gap time, sec

t = pedestrian perception-reaction time, sec

This means that the rate of change in the adequate gap time with respect to a change in the pedestrian perception-reaction time is constant. In simpler terms, a one second increase in the pedestrian perception-reaction time causes a one second increase in the adequate gap time. By dividing both sides of this equation by the adequate gap time, the fractional rate of change in the adequate gap time with respect to a change in the pedestrian perception-reaction time can be derived. In mathematical terms; that is:

$$\frac{\Delta G/\Delta t}{G} = \frac{1}{G} = \frac{1}{t + \frac{W}{V} + h(N-1)} \qquad (37)$$

where: W = width of the roadway, ft (m)

v = walking speed, fps (mph)

N = number of rows

h = time interval between rows, sec

The fractionsl rate of change in the adequate gap time with respect to a change in the pedestrian perception-reaction time is, therefore, a function of the current specifications of the pedestrian perception-reaction time and the walking speed, the width of the roadway, the number of pedestrians in the 85th percentile group, and the assumed time interval between rows of 5 pedestrians. This fractional rate of change is shown in Figure 14 as a function of the roadway width and the number of rows. The figure shows that the rate of change in the adequate gap time with respect to a change in the current pedestrian perception-reaction time diminishes as the number of rows and/or roadway width increase. In other words, the effect of a change in the pedestrian perception-reaction time on the adequate gap time is less pronounced for wide roadways and/or intersections where many children cross.

The second analysis investigated the effect of a change in the walking speed on the adequate gap time. Assuming all other factors are independent of the walking speed, the sensitivity of the adequate gap time with respect to the walking speed is:

$$\frac{\text{Change in G}}{\text{Change in V}} = \frac{-W}{V^2} \qquad (38)$$

This means that the magnitude of the rate of change in the adequate gap time with respect to a change in the walking speed is not constant but increases as roadway width increases, and that change is in the negative direction. For example, a 1 fps (0.3 m/s) increase in the walking speed causes a 0.8 second decrease in the adequate gap time for a 12 foot (3.7 m) wide road and a 5.3 second decrease in the adequate gap time for an 84 foot (25.6 m) wide road. By dividing the above equation by the adequate gap time, the fractional rate of change in the adequate gap time caused by a change in the walking speed can be derived. In mathematical terms that is:

$$\frac{\partial G/\partial v}{G} = \frac{-W}{Gv^2} = \frac{-W}{[t + h(N-1)]\, v^2 + Wv} \qquad (39)$$

The fractional rate of change in the adequate gap time with respect to a change in the walking speed is, therefore, a function of the current specifications in walking speed, the pedestrian perception-reaction time and the time interval between rows, the width of the roadway, and the number of rows. The fractional rate of change in the adequate gap time with respect to a change in the current walking speed is shown in Figure 15 as a function of the roadway width and the number of rows. This figure shows that the rate of change in the adequate gap time with respect to a change in the current walking speed diminishes as the number of rows increases and/or the roadway width decreases. In other words, the effect of a change in the walking speed on the adequate gap time is less pronounced for narrow roadways and/or intersections where many children cross.

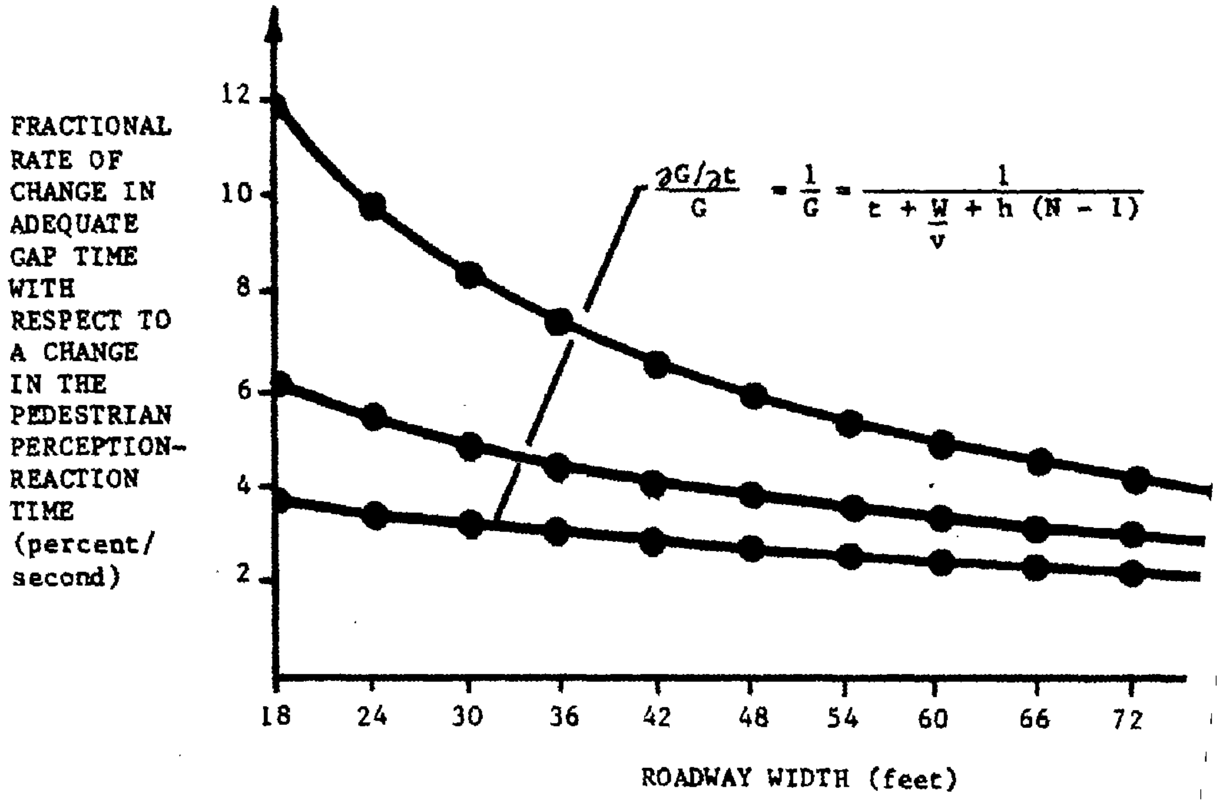
The third analysis investigated changes in both the pedestrian perception-reaction time and the walking speed. The question that this analysis sought to answer was simply, "In order for the adequate gap time to remain unchanged, by how much does the pedestrian perception-reaction time have to be changed if the walking speed is changed?" If the adequate gap time is assumed to be constant and, for this analysis, independent of the walking speed and the pedestrian perception-reaction time, then the pedestrian perception-reaction time can be expressed as the following:

$$t = G - \frac{W}{V} - h(N-1) \qquad (40)$$

The derivative, then, of the pedestrian perception-reaction time with respect to the walking speed is:

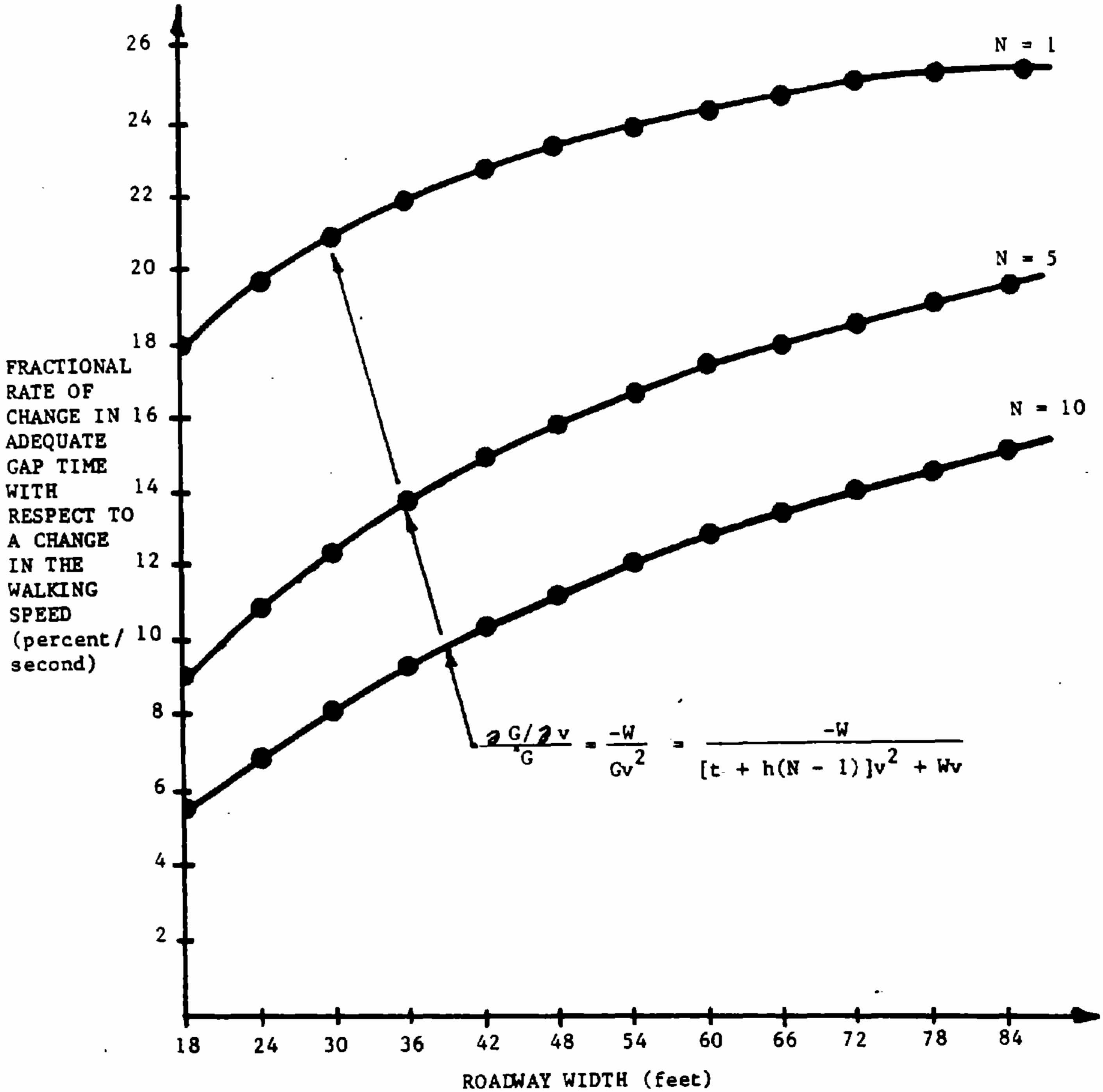
$$\frac{\partial t}{\partial v} = \frac{W}{v^2} \qquad (41)$$

The magnitude of the rate of change in the pedestrian perception-reaction time with respect to a change in the walking speed is equal to the rate of change in the adequate gap time with respect to a change in the walking speed. However, the change in the pedestrian perception-reaction time is in the positive direction. This means that a 1 fps (0.3 m/s) increase in the walking speed results in a 0.8 second increase in the pedestrian perception-reaction time for a 12 foot



The SI (metric) conversion is 1 ft = 0.3 m.

FIGURE 14--Functional Rate of Change in Adequate G
with Respect to a Change in the Pedestr
Perception-Reaction Time as a Function
Number of Rows and Width



The SI (metric) conversion is 1 ft = 0.3 m.

FIGURE 15--Fractional Change in Adequate Gap Time with Respect to a Change in the Walking Speed as a Function of the Number of Rows and Roadway Width

(3.7 m) wide road and a 5.3 second increase in the pedestrian perception-reaction time for a 84 foot (25.6 m) wide road. By dividing the derivative by the pedestrian perception-reaction time, the fractional rate of change on the pedestrian perception-reaction time with respect to a change in the walking speed can be derived. In mathematical terms, that is:

$$\frac{\frac{\partial t}{\partial v}}{t} = \frac{W}{tv^2} \qquad (\underline{42})$$

The fractional rate of change of the pedestrian perception-reaction time is, therefore, a function of the width of the roadway, the current specification of the pedestrian perception-reaction time, and the walking speed and is independent of the number of pedestrians in the 85th percentile group. Figure 16 shows that the rate of change in the pedestrian perception-reaction time with respect to a change in the current walking speed greatly increases at a constant rate as the roadway width increases. As can be seen from Figure 16, if the specification of the walking speed is changed by only a small percentage, in order for the adequate gap time to remain constant, the specification of the pedestrian perception-reaction time must be changed by a much larger percentage. If the adequate gap time is to remain constant, then the pedestrian perception-reaction time is very sensitive to a change in the walking speed. Conversely, the walking speed is very insensitive to a change in the pedestrian perception-reaction time. Moreover, the adequate gap time is more sensitive to a change in the walking speed than a change in the pedestrian perception-reaction time.

Exclusion Analysis

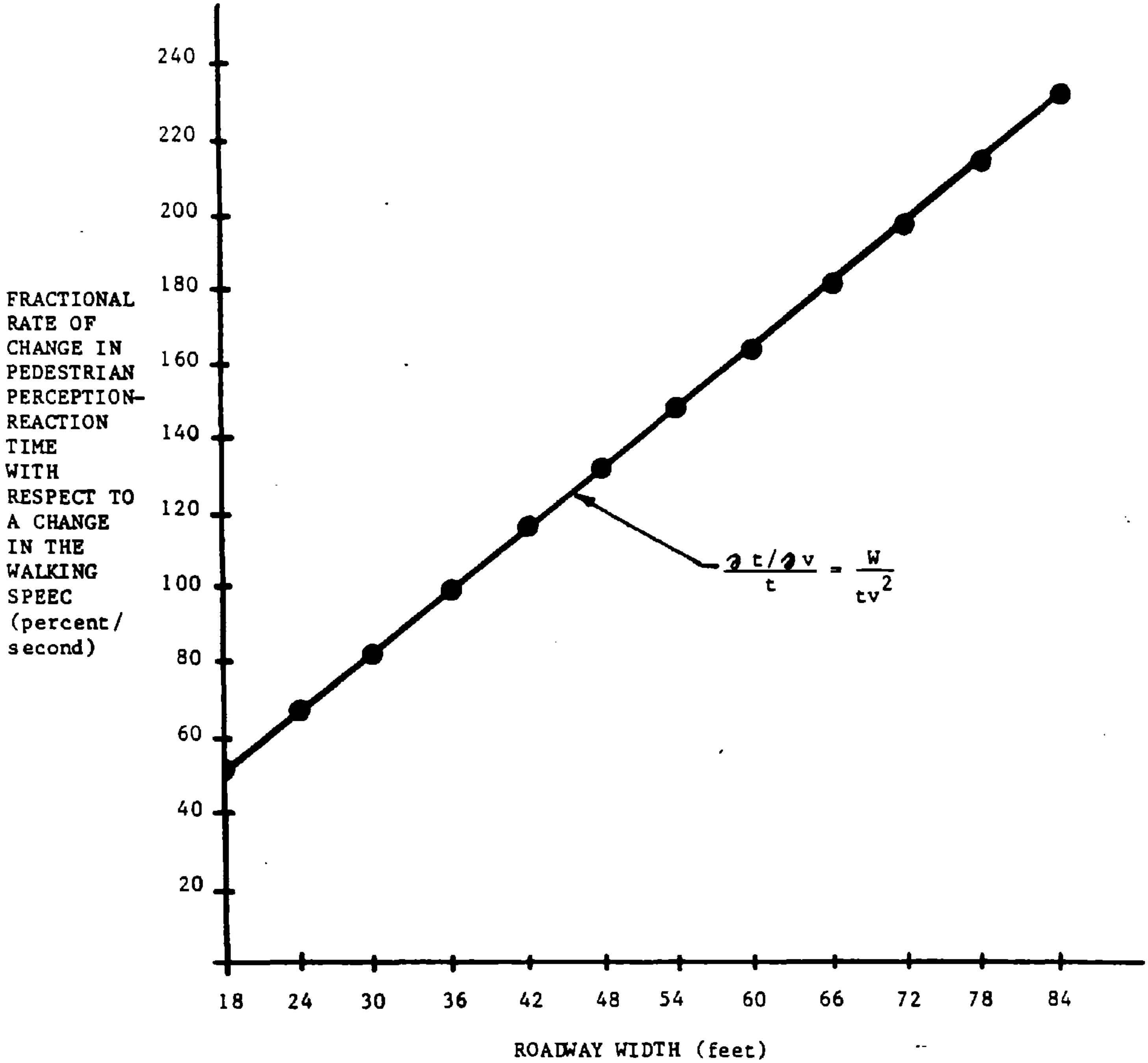
Without reliable data, it is not possible to estimate the percentage of school children having perception-reaction time longer than the current specification of 3.0 seconds. Consequently, it is not possible to assess the exclusion effects. It could be argued that younger school children, "special children", and handicapped school children may have perception-reaction times longer than 3.0 seconds. In which case, they may be excluded. Yet, there is no available data to substantiate that argument. Moreover, it is also likely that these children would be assisted in crossing the intersection.

The only available data on the walking speeds of children and/or students appear to indicate that children walk faster than adults (81), and the average "free flow" walking speed is greater than the current specification of 3.5 feet per second (1.07 m/s). One study (84) noted a "free flow" speed of 5.3 fps (1.6 m/s) and another study (81) found a median speed of 5.25 fps (1.6 m/s). This data would indicate that very few school children are excluded. However, not enough information (e.g., sample sizes, conditions existing during data collection, location and type of intersections analyzed, range in ages of children/students, etc.) is available about these studies to use them as a basis to formulate distribution profiles.

Reliable data was available for adults, though. The distribution developed from this data revealed that the current specification employed in the standard is the 88th percentile adult walking speed. Using this distribution for a conservative estimate of the walking speeds of school children, only 12 percent of the population of school children who cross at intersections would be excluded. This does not mean that 12 percent cannot cross the intersection safely but rather that the walking speeds of those school children are not considered in determining the adequate gap time. If these speeds were employed, the adequate gap times would be longer. Subsequently, more of the gaps measured in the field would be discarded, resulting in a higher calculated percent pedestrian delay time. The end result would be that more intersections would warrant some type of traffic control. By not designing for this 12 percent, there may be fewer intersections that warrant traffic control but more positive economies of scale. For example, the rate of return of the dollar invested for signal control will be much greater if designing for the 88th percentile rather than the 100th percentile.

Critique of the Specifications

As previously stated, there is no available data to compute any population distribution profile for the pedestrian perception-reaction time. Therefore, it is not possible to determine if the specification is too conservative or too liberal. Based on the findings related to the driver's perception-reaction time for the intersection sight distance standard, it would appear that the current specification of 3.0 seconds is a reasonable pedestrian perception-reaction time. Moreover, based on the sensitivity analysis, the adequate gap time is not very sensitive to the pedestrian perception-reaction time.



The SI (metric) conversion is 1 ft = 0.3 m

FIGURE 16--Fractional Change in the Pedestrian Perception-Reaction Time with Respect to a Change in the Walking Speed as a Function of the Roadway Width

The data on children/students' walking speeds is not reliable enough to serve as the basis for the distribution profile. The findings of one study (81) indicated that children walk faster than adults, although it is unknown under what conditions those results were determined. Using a distribution for adult walking speed, the present specification of 3.5 fps (1.07 m/s) is neither too conservative not too liberal. In fact, it closely approximates the 85th percentile adult walking speed. It should be noted, however, that the adequate gap time is somewhat sensitive to changes in the specification of the walking speed, especially at wide intersections.

VEHICLE CHANGE INTERVAL FOR TRAFFIC SIGNALS

Sensitivity Analysis

The sensitivity of both yellow change and minimum clearance intervals to changes in the driver characteristic perception-reaction time can be calculated by taking the first derivative of equations (19) and (20). For each, the direct sensitivity rate is 1. That is, if perception-reaction time is increased by 0.5 seconds, the yellow change interval (or the minimum clearance interval or the yellow clearance interval) is likewise increased by 0.5 seconds. The estimated 85th percentile value for perception-brake reaction time at yellow signal phase changes is 1.77 seconds. Therefore, in order for the 85th percentile driver to be accommodated, the minimum clearance intervals as contained in the Transportation and Traffic Engineering Handbook (16) need to be increased an increment of 0.77 seconds.

Regarding the driver characteristic, "desired" deceleration rate, it was shown previously that a rough estimate of the driver's desire limit is 9.68 ft/s^2 (2.95 m/s^2). Investigation into the effect of using a 9.68 ft/s^2 (2.95 m/s^2) deceleration rate rather than the current specification of 10 ft/s^2 (3.05 m/s^2) reveals that only very small incremental changes in the change interval result.

When the approach speed is 20 mph (32 km (32 km/h) a change in deceleration rate is 9.68 ft/s^2 (2.95 m/s^2) necessitates a 0.048 second increase in the standard clearance interval. At 60 mph (97 km/h), the effect is still only 0.145 second.

Exclusion Analysis

The current standards for clearance intervals are based on an assumed perception-reaction time 1.0 second and a deceleration rate of 10 ft/s^2 (3.05 m/s^2). The distribution of the perception-brake reaction time as developed earlier is estimated to have a median value of 1.23 seconds and an 85th percentile value of 1.77 seconds for the general driving population. If we can assume that this distribution is relatively accurate, then over one-half of the driving population is excluded by the current standard due to a deficiency in their perception-reaction time. It would be expected, therefore, that intersections with shorter yellow signal phases would have a greater number of vehicles entering the intersection on the red indication. The Wortman and Matthias (71) data confirms this hypothesis. At one intersection which permitted a perception-reaction time of 2.09 seconds (based on the mean speed of each of the last vehicles through the intersection) only 2.3 percent of these "last vehicles" entered the intersection on the red indication. At another intersection, the allowable PRT was 0.58 second and 8.3 percent entered on red. And finally an intersection which theoretically allowed a PRT of 0.08 second had a nearly 30 percent "enter-on-red" rate. Although this analysis is admittedly overly-simplistic, it nevertheless illustrates what intuitively should be expected to occur at signals with short (or long) yellow signal phase lengths.

Critique of Specification

Recent field tests and the laboratory simulations described earlier in this report confirm that the current 1.0 second specification for perception-reaction time does not accommodate the "slowest" 15 percent of the driving population. In fact, the field tests and simulation results indicate that the current specification may not even accommodate the "average" (mean or median) driver. In order to obtain a more definitive distribution of the driver characteristic than is currently available from the one field study by Wortman and Matthias (71) or from the aggregrated simulation results presented earlier in this chapter, it will be necessary to conduct extensive field tests. An upcoming FHWA research effort entitled "Engineering Factors Affecting Traffic Signal Yellow Time" should provide an adequate base from which to build an estimated population.

The current specification for deceleration rate is 10 ft/s^2 (3.05 m/s^2). An approximation of the driver's desired limit is roughly the same as the current specification. The characteristic deceleration rate, however, is both a driver and a vehicle; characteristic. A thorough analysis of vehicle deceleration rate capabilities is expected to be undertaken in the FHWA research effort "Vehicle Characteristics Affecting Highway Design".

PEDESTRIAN SIGNAL TIMING

Sensitivity Analysis

As has been shown by Abrams and Smith (72) the pedestrian WALK interval is more a function of the pedestrian queue than of an individual's perception-reaction time. Because the standard for the WALK interval (4 to 7 seconds) is well above the expected range of perception-reaction time values, the WALK interval provides sufficient time for all pedestrians to perceive the WALK signal and to react to it. The pedestrian's ability to reach the crosswalk during the WALK interval is constrained only by the impedance of other queued pedestrians. Therefore, an analysis of the sensitivity of the standard (pedestrian WALK interval) to the pedestrian characteristic (perception-reaction time) need not be developed.

With regard to the pedestrian clearance interval (PCI) standard, the instantaneous percentage change in PCI per one fps (mph) change in walking speed is simply the negative inverse of the walking speed. Therefore, at a walking speed of 3.0 fps (0.91 m/s) the instantaneous percentage change in PCI per 1 fps (0.3 m/s) change in walking speed is -33.3 percent. At a walking speed of 4.0 fps (1.2 m/s), the instantaneous percentage change is -25 percent.

It should be emphasized that the above sensitivity rates are instantaneous rates. The instantaneous rate per unit (e.g., 1 fps (0.3 m/s)) change in walking speed does not equal the actual incremental change if walking speed is changed 1 fps (0.3 m/s). The following sensitivity analyses deal strictly with incremental (rather than instantaneous) changes.

Earlier in this report, an estimated range of walking speed values was developed for various percentiles within the pedestrian population. These estimated walking values correspond to the following percentage changes from the current specification, 4.0 fps (1.2 m/s).

59th percentile = -2.5 to 12.5 percent

85th percentile = -15 to -10 percent

95th percentile = -25 to -22.5 percent

The effect of these percentage changes in the walking speed on the pedestrian clearance interval are as follows:

50th percentile = -11.1 to 2.6 percent change in PCI

85th percentile = 11.1 to 17.6 percent change in PCI

95th percentile = 29.0 to 33.3 percent change in PCI

It is clear from these calculations that use of the 85th percentile value estimates results in a need for relatively significant increases in the pedestrian clearance interval standard in order to accommodate the slower walking speeds.

The percentage incremental changes discussed above are strictly based on only the pedestrian clearance interval. It differs from the percentage change in the total cycle time available for a pedestrian crossing (clearance interval plus WALK interval). When the WALK interval is added, the relative effect of walking speed is reduced. And as street width increases, the sensitivity of the total cycle time to walking speed likewise increases. Based on assumed walking distances ranging between 30 and 90 feet (9.1 and 27.4 m).

- in order to accommodate the walking speed of the 50th percentile pedestrian, the current standard for total pedestrian cycle time needs to be changed between an increase of 2 percent to a decrease of 8 percent;
- in order to accommodate the 85th percentile pedestrian, total cycle time needs to be increased between 6 and 13 percent; and
- in order to accommodate the 95th percentile pedestrian, total cycle time needs to be increased between 15 and 25 percent.

EXCLUSION ANALYSIS AND CRITIQUE OF SPECIFICATION

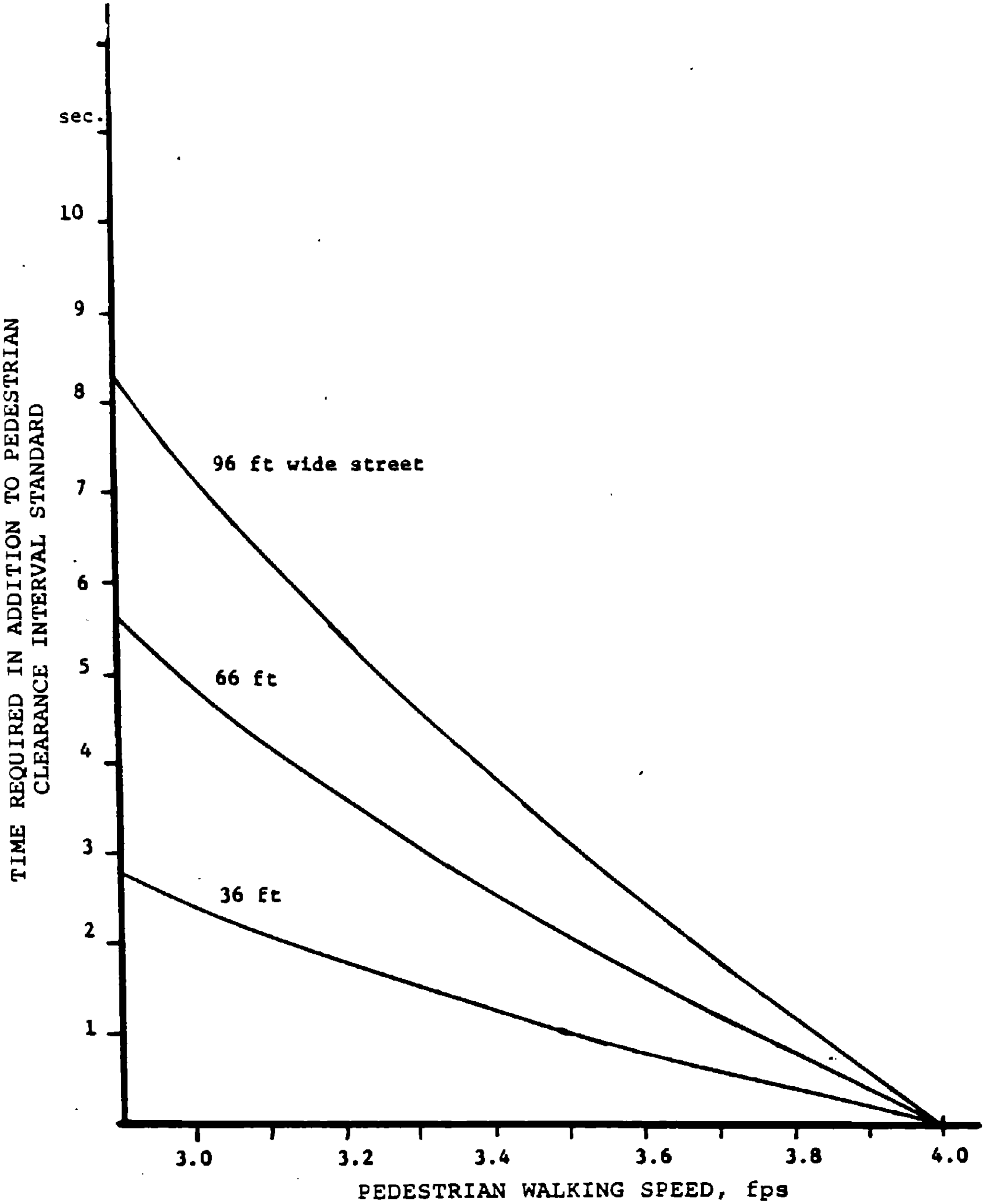
The total amount of time available for a pedestrian standing within a queue at the curb to cross a signalized intersection is the sum of the WALK interval and the clearance interval. If the pedestrian steps off the curb before the end of the WALK interval and then walks at or above the walking speed specification, 4 fps (1.2 m/s), the pedestrian will reach the middle of the farthest traveled lane before the end of the Flashing Don't Walk cycle. If the same pedestrian walks at a speed less than the 4.0 fps (1.22 m/s) specification, it is still possible for the pedestrian to reach the middle of the farthest traveled lane provided the pedestrian leaves the curb prior to onset of the pedestrian clearance cycle. The amount of time prior to the end of the WALK cycle which the slower walking pedestrian must leave is a function of both the walking speed of the pedestrian and the width of the street. Figure 17 illustrates this relationship for three representative street widths. For example, a pedestrian with a walking speed of 3.5 fps (1.07 m/s) is attempting to cross a 66 foot (20.1 m) wide street and thus needs to travel approximately 60 feet (18.3 m) in order to reach the middle of the farthest traveled lane. In order for the pedestrian to complete this maneuver, the pedestrian must step from the curb 2.1 seconds prior to the beginning of the 15 seconds design clearance interval. If the WALK interval is 7 seconds, the pedestrian has 4.9 seconds from the onset of the WALK interval to leave the curb. For 36 and 96 feet (11.0 m and 29.3 m) wide streets, the above pedestrian must leave 1.1 and 3.2 seconds, respectively, prior to the beginning of the clearance interval.

Although the "looseness" of the WALK interval standard can in some instances help accommodate the slower pedestrians who otherwise are restricted by the standard for pedestrian clearance interval, the exclusion effects of this latter standard should be evaluated based on the worst design condition. In this case, the worst design situation is defined as a pedestrian stepping off the curb to cross the street at the precise moment the WALK interval terminates and the clearance interval starts. It is recognized that a significant proportion of the pedestrian population does not heed the Flashing Don't Walk indication and proceeds to leave the curb and cross the street even after the WALK interval ends. However, because these pedestrians are not complying with the instructions of the traffic control device, they do not fall within the design constraints.

In the design situation for the clearance interval, a pedestrian initiating a street crossing concurrent with the beginning of the Flashing Don't Walk cycle must walk at a speed of 4.0 fps (1.22 m/s) in order to satisfy the requirements of the standard (i.e., to reach the middle of the farthest traveled lane). A pedestrian walking slower than 4.0 fps (1.22 m/s) will not reach the middle of the farthest traveled lane. Figure 18 illustrates where pedestrians with various walking speeds are located at the end of the clearance interval. On a 96 foot (29.3 m) wide street, a 3.2 fps (0.98 m/s) pedestrian will still be two travel lanes away from the far curb when the clearance interval ends. On a 66 foot (20.1 m) street, a 3.6 fps (1.10 m/s) pedestrian is over a full travel lane short of the far curb. The seriousness of these shortfalls can best be understood by examining the potential conflicts with vehicular traffic.

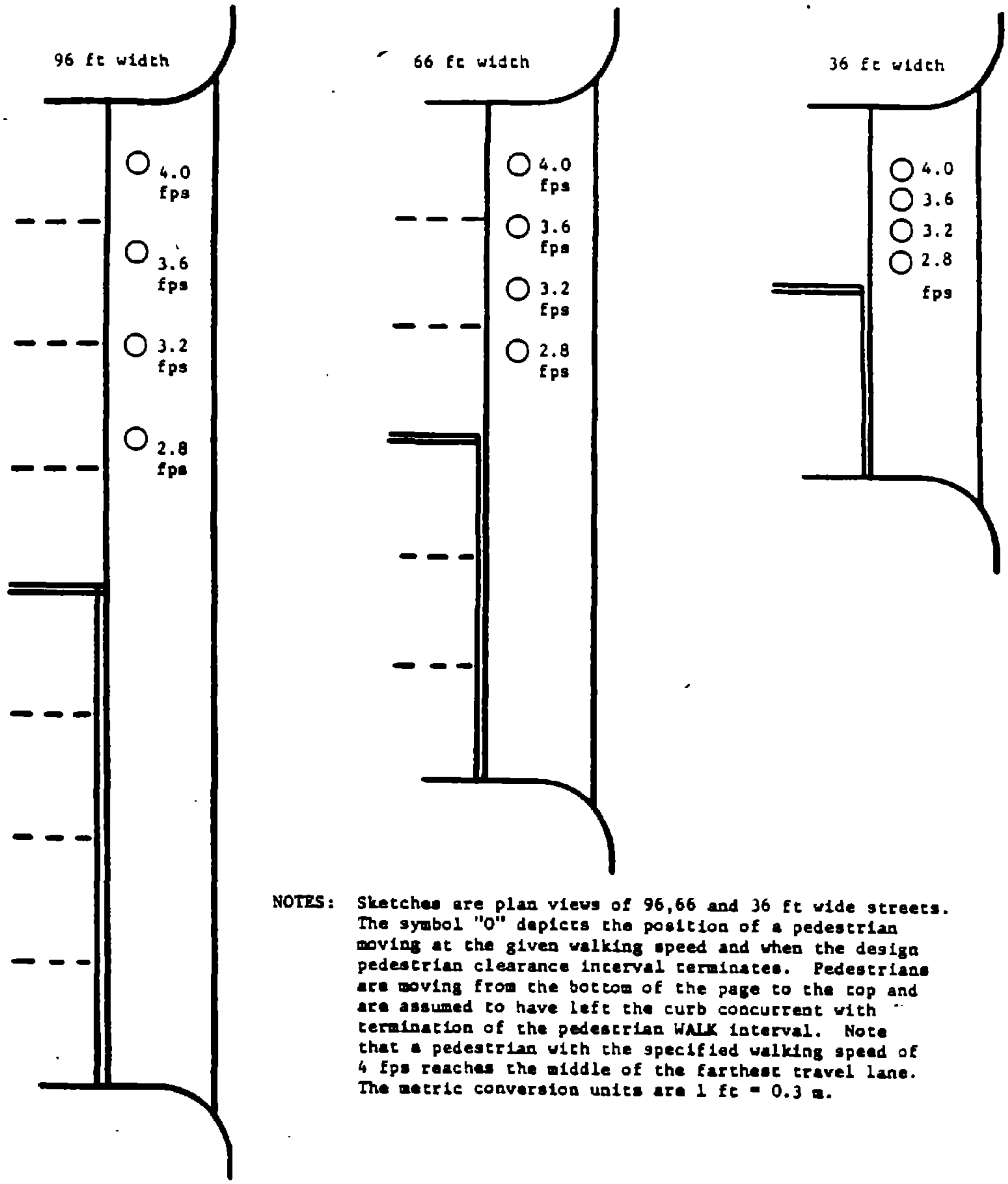
The MUTCD calls for a clearance interval of sufficient length "to allow a pedestrian crossing in the crosswalk to leave the curb and travel to the center of the farthest traveled lane before <u>opposing vehicles receive a green indication</u>" (emphasis added). In other words for combined pedestrian-vehicular signal phasing (which is the prevalent type of phasing used in the U.S.), the Flashing Don't Walk pedestrian signal and the amber indication of the vehicular traffic signal end at the same moment. Therefore, immediately prior to the end of the pedestrian clearance interval, the amber indication is illuminated signifying that all vehicles should clear the intersection. However, neither the right- or left-turning vehicles are able to clear the intersection if a pedestrian is still well into the vehicle travel lanes as could be the case even for a pedestrian walking at the design speed and who legally left the curb on the WALK signal. For the slower walking pedestrian, the safety problem is further exacerbated by the greater shortfall from refuge at the far curb. The MUTCD states that the green indication for opposing traffic may be given immediately upon the end of the pedestrian clearance interval. Again, even a legally-crossing, design speed pedestrian is still one-half of a travel lane short of clearing the crosswalk at the end of the clearance interval. A vehicle stopped at the intersection and on the same side of the intersection as the crossing pedestrian must delay acceleration upon the start of the green GO indication until the pedestrian has reached the destination curb. In the case of a design speed pedestrian (4.0 fps (1.22 m/s)), the time involved is 1.5 seconds for 12 feet (2.7 m) travel lanes. In the case of a 3.5 fps (1.07 m/s) pedestrian crossing a 96 foot (29.3 m) street, the time involved is 4.9 seconds.

Application of the MUTCD standards at intersections using early pedestrian release phasing, late pedestrian release phasing, or scramble timing is similarly affected by slower pedestrian walking speeds.



The SI (metric) conversions are 1 ft = 0.3 m.

FIGURE 17--Additional Time Beyond Clearance Interval Required by Slow Pedestrian Corssing a Street



NOTES: Sketches are plan views of 96,66 and 36 ft wide streets. The symbol "O" depicts the position of a pedestrian moving at the given walking speed and when the design pedestrian clearance interval terminates. Pedestrians are moving from the bottom of the page to the top and are assumed to have left the curb concurrent with termination of the pedestrian WALK interval. Note that a pedestrian with the specified walking speed of 4 fps reaches the middle of the farthest travel lane. The metric conversion units are 1 ft = 0.3 m.

FIGURE 18 --Effect of Varying Walking Speeds on the Final Position of Pedestrians at Typical Intersections

It is difficult to estimate the distribution of pedestrians who are adversely impacted by the current specification for walking speed, 4.0 fps (1.22 m/s). Based on observations by Sleight (82), just under 30 percent of the adult (non-elderly) pedestrian population is excluded by the current specification; slightly more than 20 percent of the elderly and only less than 10 percent of the children fail to meet the current specification for walking speed. Bruce (83) estimated that in general approximately 35 percent of the pedestrian population is excluded by the current specification. However, the exact proportion of the pedestrians crossing a particular street who are not walking at a speed of 4.0 fps (1.22 m/s) is entirely a site-dependent function. The pedestrian density, the age breakdown of the pedestrians, the sex distribution of the pedestrians, the width of the street and trip purposes of the pedestrians can all vary from one intersection to another intersection and from one time-of-day to another.

SIGN LETTER HEIGHT

Sensitivity Analysis

Sign letter height is determined from equation (25) with the implied driver characteristic being visual acuity. For the purpose of the sensitivity analyses, letter legibility, ℓ, is used as a surrogate for driver visual acuity since the standard for letter legibility represents the distance at which words (thus letters) are read in field conditions.

The sensitivity of the standard with respect to a change in the driver characteristic can be determined by substituting values of $H + \Delta H$ and $\ell + \Delta \ell$ in equation (25) where Δ implies an incremental increase or decrease, and solve for H. The resulting equation simplifies to

$$\Delta H + \frac{-L \Delta \ell}{\ell(\ell + \Delta \ell)} \qquad (43)$$

The preceding equation can be solved for various increments of ℓ, given values of L, ℓ, and H.

The results of the application of this formula for a given legibility distance of 700 feet (213m) can be seen in Table 31 Change in the minimum letter height are determined for ± 10 percent changes in the letter legibility factor. The results are also displayed in Figure 19 which shows the percent change in the minimum letter height as a function of a percent change in letter legibility. It is evident from the figure that:

- As ℓ decreases for a given value of L, H increases non-linearly. That is, the letter height, H, becomes more sensitive to a change in the letter legibility factor, ℓ, with decreasing values of ℓ.
- Conversely, as ℓ increases for a given value of L, H decreases approximately linearly. That is, H becomes less sensitive to a change in ℓ with ever increasing values of ℓ.

Consequently, the sensitivity of the standard for minimum letter height on highway guide signs is stated in terms of the letter legibility factor. For instance, in the case of $\ell = 50$, a 10 percent increase in the letter legibility value, L , would justify a 9 percent decrease in the minimum letter height, and a 10 percent decrease would require an 11 percent increase in the minimum letter height. Extreme changes of 50 percent increase or decrease would necessitate corresponding changes of +100 percent and -34 percent in letter height, respecitively.

Exclusion Analysis

The decreasing letter legibility values and their respective increased letter heights resulting from the sensitivity analysis (Table 31) are of practical significance to the design of highway guide signs when comparing the relationship of the driver characteristic, visual acuity (in terms of letter legibility), to the current letter height standard. Equivalent static visual acuity scores for letter legibility values were determined and are presented next to their corresponding letter legibility value in Table 31. The initial results indicate that those drivers with less than 20/25 vision need more letter height to read highway guide signs than what may be currently provided them by the standard.

In order to demonstrate the consequences of a letter height standard using a constant of 50 feet of legibility distance per inch of letter height for highway signs, Table 32 compares the legibility distance required to read a sign 400 feet (122 m) in front of the hazard at 55 mph (89 km/h), and the actual legibility distance given to drivers who have less than 20/25 visual acuity. The table also shows the reduced speeds necessary for those drivers excluded by the standard to be able to read the sign using a letter (10.6 inches (269 mm) in this case) determined by the current standard.

TABLE 31--Changes in Letter Height for Changes in Letter Legibility

% Δ from ℓ	New ℓ Value	Visual Acuity Equivalent	New Letter Hgt. (inches)	% Δ from Standard
+50	75	--	7.1	-33
+40	70	--	7.6	-28
+30	65	--	8.1	-24
+20	60	20/20	8.8	-17
+10	55	--	9.6	- 9
(0)	(50)	20/24	(10.6)	(0)
-10	45	--	11.8	+11
-20	40	20/30	13.3	+25
-30	35	--	15.1	+43
-40	30	20/40	17.7	+67
-50	25	20/50	21.2	+100

The SI (metric) conversion is 1 in = 25.4 mm.

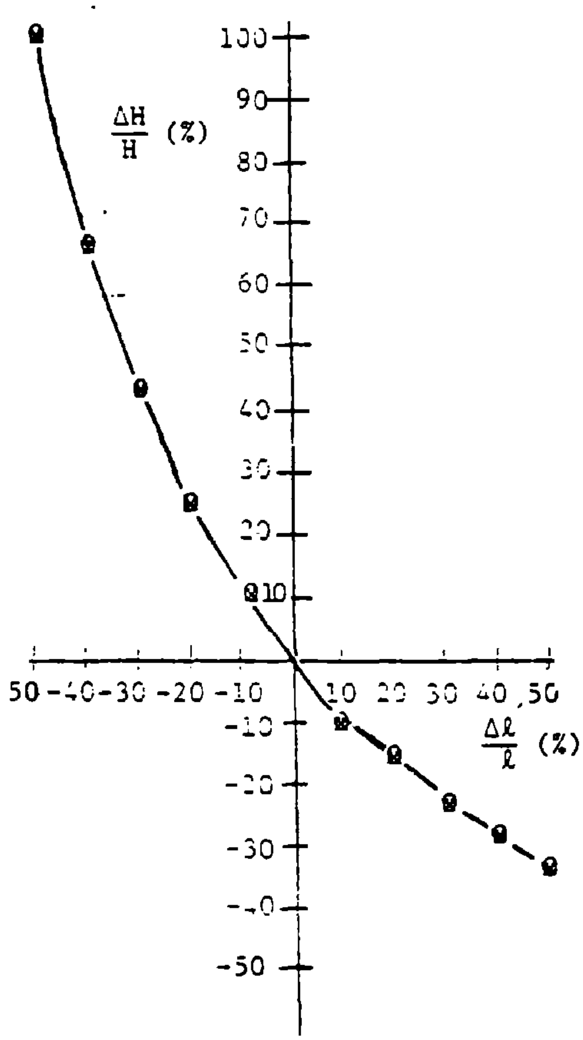


FIGURE 19 --Percent Change in Minimum Letter Height as a Function of Percent Change in Letter Legibility

The comparison of required versus actual legibility distance for drivers with varying static visual acuity strongly indicates that 25 percent of the driving population may not be able to read highway guide signs at the distances necessary to comfortably and/or safely decelerate to the point at which the next driving maneuver should take place. Legibility distances for this portion of the driving population range from a 20 percent decrease in required legibility distance to a 60 percent decrease.

It is very probable that many drivers compensate for poorer vision and thus shorter legibility distances (as a function of the current letter height standard) by driving slower than the 90th percentile speed. Table 32 reveals that a range of decreased speed from 11 percent to 35 percent would be required in order for drivers with less than 20/25 vision to read the sign in enough time to decelerate comfortably and safely. The consequence of this probable compensation would be the interruption of the flow of traffic to some degree (even in the right lane designed for lower speeds) by complicating and aggravating the driving responses of other motorists.

Statis visual acuity decreases significantly with age--especially at night--beginning at about age 50 and continuing with advancing age. The majority of 20/20-20/25 drivers are younger drivers (< age 50). It is, therefore, noted that the current standard for letter height design on highway guide signs, which uses a letter legibility value of 50 feet per inch of letter height, does not adequately consider the visual needs of the elderly, whose average letter legibility value is 26 feet per inch of letter height. The standard excludes a significant portion of the population--that of the elderly--which has been increasing steadily per decade over the past forty years. It is also worthwhile noting that of those motorists driving under the design speed limit, many of them happen to be the elderly.

Critique of Specification

By directly relating letter legibility values to static visual acuity scores, letter legibility is being evaluated as the driver characteristic involved in the minimum letter height standard. From the sensitivity analysis, it is apparent that the minimum letter height standard is sensitive to a change in the letter legibility value. Of particular interest and concern, is the implication and ramification of decreasing values of letter legibility and the resulting substantial increases for letter height. Currently, the standard uses a daylight letter legibility value of 50 feet (15.2 m) of legibility distance per inch of letter height (versus a 33 foot (10.1 m) value obtained in nighttime studies (22)) for 20/25 drivers. Consequently, because of the poorer vision (less than 20/25) of approximately 25 percent of the driving population--comprised mostly of the elderly--further research is needed to evaluate the relationship of static visual acuity difference and minimum letter height requirements on highway guide signs for both day and night (using reflectorized letters and/or floodlights) conditions.

TABLE 32--Comparison of Required Versus Actual Legibility Distances According to Driver Population Static Visual Acuity

	STATIC VISUAL ACUITY						
	20/20 55th %tile	20/25 75th %tile	20/30 85th %tile	20/40 95th %tile	20/50 99th %tile	20/60 100th %tile	20/70 100th %tile
Letter Legibility, ℓ, ft per inch	60	50	40	30	25	20	15
Required Legibility Distance, ft, for 55 mph	530	530	530	530	530	530	530
Actual Legibility Distance, ft, using Current Standard (10.6 inch, letter height)	636 (+106)	530 (0)	424 (-106) (20% dec.)	318 (-212) (40% dec.)	265 (-265) (50% dec.)	212 (-318) (60% dec.)	159 (-371) (70% dec.)
Reduced Speed, mph, Needed to Read Sign using 10.6 inch Letters	---	---	49 (- 6) (11% dec.)	42 (-13) (24% dec.)	39 (-16) (29% dec.)	36 (-19) (35% dec.)	33 (-22) (40% dec.)

The SI (metric) conversions are 1 ft = 0.3 m, 1 in = 25.4 mm, and 1 mph = 1.61 km/h.

V -- SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

The principal objectives of this study were:

1. Identify highway design and operation standards which are based on a driver or pedestrian characteristic.

2. Determine which of these standards are sensitive to a change in the characteristic specification.

3. Determine which current characteristic specifications are too liberal or too conservative given the estimated population distribution.

4. Identify where further research is needed to better define the population distribution of the characteristic.

This chapter addresses these main concerns by summarizing the analyses presented in earlier chapters, drawing conclusions, suggesting modifications to the standards or specification, and recommending further research where deemed necessary.

STANDARDS BASED ON DRIVER/PEDESTRIAN CHARACTERISTICS

Highways and their controls and devices are designed on the bases of some key parameters; namely, the current or projected volume and directional distribution of traffic, vehicle type and mix, the desired level of service and speed, and the degree of access control. Once these parameters are set, then certain specific features can be designed. For example, sight distance requirements are determined from the desired design speed.

In this regard, the driver or pedestrian is not a major design control. That is to say, when the overall design type of a highway is being determined, the types and characteristics of the driver or pedestrian are not explicitly considered. To do so would be to require the establishment of a design driver which is virtually impossible given the wide variation in his/her performence depending upon the highway situation or the person's condition.

Driver and pedestrian characteristics, however, are considered within the specific elements of highway geometric traffic operations and control design. A review of numerous manuals identified some 13 geometric design and 5 traffic control devices standards that explicitly consider a driver or pedestrian characteristic with an associated specification. These are noted in Table 2 and described in Chapter II. The standards review also identified other specific standards which implies a driver characteristic but for which there is no explicit consideration and, of course, no specification. An example of this would be any standard related to the visibility of a device. Implicit in these standards is the visual capabilities of the driver which are rarely specified.

Aside from identifying standards which include a driver/pedestrian characteristic, each standard was critiqued. Recommendations for modification of the standard or at least further examination are highlighted below.

The differences between AASHTO design and MUTCD marking concepts for Passing Sight Distance should be examined with an eye toward developing a uniform approach which explicitly takes into account both driver and vehicle characteristics. The current standards do not explicitly account for the actions of the driver in the passing maneuver. One potential technique would center on the "pass abort" situation which explicitly involves both driver and vehicle characteristics.

The analysis of Case I, Intersection Sight Distance indicates a need for a revised methodology. The existing formulation does not allow even a minimal margin of safety for a driver faced with an extremely hazardous situation at an uncontrolled intersection. It is recommended that the methodology presented briefly in the text, or one that follows a similar approach, be employed as the revised methodology for computing Case I Intersection Sight Distance.

The methodology recommended for use by AASHTO in determining Case II Intersection Sight Distance is simply the use of stopping sight distance for each leg of the minimum sight triangle. However, as was discussed in the previous chapter, the AASHTO methodology does not provide adequate sight distance within which a vehicle can be brought to a stop to avoid another vehicle. Instead the AASHTO methodology for Case II Intersection Sight Distance produces sight distances which will, in some instances, allow the driver only enough time and distance to modify the speed of the vehicle (not bring it to a stop) before entering the uncontrolled intersection. Therefore, it is recommended that the Case II Intersection Sight Distance methodology be modified in order that the actions called for in the definition of Case II may actually be accomplished. The text provides an example of one such methodology that could be used.

The current methodology adequately depicts a driver's actions in situations described by the Case III Intersection Sight Distance standard. However, another equally valid approach would be to use gap acceptance as a criterion for determining minimum Case III (and Cases IV and V) Intersection Sight Distance. It is recommended that the feasibility of such an approach be determined by comparing it to the existing methodology for their impact on safe and efficient traffic flow.

The Sight Distance Measuring Criteria of 6 inch (0.15m) object height for Stopping Sight Distance and a 4.25 foot (1.30m) object height for Passing Sight Distance have been the subject of debate for many years. It is recommended that the reasonableness of these criteria be re-examined with particular emphasis being placed on the issue of whether or not a driver is capable of detecting the "design object" at the "design distance". In addition, the analysis should address the issues of criticality of sighting an object and frequency of an occurrence.

It is recommended that the formulation for computing the Adequate Gap Time for School Crossing Traffic Signal Warrant be re-evaluated. In particular, it is suggested that the evaluation focus on the history behind and the current validity of the assumption that pedestrians cross in rows of 5 with 2 seconds between rows.

The current MUTCD standard for the Pedestrian Clearance Interval is intended to provide sufficient time for the 4.0 fps (1.22 m/s) pedestrian to reach the middle of the farthest traveled lane. As has been demonstrated earlier in the text, this further compounds the difficulty experienced by slow walkers in crossing a signalized intersection. We were unable to trace the original justification for not designing for a full curb-to-curb crossing. It is recommended that consideration be given to redefining the walking distance as the curb-to-curb or safe refuge distance. The current standard, by designing for an incomplete crossing, fails to provide for left or right turning vehicles which are attempting to clear the intersection at the end of their green "GO" cycle.

STANDARDS THAT ARE SENSITIVE TO DRIVER CHARACTERSITCS

A major concern of this study was to identify those standards which are sensitive to a change in the driver characteristic specification. In other words, what standards would be significantly affected by a change in specification. This is of particular interest if it is further determined that the current specification is set too high or low for a given percentile of the population.

It should be recognized that sensitivity is a relative indicator with no precise demarcation as to when a standard is sensitive or insensitive. Very large or small changes can easily be termed sensitive or insensitive, but it is not so obvious to describe the sensitivity where the standard changes by say 5-15 percent for a reasonable change in the specification. Furthermore, the real measure of the sensitivity lies with ecomomic consequences. For instance how much more will it cost if stopping sight distance standards are increased by 5 percent, 10 percent, etc?

Having noted these concerns, the sensitivity analysis presented in Chapter III identified that most of the design and operations standards are deemed "sensitive" to realistic changes in the driver characteristic specification. These include:

Geometric Design

- Stopping Sight Distance
- Decision Sight Distance
- Intersection Sight Distances for Case I and Case II
- Railroad Grade Crossing Corner Sight Triangle
- Crest Vertical Curve Length (due to Stopping Sight Distance)
- Sag Vertical Curve Length
- Horizontal Curvature
- Lateral Clearance to Sight Obstructions on Horizontal Curve

Traffic Operations

- Adequate Gap Time for School Crossing Traffic Signal Warrant
- Pedestrian Clearance Interval for Pedestrian Signal Timing
- Vehicle Clearance Interval for Traffic Signals
- Sign Letter Height

DRIVER/PEDESTRIAN CHARACTERISTIC SPECIFICATIONS

There are innumerable human characteristics that are involved in all aspects of driving or walking. These characteristics can be categorized as physical, mental, sensory, and motor factors. However when considering specific geometric design and operations standards, the only relevant human characteristics are as as follows:

- Perception of objects
- Visual capabilities
- Physical reaction time for braking, shifting, etc.
- Eye height
- Walking speed

Discussion of these characteristics and their distribution profile can be found in Chapter III.

A few general conclusions concerning driver/pedestrian characteristics and their current specifications are enumerated below:

1. The physical and performance characteristics of the driver and the pedestrian have not been adequately considered in the various standards. For most standards the driver or pedestrian characteristic is either weakly supported by empirical research, incorrectly considered, or uses an inappropriate specification. Furthermore, for nearly all driver and pedestrian characteristics there is insufficient data to determine a reasonably accurate estimate of the distribution profile for all drivers and pedestrians. This means that for most of the specifications, it is not known with certainty what percentage of drivers or pedestrians are being excluded by the standard.

2. The role of driver visual performance has been largely ignored in most design and operations standards which involve detection/perception/recognition of stationary and moving objects. The research presented in this study indicates that the 95th percentile driver is capable of detecting a 6 inch (0.15 m) object at the current Stopping Sight Distance standard. However, whether or not the driver can recognize it as a 6 inch (0.15 m) object, vis-a-vis a 4 or 2 inch (0.10 or 0.05 m) object is questionable. But at the AASHTO and MUTCD passing sight distances, the estimated 85th percentile driver (based on visual acuity) is incapabile of detecting an approaching vehicle as specified by the sight distance measuring criteria. Furthermore, with regard to nighttime visibility, the research documented in this study provides strong evidence that drivers are incapable of detecting the vast majority of objects at Stopping Sight Distance design values in the absence of ambient lighting. This shortfall occurs on flat, tangent sections as well as on crest and sag vertical curves and on horizontal curves.

3. There is a basic incompatibility between the minimum visual capabilities required for licensing of drivers and the visual capabilities of drivers assumed for design standards. In a majority of States a minimum static visual acuity level of 20/40 is all that is required. Testing of licensed drivers indicates that the 85th percentile driver is at about a 20/33 level. In designing of sign letter size, a visual acuity better than these levels is assumed. For night driving, the problem is more severe as visual capabilities under low illumination levels is much worse. Also, vision capabilities deteriorate with age, especially around 60 years and older. The elderly continue to comprise an increasing percentage of the drivers. These factors lead to the need for a complete evaluation of the compatibility of the visual ability of the driver, design standards and driver licensing.

Keeping in mind the results of the sensitivity analysis, recommendations concerning specific driver/pedestrian characteristics are highlighted below.

The current 2.5 second specification for <u>perception-brake reaction time</u> has been used for Stopping Sight Distance and its derivative standards for horizontal and vertical curves for many years. It has been argued by others that this value is too low (e.g., Glennon (86)) and estimates of the characteristic developed for this study support this claim. It is estimated that the 85th percentile value may be as high as 3.2 seconds. On the other hand, there is no strong documented evidence that the current minimum and desirable stopping sight distances, which are based on 2.5 seconds, are inadequate or at least unacceptable on the grounds of safety. There might be justification for increasing the specification if the values estimated here are considered reliable. Before this is considered, further empirical research is needed to derive a more reliable distribution of perception-brake reaction time for the driving population.

Estimates of <u>perception-reaction time</u> for several other standards have been developed for this study and indicate that the current specification values are inadequate. These standards are Intersection Sight Distance (Cases I and II), Railroad-Highway Grade Crossing Sight Distance and Vehicle Change Interval. Each of these standards is sensitive to changes in the value of perception-reaction time and thus it is important to validate as closely as possible the proper value for the driver characteristic.

The specification value for Cases III, IV and V Intersection Sight Distance perception reaction time, 2.0 seconds, should be retained but the current definition of the driver characteristic should be changed to reflect only one head movement which is a more accurate description of the driver's action.

With regard to perception-reaction in general, there is a need for better empirical data on the time related to detecting and reacting to:

- an object in the road such as another vehicle, a pedestrian, or a "6 inch object" for stopping sight distance;
- a vehicle in the opposing lane for passing sight distance;
- a vehicle approaching an intersection for intersection sight distance; and
- a train approaching a crossing.

Studies should consider alerted versus unalerted driver, age of driver, and effects of speed, time of day (day, night), ocation (urban, rural) and other influencing variables.

There is no definitive data to support the present specification of the pedestrian perception-reaction time for the adequate Gap Time for School Crossing Traffic Signal Warrant. However, the specification does seem reasonable; and because of this, and the fact that adequate gap time is relatively insensitive to a change in specification, data collection is not necessary.

By directly relating letter legibility values to static visual acuity scores, letter legibility is being evaluated as the driver characteristic involved in the minimum letter height standard. From the sensitivity analysis, it is apparent that the minimum letter height standard is sensitive to a change in the letter legibility value. Of particular interest and concern is the implication and ramification of decreasing values of letter legibility and the resulting substantial increases for letter height. Currently, the standard uses a daylight letter legibility value based on drivers with 20/25 visual acuity. Consequently, because of the poorer vision (less than 20/25 of approximately 25 percent of the driving population--comprised mostly of the elderly--further research is needed to evaluate the relationship of static visual acuity differences and minimum letter height requirements on highway guide signs for both day and night (using reflectorized letters and/or floodlights) conditions.

The current specification for pedestrian walking speed conforms to approximately the "average" or "median" pedestrian. The estimated 85th percentile value is 3.5 fps (1.07 m/s). The sensitivity analyses presented earlier illustrate the high sensitivity of walking distance to walking speed within the range of expected values. In order to accommodate the 85th percentile pedestrian, it is recommended that the design specification for walking speed be lowered to 3.5 fps (1.07 m/s).

Driver eye height estimates presented in this report illustrate the existing trend toward an overall lower eye height population distribution. To accommodate this trend, it is recommended that the specification be reduced from the current 42 inches (1.07m) value to 41 inches (1.04m), or possibly to 40 inches (1.02m). A decrease to the former value would necessitate slight increases in current crest vertical curve design standards for 50 and 60 mph (80 and 97 km/h) design speeds. On horizontal curves it is recommended that the midpoint design height of a sight obstruction should be lowered to 20 inches (0.51m) in order to accommodate reduced driver eye heights and to enable the driver to sight an object in the roadway from the roadway surface up.

The underlying assumption for selection of the current side friction factors on horizontal curves is that they are related to the motorists' threshold of discomfort. Since this study did not show if these side friction factors values used are applicable for current vehicles, it cannot be stated whether the specifications are too stringent or to unconstrained. Arguments have been presented earlier in this report for both cases. It is apparent that horizontal curvature standards are sensitive to a change in the side friction factor--at least sufficiently so to warrant further research into evaluating the relationship of side friction factor, driver comfort and minimum radius. In this regard, it is recognized that FHWA has initiated a research project entitled "Side Friction for Superelevation on Horizontal Curves" which should resolve this issue.

For several standards, it has been

suggested that the specification for the driver characteristic is too liberal. This implies that the standards are inadequate in that they do not accommodate the "vast majority" of drivers, a situation which should be revealed by an unsafe or at least inefficient highway system or both. Aside from knowing that roads of high design (e.g., Interstates) have better accident rates than roads with inadequate design features (e.g., a large percentage of the two lane rural system), we do not know the relationships of specific design elements to safety. Therefore, it can not be stated that current design standards are resulting in accidents because of an inadequate driver characteristic specification. Furthermore, while it should not be assumed, it is likely that drivers who are excluded by a particular specification level are compensating by, for instance, driving slower or not driving at certain times, e.g., at night.

REFERENCES

1. A Policy on Geometric Design of Rural Highways, American Association of State Highway Officials. Washington D.C., 1965.

2. A Policy on Design of Urban Highways and Arterial Streets, American Association of State Highway and Transportation Officials. Washington, D.C., 1973.

3. Manual on Uniform Traffic Control Devices, Federal Highway Administration, Washington, D.C., 1978.

4. "Design Standards for Highways", Federal-Aid Highway Program Manual, 6-2-1-1. Federal Highway Administration, Washington, D.C., 1978.

5. A Policy on the Geometric Design of Highways and Streets (Review Draft 2), John F. Holman & Co. Inc., Dec. 1979 and (Review Draft 3), July 1982.

6. Prisk, C.W. "Passing Practices on Rural Highways," HRB Proceedings, Vol. 19, 1939.

7. Alexander, G.J. and Lunenfeld, H., "Positive Guidance in Traffic Control" Federal Highway Administration, Washington, D.C., April 1975.

8. McGee, H.W., Moore, W., Knapp, B.G. and Sanders, J.H., "Decision Sight Distance for Highway Design and Traffic Control Requirements." FHWA-RD-78-78, Federal Highway Admin., Feb. 1978.

9. Correspondence and Verbal Communication with Jack E. Leisch of Jack E. Leisch Associates. Evanston, IL.

10. Glennon, J.C., Balenta, J.J., Thorson, B.A. and Azzeh, J.A., "Technical Guidelines for the Control of Direct Access to Arterial Highways", Vol. I: General Framework for Implementing Access Control Techniques and Vol. II: Detailed Description of Access Control Techniques. Report No. FHWA-RD-76-86 & -87, Federal Highway Administration, Aug. 1975.

11. Reilly, W.R., Kell, J.H. and Fullerton, I.J., "Design of Urban Streets", Technology Sharing Report #80-204. Federal Highway Administration. Washington, D.C. 10590, Jan. 1980.

12. Stover, V.G. et al., "Guidelines for Medial and Marginal Access Control on Major Roadways", NCRHP Report 93, Highway Research Board.

13. "Guidelines for Driveway Design and Location", Institute of Transportation Engineers, 1974.

14. Schoppert, D.W. and Hoyt, D.W., "Factors Influencing Safety at Highway-Rail Grade Crossings", National Cooperative Highway Research Program, Report 50, 1968.

15. Correspondence from Federal Highway Administration to American Association of State Highway & Transportation Officials Policy Committee, 1981.

16. Baerwald, J.E. (editor), Transportation and Traffic Engineering Handbook, Institute of Transportation Engineers, Prentice Hall, Inc., Englewood Cliffs, N.J. 1976.

17. Traffic Control Devices Handbook, An Operating Guide, Federal Highway Administration (revised edition in print).

18. A Program for School Crossing Protection, Institute of Transportation Engineers, 1962.

19. "Report on Massachusetts Highway Accident Survey", Massachusetts Institute of Technology, 1934.

20. King, G.F., "Guidelines for Uniformity in Traffic Signal Design Configuration". Final Report Volume 1, KLD Associates, Huntington Station, NY July 1977.

21. Forbes, T.W. and Holmes, R.S., "Legibility Distances of Highway Destination Signs in Relation to Letter Height, Letter Width, and Reflectorization", Proceedings of the Highway Research Board, 19, 321-335, 1939.

22. Forbes, T.W., "Factors in Visibility and Legibility of Highway Signs and Markings", NAS/NRC Committee on Vision, National Academy of Sciences, Pub. No. 574, Washington, D.C., 1979.

23. McKnight, A.J. and Adams, B.B., "Driver Education Task Analysis, Vol. I: Task Descriptions". Human Resources Research Organization, Alexandria, VA. DOT HS 800 368, Nov. 1970.

24. U.S. Bureau of Census, "Nationwide Personal Transportation Study 1969-1970".

25. Marsh, G.W., "Aging and Driving" Traffic Engineering Vol. 31, No. 2 1960.

26. Berg, A., "Characteristics of Drivers", in Human Factors in Highway Traffic Safety, T. Forbes (ed)

27. Federal Highway Administration, "Highway Statistics", 1979.

28. Federal Highway Administration, "1977 Nationwide Personal Transportation Study".

29. National Highway Traffic Safety Administration, "Comparative Data, State and Provincial Licensing Systems" Report DOT HS 805-335, Oct. 1980.

30. Burg, A., "Visual Acuity as Measured by Dynamic and Static Tests". Journal of Applied Psychology, 1966, Vol. 50, No. 6, pp. 460-466.

31. Burg, A.,"Vision Test Scores and Driving Record: Additional Findings," Report No. 68-27, Los Angeles: University of California, Dept. of Engineering, Dec. 1968.

32. Davison, P.S. and Irving, A., Survey of Visual Acuity of Drivers. TRRL Lab Report No. 945, Transports and Road Research Laboratory, 1980.

33. Henderson, R.L. and Burg, A., Vision and Audition in Driving. NHTSA Report DOT-HS-801-265, Nov. 1974.

34. Allen, R.W., Parseghian, Z. and Van Valkenburgh, P.G., "Age Effects on Symbol Sign Recognition". Systems Technology Inc., FHWA/RD-80/126, Dec. 1980.

35. Sivak, M. and Olson, P.L., "Nighttime Legibility of Traffic Signs: Conditions Eliminating the Effects of Driver Age and Disability Glare". Accident Analysis and Prevention, 1981, in press.

36. Burg, A. and Hulbert, S.F., "Dynamic Visual Acuity and Other Measures of Vision", Percept. Mot. Skills, 9, p. 334, 1959.

37. Mitchell, A. and Forbes, T.W., "Design of Sign Letter Sizes". Proceedings of American Society of Civil Engineers. Vol. 68, 1942, pp. 95-104.

38. Lee, R.L. and Scott, C.P., "Driver Eye Height and Vehicle Sales Trends (1969-1979)", FHWA Office of Highway Safety, May 1980, updated in January 1981.

39. Judd, D.G., "Facts of Color-Blindness", Journal of Optical Society of America, Vol. 33, 1943, pp. 294-307.

40. Lee, C.E., Driver Eye Height and Related Highway Design Features. Highway Research Board Proceedings, Vol. 39, 1960, pp. 46-60.

41. Cunagin, W. and Arbrahamson, T., "Driver Eye Height: A Field Study", ITE Journal, Vol. 49, No. 5, May 1979.

42. Scott, A., "Developing Standards for Stopping Sight Distances and Crest Vertical Curvature", RTAC Forum, Vol. 2, No. 1.

43. Hills, B.L., "Proceedings of the May 8, 1976 OECD Meeting on Geometric Road Design Standards", 1976.

44. Driver Eye Height Comparison for 1976 and 1978, Motor Vehicle Manufacturers Association.

45. Boyd, M.W.; Littleton, A.C.; Boenau, R.E.; and Pilkington, G.B., Determination of Motor Vehicle Eye Height for Highway Design. Report No. FHWA-RD-78-66, FHWA, April 1978.

46. Stonex, K.A., "Driver Eye Height and Vehicle Performance in Relation to Crest Sight Distance and Length of No-Passing Zones," HRB Bulletin 195, 1958.

47. Lee, R.E., "Driver Eye Height", Australian Road Research, Vol. 1, No. 6, Jun. 1963.

48. Seger, E.E. and Brink, R.S., "Trends of Vehicle Dimensions and Performance Characteristics from 1960 through 1970". Highway Research Record 420, 1972, pp. 1-15.

49. Anderson, G.M., "Driver Eye Height Study", Australian Road Research, Vol. 4, No. 4, Jun. 1970.

50. Stoudt, H.W.; Crowley, T.J.; McFarland, R.A.; Ryan, A.; Gruber, B.; and Ray, C., "Static and Dynamic Measurements of Motor Vehicle Drivers". Guggenheim Center for Aerospace Health and Safety, Harvard School of Public Health, Harvard University, Jun. 1969.

51. Haslegrave, C.M., "Anthropometric Profile of the British Car Driver". Ergonomics, 1980, Vol. 23, No. 5, pp. 437-367.

52. "1977 Census of Transportation, Truck Inventory and Use Survey," U.S. DOT, 1977.

53. Sanders, M.S., "Anthropometric Survey of Truck and Bus Drivers", FHWA Feb. 1977.

54. Johnansson, G. and Rumar, K., "Drivers Brake Reaction Times." Human Factors, Vol. 13, No. 1, 1971, pp. 23-27.

55. Taoka, G.T., "System Identification of Safe Stopping Distance Parameters". Department of Civil Engineering, University of Hawaii at Manoa, Sept. 1980.

56. Rackoff, N.J. and Rockwell, T.F., "Driver Search and Scan Patterns in Night Driving". Transportation Research Board Special Report 156, pp. 53-63.

57. Bartlett, N.R.; Bartz, A.E.; and Wait, J.V., Recognition Time for Symbols in Peripheral Vision. Highway Research Board Bulletin No. 330, 1962, pp. 87-91.

58. White et al., "Latency and Duration of Eye Movements in the Horizontal Plane", Journal of the Optical Society of America, Vol. 52, No. 2, Feb. 1962.

59. Matson, T.M.; Smith, W.S. and Hurd, F.W., Traffic Engineering, McGraw-Hill Book Company, NY, 1955.

60. Mourant, R.R.; Rockwell, T.H. and Rackoff, N.F., "Drivers' Eye Movements and Visual Workload". Highway Research Record No. 292, 1969, pp 1-10.

61. Ells, J.G. and Dewar, R.E., "Rapid Comprehension of Verbal and Symbolic Traffic Sign Messages". Human Factors, Vol. 21, No. 2, 1979, pp. 161-168.

62. Ells, J.G.; Dewar, R.E.; and Milloy, D.G., "An Evaluation of Six Configurations of the Railway Crossbuck Sign". Ergonomics, Vol. 23, No. 4, 1980, pp. 359-367.

63. Lunenfeld, H., "Improving the Highway System by Upgrading and Optimizing Traffic Control Devices". Report No. FHWA-TO-77-1, Federal Highway Administration, Apr. 1977.

64. Mortimer, R.G., "Dynamic Evaluation of Automobile Real Lighting Configurations", Highway Safety Research Institute, University of Michigan, 1969.

65. Sivak, M.; Olson, P.L. and Farmer, K.M., "High-Mounted Brake Lights and the Behavior of Following Drivers". Report No. UM-HSRI-81-31, Highway Safety Research Institute, Jul. 1981.

66. Robinson, G.H., "Visual Search by Automobile Drivers". Human Factors, Vol. 14, No. 4, 1972, pp. 315-323.

67. Johansson, G., Visible Distances During Night Driving, Report No. 9. The Psychological Laboratory, University of Uppsala, Sweden, Aug. 1961.

68. Normann, O.K., "Driver Passing Practices". HRB Bulletin 195, 1958.

69. Gazis, D.; Herman, R.; and Maradudin, A., "The Problem of the Amber Phase of Traffic Signals", Traffic Engineering, Feb. 1962, pp. 17-29.

70. Olson, P.L. and Rothery, R.W., "Driver Response to the Amber Phase of Traffic Signals". Traffic Engineering, Vol. XXXII, No. 5, Feb. 1962.

71. Wortman, R.H. and Matthias, J.S., "An Evaluation of Driver Behavior at Signalized Intersections", Arizona Transportation Research Center, Report No. ATTI. 82-1, May 1982 (Draft).

72. Abrams, C.M. and Smith, S.A., Urban Intersection Improvements for Pedestrain Safety, Vol. III. Signal Timing for the Pedestrian, JHK & Associates, FHWA-RD-77-144, Dec. 1977.

73. Forbes, T.W. (ed.), Human Factors in Highway Traffic Safety Research, John Wiley and Sons, Inc., NY, NY 1972.

74. Shinar, D.; Szidel, D.M.; and Paarlberg, W., Driver Performance and Individual Differences in Attention and Information Processing. Vol. 1: Driver Inattention. Institute for Research in Public Safety, Indiana University NHTSA Report DOT-HS-803-793, Oct. 1978.

75. Dewar, R.E., "Psychological Factors in the Perception of Traffic Signs." Road & Motor Vehicle Traffic Safety Branch, Ministry of Transport, Canada, Feb. 1973.

76. King, G.F., and Lunenfeld, H., "Development of Infomation Requirements and Transmission Techniques for Highway Users". NCRHP Report No. 123. Highway Research Board, 1971.

77. Miller, G.A., "The Magical Number Seven Plus or Minus Two: Some Limits on Our Capacity for Processing Information", Psychological Review, Vol. 63, No. 2, pp. 81-97, 1956.

78. ITE Technical Council Committee 5-R. "Characteristics and Service Requirements of Pedestrians and Pedestrian Facilities." Traffic Engineering, May 1976, pp. 34-45.

79. Hoel, O.A., "Pedestrian Travel Rates in Central Business Districts". Traffic Engineering, Jan. 1968, pp. 10-13.

80. Weiner, E.L., "The Elderly Pedestrian: Response to an Enforcement Campaign", Traffic Safety Research Review, Vol. 12, No. 4, 1968.

81. Sjostedt, L., "Behavior of Pedestrians at Pedestrian Crossings", National Swedish Road Research Institute, Stockholm Sweden, July 1967.

82. Sleight, R.B., "The Pedestrian" (Chapter X of). Forbes, T.W., (editor), Human Factors in Highway Traffic Safety Research, NY: John Wiley & Sons,, Inc., 1972, pp. 224-253.

83. Bruce, John A., "The Pedestrian," (Chapter 4 of). Baerwald, John E. (editor), Traffic Engineering Handbook, Institute of Traffic Engineers Washington, D.C., 1965, pp. 108-141.

84. Pushharev, B.S., and Zupan, J.M.. Urban Space for Pedestrians, Cambridge, MA; MIT press, 1975.

85. Gupta, R.C. and Jain, R.P., "Effect of Certain Roadway Characteristics on Accident Rates for Two-Lane, Two-Way Roads in Connecticut," Transportation Research Board Record 541, Washington, D.C., 1975, pp. 50-54.

86. Glennon, J.C., "Evaluation of Stopping Sight Distance Criteria", Texas Transportation Institute, Report 134-3, Aug. 1969.

87. Weaver, G.D. and Glennon, J.C., "Passing Performance Measurements Related to Sight Distance Design" Report 134-6, Texas Transportation Institute, July 1971.

88. Rober, V.J. and Howard, E.A., "Seeing with Motor Car Headlamps" Transactions of Illuminating Engineering Society, Vol. 33, No. 5, May 1938.

89. Bhise, V.D. et al., "Modeling Vision with Headlights in a Systems Context", SAE Report No. 770238, 1977.

90. Farber, E.I., and Bhise, V.D., "Development of a Headlight Evaluation Model", TRB Special Report 156, Transportation Research Board, 1974.

91. Traffic Engineering Handbook, Institute of Traffic Engineers, John E. Baerwalk (editor), Washington, D.C. 1965.

92. Railroad-Highway Grade Crossing Handbook, Technology Sharing Report, TS-78-214, Federal Highway Administration, Washington, D.C., August 1978.

93. Olson, P.L., et al. "Parameters Affecting Stopping Sight Distance", Interim Report. UM-HSRI-82-28 Highway Safety Research Institute, August 1982.