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How to Abbreviate on Highway Signs

R. DALE HUCHINGSON AND CONRAD L. DUDEK

This study investigated abbreviations for 80 traffic-related words by having a sample of drivers compose abbreviations and then having a different sample identify the word after being given the most popular abbreviation. Abbreviations were classified by percentage of subjects who correctly identified the words when presented alone and, again, when presented in the context of another word. The study identified strategies employed in abbreviating words, explored the relation between highly stereotyped abbreviations and success in understanding them, and recommended a set of abbreviations that likely could be used successfully on changeable-message signs.

Abbreviations are often necessary on signs when the word message exceeds the sign's capacity. Highway engineers have developed standardized abbreviations for use on street signs such as ST, BLVD, HWY, and N, S, E, and W. With the development of the changeable-message sign (CMS), the need has arisen for a dictionary of abbreviations for words that might be displayed in real-time traffic management. The line capacity of a CMS is often limited, and the substitution of a well-understood abbreviation for a lengthy word would be highly desirable. Therefore, an investigation was undertaken to determine words that could be abbreviated efficiently and abbreviations that would be understood immediately by 85 percent or more of the drivers.

It was suspected at the outset that many words could not be abbreviated efficiently (i.e., abbreviated to a character length not exceeding two-thirds of the length of the original word). For example, the word turnpike is eight characters long, and the most frequent or sterotyped abbreviation, TRNPK, is five-eighths or 62.5 percent of the total word length--only marginally acceptable by the two-thirds rule. A six-character abbreviation for a nine-character word would be inefficient and hardly worth the effort.

The military services employ many very brief abbreviations in communications (1); however, road signing cannot assume that the viewer will have been trained previously in the meaning of the abbreviation. The meaning should be clear on the driver's initial exposure to the abbreviation of the word.

Success in developing a dictionary of abbreviations depends heavily on the users familiarity with the words in the vocabulary. Staff (2) found that common meteorological terms could be abbreviated, usually in three or four characters (e.g., HURR, TOR, PRES, HUM, and ALT). However, the potential vocabulary for traffic-related words is more lengthy, and well-understood abbreviations for long words may require five or even six characters.

It would be convenient to prescribe a strategy by which any new word could be successfully abbreviated. There are essentially three strategies commonly employed: (a) key consonants, (b) first syllable, and (c) first letter. The first-letter strategy has been popularly used in dictionary abbreviations such as v for verb, n for noun, etc., and it is used for abbreviations of multiword organizations such as USO, CIA, and ROTC. However, the first-letter strategy usually implies learning and has less application to signing except mainly for the cardinal directions and highway numbers (I, US).

Selecting key consonants as a strategy is illustrated by BLVD, RD, CNTR, and PVMT. The vowels are omitted, as are certain consonants, but the first and last consonant in each syllable is usually retained.

The third strategy is taking the first syllable or the first three or four letters of the word. It implies that, if the reader can understand the first part of a word, the remainder may follow from context. Examples are TRAF, INFO, and EMER.

Which of these strategies is more commonly employed will depend somewhat on the words selected for abbreviation, but it was suspected that word length might also be a variable in strategy.

The objective of this research was principally to develop a set of highly stereotyped abbreviations that could be employed on a CMS without significant loss of information for unfamiliar drivers. Secondary interest was in a study of the strategies employed so that acceptable abbreviations could be developed for words not investigated.

METHOD

Subjects

The subjects were largely technicians and clerical workers with a high school education, aged 18 to 62, with an approximately equal number of males and females. All held current drivers licenses. Part 1 employed 41 subjects and part 2 employed 25 subjects.

Word List

Initially, the research staff developed a list of 63 words that had commonly appeared in messages developed for a human factors design guide (3). At the time the design guide was written, a need had arisen for acceptable abbreviations, but the staff was reluctant to abbreviate without evidence the abbreviation would be widely understood.

Later, an additional 25 words were added to the list in response to a request from traffic agencies in various states to provide a list of words they needed to abbreviate in the course of their unique signing operations. Several words were deleted, so the final word list comprised 80 words.

Procedure

Part 1 involved developing easily understood abbreviations by asking a sample of drivers to compose one abbreviation for each word. Instructions were to write the shortest abbreviation that would be understood by other motorists. Each word was presented in random order on a 3x5 card. Individual administration ensured no assistance from others.

Data analysis involved tallying the frequency of each abbreviation given. Frequency data were converted to percentage of the sample that gave each abbreviation. Only the most frequent abbreviation was retained for part 2 of the study. However, whenever two abbreviations of almost equal frequency was the most commonly given, both abbreviations were retained. Strategies employed in abbreviating the most common abbreviations were also noted.

Part 2 employed an entirely different sample of local drivers. The most common abbreviations from part 1 appeared individually on a set of cards in random order. Subjects were presented first with the abbreviation alone and were asked to give the word abbreviated. Responses were recorded on tape. If the word could not be ascertained, no help was provided by the test administrator and the subject either guessed or left it blank.

Later, the same subjects were given the same set of abbreviations, but this time each abbreviation was given in the context of an unabbreviated word. The unabbreviated word, termed a "prompt word," was one that would commonly appear either before or after the abbreviation in a signed message (e.g., MAJ Accident; CONST Ahead; 15 MIN).

Data analysis was the same as for part 1. The percentage of drivers correctly identifying the word was computed both for the word alone and with the prompt word.

RESULTS AND DISCUSSION

Part 1--Abbreviation Generation

There were 27 words for which 40 percent or more of the drivers gave the same abbreviation. Table 1 gives these words and abbreviations ranked in terms of the percentage giving the abbreviation. Also, Table 1 gives in code the type of strategy employed by the subjects in abbreviating the word.

Table 2 gives 31 words for which 26-39 percent of the drivers gave the indicated abbreviations. One point of interest was whether or not a word for which there was little agreement as to an abbreviation could still be correctly identified by a new sample of drivers.

Table 3 gives 22 words for which 25 percent or less of the drivers gave the same abbreviation. For these words, many different abbreviations were given and no single abbreviation had much support.

Table 4 gives the strategy used in abbreviating. The key-consonant strategy was adopted by 53.4 percent, the first-syllable strategy by 35.2 percent, and the first-letter strategy by only 11.4 percent. First letters were used mainly for cardinal directions and contractions, such as N BND for northbound.

Table 1. Words for which 40 percent or more gave the same abbreviation.

Word (n = 27)	Abbreviation	Strategy	Agreement (%)
Road	Rd	A	94
Information	Info	B	83
Minute(s)	Min	B	78
Highway	Hwy	A	72
West	W	C	72
South	S	C	67
East	E	C	67
North	N	C	67
Route	Rt	A	66
Boulevard	Blvd	A	61
State highway	St Hwy	A	56
Moderate	Mod	B	56
Temporary	Temp	B	54
Right	Rt	A	54
Lane	Ln	A	54
Miles(s)	Mi	B	51
Clear	Clr	A	51
Heavy	Hvy	A	51
Construction	Const	B	50
Roadwork	Rdwk	A	49
Level	Lvl	A	48
Blocked	Blkd	A	46
Condition	Cond	B	46
Freeway	Frwy	A	44
Major	Maj	B	41
Center	Cntr	A	41
Vehicle	Veh	B	41

Note: A = key consonant, B = first syllable, and C = first letter. The breakdown by strategy is as follows: A = 14, B = 9, and C = 4.

Table 2. Words for which 26-39 percent gave the same abbreviation.

Word (n = 31)	Abbreviation	Strategy	Agreement (%)
Bridge	Brdg	A	39
Prepare	Prep	B	39
Pavement	Pvmt	A	37
Chemical spill	Chem Spl	A/B	35
Tunnel	Tnl	A	35
Upper	Upr	A	35
Local	Loc	B	35
Minor	Mnr	A	34
Service	Serv	B	34
Maintenance	Maint	B	32
Left	Lft/L	A/C	32/27
Entrance	Ent	B	30
Speed limit	Spd Lmt	A/A	30
Normal	Nrml/Norm	A/B	30/26
Stadium	Stad	B	29
Ahead	Ahd	A	29
Beltway	Bltwy	A	29
Traffic	Traf	B	29
N,S,E,W-bound	N-bnd, etc.	C/A	29
Slippery	Slip	B	29
Exit	Ext	A	27
Pollution	Poll	B	26
Quality	Qlty/Qual	A/B	26/30
Blizzard	Blzrd	A	26
Stalled	Stld	A	26
Material	Mat	B	26
Lower	Lwr	A	26
Hazardous	Haz	B	26

Note: Strategies are defined in Table 1. The breakdown by strategy is as follows: A = 21, B = 14, and C = 5.

Table 3. Words for which 25 percent or less gave the same abbreviation.

Word (n = 22)	Abbreviation	Strategy	Agreement (%)
Congestion	Cong	B	24
Light	Lt	A	24
Travelers	Trvlrs	A	24
Delay	Dly	A	24
Flooded	Fld	A	24
Accident	Acc	B	22
Turnpike	Trnpk	A	22
Feeder	Fdr	A	22
Access	Acc	B	22
Warning	Wrng/Warng	A/B	22
Oversized	Ovrsz	A	22
Township	Twnshp	A	22
Emergency	Emer	B	19
Expressway	Exwy	A	17
Frontage	Front/Frntg	B/A	17
Express	Exp	B	17
Carpool	C-Pool	C	17
Interchange	Intrchng	A	15
Parking	Park	B	12
Express lanes			0
Pollutant			0
Motorcycle			0

Note: Strategies are defined in Table 1. The breakdown by strategy is as follows: A = 12, B = 8, and C = 1.

Table 4. Percentage that use various strategies for abbreviating traffic-related words.

		Group Percentages[a]		
Strategy	Total (%)	40 Percent or More	26-39 Percent	25 Percent or Less
A: key consonant	53.4	51.9	52.5	57.1
B: first syllable	35.2	33.3	35.0	38.1
C: first letter	11.4	14.8	12.5	4.8

[a] Breakdown of percentages using a strategy by the percentage agreeing to a preferred abbreviation.

Part 2--Abbreviation Understanding

In the second part of the study, subjects were given these abbreviations and asked to generate the word. Table 5 gives 21 words correctly identified by 88 percent or more of the drivers when the abbreviation was given alone or unprompted. Table 6 gives other words understood by 55-84 percent.

Tables 7, 8, and 9 give the results of a study in which the word was given with another word either before or after it. Those prompt words given after the abbreviation are shown with a large dot. Also shown is the percentage improvement from prompting. Certain words improved substantially from the prompt

Table 5. Abbreviations understood by 88 percent or more without prompt words.

Word (n = 21)	Abbreviation	Strategy	Understanding (%)
Freeway	Frwy	A	100
Highway	Hwy	A	100
Left	Lft	A	100
Parking	Pking	-[a]	100
Service	Serv	B	100
Traffic	Traf	B	100
Warning	Warn	B	100
Boulevard	Blvd	A	96
Speed	Spd	A	96
Center	Cntr	A	92
Entrance	Ent	B	92
Freeway	Fwy	A	92
Information	Info	B	92
Normal	Norm	B	92
Shoulder	Shldr	A	92
Emergency	Emer	B	88
Expressway	Expwy	A	88
Maintenance	Maint	B	88
Travelers	Trvlrs	A	88
Road	Rd	A	88
Slippery	Slip	B	88

Note: Strategies are defined in Table 1. The breakdown by strategy is as follows: A = 11 and B = 9.

[a] Last syllable.

Table 6. Abbreviations understood by 55-84 percent without prompt words.

Word (n = 28)	Abbreviation	Strategy	Understanding (%)
Bridge	Brdg	A	84
Quality	Qlty	A	84
Access	Accs	A	80
Condition	Cond	B	80
Parking	Park	B	80
Vehicle	Veh	B	80
Construction	Const	B	76
Downtown	Dwntn	A	76
Exit	Ex	B	76
Hazardous	Haz	B	76
Carpool	C-Pool	C	72
Chemical	Chem	B	72
Lane	Ln	A	72
Major	Maj	B	72
Mile(s)	Mi	B	72
Minute(s)	Min	B	72
Oversized	Ovrsz	A	72
Prepare	Prep	B	72
Turnpike	Trnpk	A	72
Ahead	Ahd	A	68
Express	Exp	B	64
Exit	Ext	A	64
Interchange	Intrchng	A	64
Roadwork	Rdwk	A	64
Right	Rt	A	62
Heavy	Hvy	A	60
Blocked	Blkd	A	56
Frontage	Frntg	A	56

Note: Strategies are defined in Table 1. The breakdown by strategy is as follows: A = 15, B = 12, and C = 1.

Table 7. Abbreviations understood by 100 percent and percentage improvement with prompt word indicated.

Abbreviation (n = 18)	Prompt Word	Strategy	Improvement[a] (%)
Ahd	Fog	A	32
Emer	•Vehicle	B	12
Ex	Next	B	24
Frwy, Fwy	(name)	A/A	0
Hwy	•(number)	A	0
Lft	Merge	A	0
Maj	•Accident	B	28
Mi	(number)	B	28
Pking	Coliseum	-[b]	0
Prep	•To Stop	B	28
Pvmt	Wet	A	88
Rd	(name)	A	12
Rt	Keep	A	38
Serv	•Road	B	0
Shldr	Soft	A	8
Spd	•Limit	A	4
Traf	•Advisory	B	0
W	(name/ number)	C	88

Notes: • = prompt word given after abbreviation.
Strategies are defined in Table 1. The breakdown by strategy is as follows: A = 10, B = 7, and C = 1.

[a] Percentage improvement over understanding without prompt word.
[b] Last syllable.

Table 8. Abbreviations understood by 96 percent and percentage improvement with prompt word indicated.

Abbreviation (n = 15)	Prompt Word	Strategy	Improvement (%)
Blkd	Lane	A	40
Blvd	(name)	A	0
Brdg	(name)	A	12
Cntr	•Lane	A	4
Chem	•Spill	B	24
Ent	Freeway	B	4
Maint	•Work	B	8
Ovrsz	•Load	A	24
Qlty	Air	A	12
Ext	Next	A	32
Info	Traffic	B	4
Trvlrs	•Warning	A	8
Slip	•Pavement	B	8
Veh	Stalled	B	12
Warn	Blizzard	B	-4

Notes: • = prompt word given after abbreviation.
Strategies are defined in Table 1. The breakdown by strategy is as follows: A = 8 and B = 7.

Table 9. Abbreviations understood by 88-92 percent and percentage improvement with prompt word indicated.

Abbreviation (n = 13)	Prompt Word	Strategy	Improvement (%)
Accs	•Road	A	8
Const	•Ahead	B	16
Exp	•Lane	B	24
Expwy	(name)	A	0
Haz	•Driving	B	12
I	•(number)	C	44
Min	•(number)	B	16
Mnr	•Accident	A	72
Norm	•Traffic	B	0
Rt	Best	A	48
Trnpk	(name)	A	20
Twnshp	•Limits	A	72
Upr	•Level	A	48

Notes: • = prompt word given after abbreviation.
Strategies are defined in Table 1. The breakdown by strategy is as follows: A = 7, B = 5, and C = 1.

word, while the words with high understanding when seen alone obviously did not improve much with prompting.

A question of interest related to how well does giving the same abbreviation in part 1 correlate with understanding the abbreviation in part 2. In other words, Is agreement on an abbreviation a valid predictor of understanding the abbreviation? It was found that, for those 16 words that were given by 54 percent or more of the drivers in part 1, all except 2 were understood by 85 percent or more when given with another word. The two exceptions almost reached the criteria. TEMP for temporary was at 84 percent and MOD for moderate was at 75 percent understanding. Therefore, it was concluded that highly stereotyped abbreviations would be understood with a prompt word.

This finding did not hold for the entire list of 80 words (i.e., the higher the initial stereotyping percentage, the higher the understanding percentage was not found to be true). A product-moment correlation between the two distributions yielded a value of only +0.1729. Hence, for all words, the degree of stereotyping is not a strong predictor of understanding in context. Many words with poorly stereotyped abbreviations in part 1 were found to be easily understood, especially when seen with a prompt word.

An interesting finding regarding strategy for abbreviating words was that the length of the word was a factor. For words of nine letters or more, the first-syllable strategy was employed 64 percent of the time in abbreviating. Examples are CONG for congestion and TEMP for temporary. Staff (2), who studied long meteorological words, also found that the first-syllable strategy was used for 16 of the 20 words she studied. Shorter words (5 to 7 letters) typically were abbreviated by key consonants.

There were some other interesting findings regarding the length of abbreviation. Seventy percent of the five-letter words were abbreviated in three letters and 25 percent in two letters, excluding cardinal directions. Words with six to eight letters were abbreviated in three or four letters. Words requiring five or more letters to abbreviate had very low agreement on an abbreviation, i.e., 29 percent or less. Examples were BLTWY, TRVLRS, TRNPK, FRNTG, and OVRSZ. Note that these words were all abbreviated by the key-consonant strategy. In general, long abbreviations are not efficient and should be avoided if they exceed two-thirds of the word's length.

CONCLUSIONS AND RECOMMENDATIONS

This study identified 21 abbreviations that are understood by more than 85 percent of the drivers when the abbreviation is given alone and an additional 47 words understood by 85 percent when given with a common prompt word. These words may be considered by agencies for use on CMSs. Those that require a prompt word for understanding should be used only if the exact prompt word is used. If other words are substituted, the message should be tested before using the abbreviation.

When abbreviating new words not in the word list, the first-syllable strategy will likely result in better understanding if the word is nine letters or more. For short words of five to seven letters, the key-consonant strategy will likely lead to better understanding, although there are exceptions.

Whenever the best-understood abbreviation is longer than two-thirds of the word itself, abbreviating is discouraged. Some words cannot be efficiently abbreviated, and the agency should consider a synonym, such as FOG for reduced visibility. When the first-syllable strategy results in another word such as RED for reduced or POLL for pollution, the abbreviation should be avoided also.

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Studies of Highway Advisory Radio Messages for Route Diversion

CONRAD L. DUDEK, R. DALE HUCHINGSON, AND R. QUINN BRACKETT

A series of in situ controlled field studies were conducted to establish the effectiveness of highway advisory radio (HAR) message characteristics in aiding motorists to negotiate diversion routes. The research investigated the effects of language style, message load, manner of message repetition, use of landmarks and other route descriptors, and driver familiarity with the street system. Drivers listened to simulated HAR messages while on a metropolitan Interstate and were then requested to negotiate complex diversion routes by recalling the information given. Recommendations for designing HAR messages are given based on the study findings.

Motorist information systems perform a critical role in the successful operation of real-time corridor control systems in metropolitan areas. Flexible systems can provide information that enables drivers to use the highway system more efficiently and safely. One method that provides the flexibility to transmit a variety of information is the highway advisory radio (HAR). A HAR system is composed of a low-frequency, low-power (10- to 50-W) transmitter and an antenna that can be positioned beside a roadway to give drivers up-to-the-minute travel information via their AM radios. The number and types of messages are limited only by motorist information needs and processing capability.

In previous research (1), the Texas Transportation Institute investigated certain characteristics of HAR messages in the laboratory with the objective of developing design criteria for these messages. The laboratory findings dealt with subjects' ability to recall information under various message conditions and with subject preferences. Prior to recommending design criteria, it was first necessary to validate previous findings by investigating these message characteristics under actual driving conditions.

The experimental protocol for the series of studies reported in this paper involved selecting drivers who were generally unfamiliar with a particular diversion route. With a test administrator in the car, they would drive on a local Interstate through a metropolitan area and receive a tape-recorded HAR message. The message advised of an accident ahead and a diversion route. They would then attempt to recall pertinent guidance information in the message and drive the diversion route. The administrator recorded any errors in following the route (e.g., missed turns, turning too soon, etc). In addition, the administrator asked post-test questions, as applicable.

There were four major HAR route-diversion studies conducted in San Antonio, Texas, during summer 1980. The first was concerned with two issues: message load and language style. The second study dealt with the effects of repeating either the entire route description or parts of the description. The third study dealt with the effects of mentioning in the message easily observable landmarks, traffic signals, and businesses along the route. It was hypothesized that these route descriptors would aid the drivers by assuring them that they were still on the correct diversion route. The final study was concerned with message criteria for drivers already familiar with the street system in the area. It was assumed that familiar drivers would require less information in the message in order to follow the diversion route. [Note, these studies were part of a Federal Highway Administration (FHWA) human factors research project (2).]

STUDY 1: MESSAGE LOAD AND LANGUAGE STYLE

The first study was a joint investigation of two major variables in message design that were previously defined and investigated in the laboratory (1). These variables were message load and language style. A brief introduction to the meaning of these concepts is necessary.

Message Load

Message load, as used in this research, refers to the number of informational elements that must be recalled by the driver to successfully negotiate the route. For example, if the diversion route involved exiting the Interstate and following a parallel arterial to the right of the Interstate, the message would require, at a minimum, eight units of information:

1. Where to exit (the street name);
2. Which direction, left or right, to turn on the street;
3. Name of the parallel arterial;
4. Direction of turn--left or right;
5. Name of the return street;
6. Direction of turn;
7. Name of the Interstate; and
8. Direction of turn to reenter and continue in the original direction.

Note that message load refers here to a demand on the driver. It is possible that some of the above-listed information could be partly implied in the message without actually stating it (e.g., reentering the Interstate). Some feel that the direction of turns, although stated, are also fairly obvious given the initial turn direction from the Interstate. Nevertheless, the diversion route requires knowing eight units of stated or implied information.

Message load has been used also to refer to all information given, including the problem that necessitates diversion. In this research, only information needed to negotiate the route was included in the assessment of units of information.

Figure 1 presents the routes selected for this research. Note that a six-unit problem requires negotiating only three turns and recalling only three streets: Jackson-Keller, San Pedro, and I-410. The eight-unit problem has four turns and three street names plus I-410--a total of eight pieces of information to be learned. Similarly, the 10-unit problem has 5 turns and 5 legs, including I-410.

The experimental question was, Could a driver listen to a radio message and then recall the information sufficiently well to negotiate these routes without error, or would the longer (8- and 10-unit) routes require recall of too much information?

Language Style

In previous research (1), it was found that recalling the route was improved by simplifying the language in the message. Rather than using a long, wordy message with complete sentences and many adjectives, it was better to use a terse message that contained only the information that needed to be recalled. The wordy message was termed "conversational", an intermediate level was termed "short form", and the briefest language style was called "staccato".

In the first study, nine messages were investigated that involved three language styles in combination with three levels of message load.

Method

Subjects

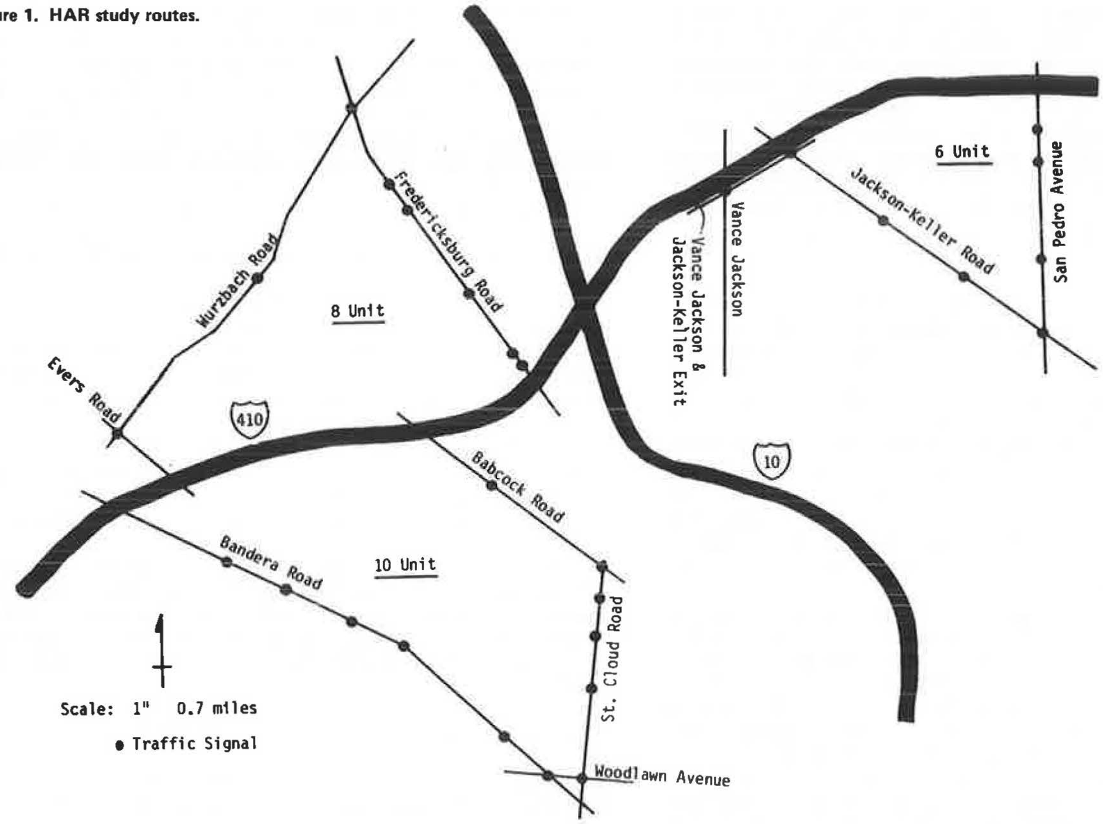
Fifty-four drivers were recruited from the San Antonio area. Each stated they were unfamiliar with the roadway system in the section of the city selected for the study. The subjects were selected to be representative of the current driving population with respect to age, sex, education, and years of driving experience.

Messages

Figures 2 and 3 present examples of messages played to the driver in study 1. Figure 2 presents three messages, all in the short-form language style. Note that they deal with the 6-, 8-, and 10-unit problems shown previously in Figure 1. The route description was repeated in each message.

Figure 3 also presents examples of language style, all for the six-unit problem. Note the key differences. For example, the staccato message states "overturned truck ahead"; the short form states "there is an overturned truck ahead"; and the conversational style states the same plus "on Interstate 410 ahead". The conversational style also states "you are advised to exit, etc.", rather than merely "exit and take Jackson-Keller". There are other interesting but unessential words and phrases.

Figure 1. HAR study routes.



Experimental Design and Procedure

The drivers were assigned to three groups of 18 each. Each group was matched with respect to the above-mentioned demographic characteristics. Each group received messages in one of the language styles and drove three test routes that involved 6-, 8-, and 10-unit problems. The order of test routes was counterbalanced across drivers. After completing the three routes, the administrator played taped messages of all three language styles and subjects were to rank the language styles in order of preference.

Results

The table below gives the findings of the first study (note, F = frequency and % = percentage of subjects making an error):

Language Style	Message Load: 6 Unit F	%	8 Unit F	%	10 Unit F	%	Total F	%
Staccato	1	6	2	11	8	44	11	41
Short form	2	11	3	17	2	11	7	26
Conversational	2	11	1	6	6	33	9	33
Total	5	18	6	22	16	60	27	100

The data given represent both frequencies of error in route negotiation and conversion of frequencies to percentages. One of the 18 drivers making an error is equivalent to approximately a 6 percent error (rounded to the nearest whole number).

It may be noted that 16 of the 27 errors (60 percent) occurred with the 10-unit problem whereas, by chance, only 9 (33 percent) would have occurred on this course. Chi-square tests found that the distribution exceeded chance probability ($p < 0.05$). It was concluded that the 10-unit problem was more conducive to errors.

The percentage differences in language style, shown in the total column, did not differ significantly. For the 6- and 8-unit problems, errors were about the same for each style; however, for the 10-unit problem, the short form had only 2 errors while the other language styles had 14 total errors.

The table below gives the results of the preference study:

Language Style	No. of Subjects: 1st Choice	2nd Choice	3rd Choice	Avg. Ranking Points
Staccato	17	18	19	2.04
Short form	23	21	10	1.76
Conversational	14	14	25	2.20

Twenty-three of the 54 drivers rated the short form the best style while only 10 rated it the poorest language style. The expected value in each cell was 18. Although it would be likely for drivers to prefer the language style that they had heard in the test messages, it is clear that a number of drivers preferred the short form that they had not previously heard and followed. The data provide some support for the short form and dislike for the conversational style. Because brevity in messages permits the HAR system to recycle more times within the broadcast area, the use of a terse message style is recommended.

STUDY 2: MESSAGE REPETITION

In previous research (1), a new concept was invented to describe repeating part of a message. The concept of "internal redundancy" refers to repeating the street name immediately after it is first mentioned in a message. Figure 4 (top) illustrates internal redundancy in a short-form version of the 10-unit problem. Note that the names Bandera, Woodlawn, St. Cloud, and Babcock are each given twice in the message.

The other technique for mentioning street names twice would be simply to give them once and then state "I repeat" and give them a second time. Figure 4 (bottom) illustrates this form of repetition, termed "external redundancy".

Method

Subjects

Eighteen drivers who had not participated in study 1 were selected for participation. They were unfamiliar with the street system and were comparable in age, sex, education, and driving experience to subjects in study 1.

Messages

The messages illustrated in Figure 4 were all in short-form language style. Messages were all in the internal-redundant format and data were compared with that in study 1, which employed the external-redundant format. The advisory portion of the message (beginning with the word "Exit") consisted of only about two-thirds as many words as the completely repeated advisory in study 1.

Experimental Design and Procedure

Each of the 18 subjects drove a 6-, 8-, and 10-unit course as had the drivers in study 1. Procedures were identical except for the message being internally redundant.

Results

The table below gives the frequency of errors committed by the 18 subjects as compared with their counterparts in study 1 who heard the message with

Figure 2. Example of message load.

Six Units
- ATTENTION EASTBOUND INTERSTATE 410 TRAFFIC
- THERE IS AN OVERTURNED TRUCK AHEAD
- TO AVOID MAJOR DELAY,
- EXIT AT JACKSON-KELLER,
  AND TAKE THE FOLLOWING ROUTE:
- TURN RIGHT ON JACKSON-KELLER,
- THEN TURN LEFT ON SAN PEDRO,
- AND PROCEED BACK TO INTERSTATE 410 WEST.

I REPEAT,

- EXIT AT JACKSON-KELLER,
  AND TAKE THE FOLLOWING ROUTE:
- TURN RIGHT ON JACKSON-KELLER,
- THEN LEFT ON SAN PEDRO,
- AND PROCEED BACK TO INTERSTATE 410 WEST.

Eight Units
- ATTENTION WESTBOUTND INTERSTATE 410 TRAFFIC
- THERE IS A MAJOR ACCIDENT AHEAD
- TO AVOID MAJOR DELAY,
- EXIT AT FREDERICKSBURG,
  AND TAKE THE FOLLOWING ROUTE:
- TURN RIGHT ON FREDERICKSBURG,
- THEN LEFT ON WURZBACH
- THEN TURN LEFT AGAIN ON EVERS
- AND PROCEED BACK TO INTERSTATE 410 WEST

I REPEAT,

- EXIT AT FREDERICKSBURG,
  AND TAKE THE FOLLOWING ROUTE:
- TURN RIGHT ON FREDERICKSBURG,
- THEN LEFT ON WURZBACH,
- AND THEN LEFT AGAIN ON EVERS,
- AND PROCEED BACK TO INTERSTATE 410 WEST,

Ten Units
- ATTENTION EASTBOUND INTERSTATE 410 TRAFFIC
- THE FREEWAY IS BLOCKED AHEAD
- TO AVOID MAJOR DELAY,
- EXIT AT BANDERA,
  AND TAKE THE FOLLOWING ROUTE:
- TURN RIGHT BANDERA,
- THEN LEFT ON WOODLAWN
- THEN LEFT ON ST. CLOUD
- THEN LEFT AGAIN ON BABCOCK
- AND PROCEED BACK TO INTERSTATE 410 EAST.

I REPEAT,

- EXIT AT BANDERA,
  AND TAKE THE FOLLOWING ROUTE:
- TURN RIGHT ON BANDERA,
- THEN LEFT ON WOODLAWN,
- THEN LEFT ON ST. CLOUD,
- AND THEN LEFT AGAIN ON BABCOCK,
- AND PROCEED BACK TO INTERSTATE 410 EAST.

Figure 3. Example of message style (six-unit diversion route).

Staccato
- ATTENTION EASTBOUND INTERSTATE 410 TRAFFIC
- OVERTUNRED TRUCK AHEAD
- TO AVOID MAJOR DELAY,
- EXIT AT JACKSOKN-KELLER,
- TURN RIGHT ON JACKSON-KELLER,
- TURN LEFT ON SAN PEDRO,
- BACK TO INTERSTATE 410 WEST.

REPEAT,

- EXIT AT JACKSON-KELLER,
- TURN RIGHT ON JACKSON-KELLER,
- LEFT ON SAN PEDRO,
- BACK TO INTERSTATE 410 WEST.

Short Form
- ATTENTION EASTBOUND INTERSTATE 410 TRAFFIC
- THERE IS AN OVERTURNED TRUCK AHEAD
- TO AVOID MAJOR DELAY,
- EXIT AT JACKSON-KELLER,
  AND TAKE THE FOLLOWING ROUTE:
- TURN RIGHT ON JACKSON-KELLER,
- THEN LEFT ON SAN PEDRO,
- AND PROCEED BACK TO INTERSTATE 410 EAST

I REPEAT

- EXIT AT JACKSON-KELLER,
  AND TAKE THE FOLLOWING ROUTE:
- TURN RIGHT ON JACKSON-KELLER
  THEN LEFT ON SAN PEDRO,
- AND THEN LEFT AGAIN ON EVERS,
- AND PROCEED BACK TO INTERSTATE 410 EAST.

Coversational
- ATTENTION EASTBOUND INTERSTATE 410 LOP
- THERE IS AN OVERTURNED TRUCK ON INTERSTATE 410 AHEAD.
- TO AVOID MAJOR DELAY,
- YOU ARE ADVISED TO EXIT AT JACKSON-KELLER ROAD
  AND TAKE THE FOLLOWING ROUTE:
- TURN RIGHT ON JACKSON-KELLER ROAD
  AND CONTINUE TO SAN PEDRO AVENUE
- THEN TURN LEFT
- AND DRIVE BACK TO INTERSTATE 410
  TO CONTINUE YOUR EASTBOUND TRIP

I REPEAT,

- YOU ARE ADVISED TO EXIT AT JACKSON-KELLER
  AND TAKE THE FOLLOWING ROUTE:
- TURN RIGHT ON JACKSON-KELLER
  AND CONTINUE TO SAN PEDRO AVENUE
- THEN TURN LEFT
- AND DRIVE BACK TO INTERSTATE 410
  TO CONTINUE YOUR EASTBOUND TRIP.

complete repetition of the route description:

Type of Redundancy	No. of Errors by Message Load 6 Unit	8 Unit	10 Unit	Total Errors
Internal	1	1	2	4
External	2	3	2	7
Total errors	3	4	4	11

It may be recalled that the fewest errors in study 1 also occurred with the short form and, hence, substantial improvement was not possible. The differences in errors were not statistically significant. The only conclusion possible from the study is that the techniques of redundancy were equally effective under the conditions of investigation.

STUDY 3: MESSAGE ROUTE DESCRIPTORS

In describing to others a particular route within a metropolitan area, a person often mentions landmarks or prominent environmental features that can be seen at a great distance and can be used either to confirm that one is on the correct route or to prepare the driver to turn. Examples of landmarks are a store, service station, or hospital. Also, the number of traffic lights through which the driver passes before turning is a commonly used descriptor.

In this study, the 10-unit problem was modified to include two landmarks and two traffic light notations. The 10-unit problem was selected because study 1 found a high percentage of errors and, hence, a need for improvement.

Method

Subjects

Eighteen new drivers were recruited for participation. They matched the previous subjects in demographic characteristics and were unfamiliar with the roadway system.

Messages

Figure 5 presents the message given with route descriptors. It is a modification of the 10-unit message from study 1. It necessarily had the complete sentence structure of conversational style but has added information that could aid in route negotiation.

Experimental Design and Procedure

The procedure was the same as the previous studies except for the content of the message given.

Results

The table below shows that only 2 of the 18 subjects with the route descriptor message made an error:

Message	Subjects Who Made Errors No.	Percent
Route descriptors	2	11
Short form	2	11
Conversational	6	33
Staccato	9	44

This performance equalled the best performance in study 1 (the short form). A succession of binomial tests found that it was significantly better than the staccato and conversational messages in study 1 ($p < 0.05$). This finding was interesting, since the message was substantially longer than previous messages. It appears that the landmarks and number of traffic lights did help in negotiating a long route.

Several subjects reported some difficulty in counting numbers of traffic lights and were confused when the traffic lights were flashing rather than operational.

STUDY 4: DRIVER FAMILIARITY AND TURN DESCRIPTORS

Previous HAR studies (3) have shown that unfamiliar

Figure 4. Examples of internal and external redundancy in an HAR message for a 10-unit problem.

Example - Internal Redundancy

- ATTENTION EASTBOUND INTERSTATE 410 TRAFFIC
- THE FREEWAY IS BLOCKED AHEAD
- TO AVOID MAJOR DELAY,
- EXIT AT BANDERA
  AND TAKE THE FOLLOWING ROUTE:
- TURN RIGHT ON BANDERA
- AND CONTINUE TO WOODLAWN
- TURN LEFT ON WOODLAWN,
- AND CONTINUE TO ST. CLOUD
- THEN TURN LEFT ON ST. CLOUD
- AND CONTINUE TO BABCOCK
- AND THEN TURN LEFT AGAIN ON BABCOCK
- AND PROCEED BACK TO INTERSTATE 410 EAST

Example - External Redundancy

- ATTENTION EASTBOUND INTERSTATE 410 TRAFFIC
- THE FREEWAY IS BLOCKED AHEAD
- TO AVOID MAJOR DELAY,
- EXIT AT BANDERA,
  AND TAKE THE FOLLOWING ROUTE:
- TURN RIGHT AT BANDERA
- TURN LEFT AT WOODLAWN
- LEFT AT ST. CLOUD
- LEFT AGAIN AT BABCOCK
- AND PROCEED BACK TO INTERSTATE 410 EAST

I REPEAT,

- EXIT AT BANDERA,
  AND TAKE THE FOLLOWING ROUTE:
- TURN RIGHT AT BANDERA
- TURN LEFT AT WOODLAWN
- LEFT AT ST. CLOUD
- LEFT AGAIN AT BABCOCK
- AND PROCEED BACK TO INTERSTATE 410 EAST

Figure 5. Message with route descriptors.

10-Unit Route

- ATTENTION EASTBOUND INTERSTATE 410 TRAFFIC
- THE FREEWAY IS BLOCKED AHEAD
- TO AVOID MAJOR DELAY,
- EXIT AT BANDERA
  AND TAKE THE FOLLOWING ROUTE:
- TURN RIGHT ON BANDERA
  AND CONTINUE TO THE SIXTH TRAFFIC LIGHT, WOODLAWN
- THERE IS A WESTERN AUTO STORE ON THE LEFT AT WOODLAWN
- TURN LEFT ON WOODLAWN
  AND CONTINUE TO ST. CLOUD
- TURN LEFT ON ST. CLOUD AND GO THE THE FOURTH TRAFFIC LIGHT, BABCOCK
- THE MORNINGSIDE MANOR REST HOME IS ON THE LEFT JUST BEFORE BABCOCK
- AT BABCOCK TURN LEFT AGAIN AND PROCEED BACK TO INTERSTATE 410 EAST

drivers must be told the names of streets at which they must turn and also the direction of turn (i.e., left or right). However, oftentimes a vast majority of drivers are local commuters who are intimately familiar with the major streets in an area where they might be diverted.

A HAR message could be greatly simplified and shortened if the direction of turning movements was omitted from the message. Given that drivers expect to be diverted first away, then parallel, and finally back to the Interstate, the only information they would really need would be a listing of the streets where they should turn. It was postulated that familiar drivers could negotiate a 10-unit diversion route given a message such as "Take the following route: Bandera to Woodlawn, to St. Cloud, to Babcock, and back to Interstate 410 East".

Method

The method was the same as previous studies. Eighteen drivers were selected who stated they were highly familiar with the street system in the area near the study routes. They were tested on the 8- and 10-unit routes. The route description gave only the names of streets with no mention of direction of turn.

Results

The table below gives the frequency of errors in comparison with the best performance for an 8- and 10-unit problem in study 1 with unfamiliar drivers:

	No. of Errors by Message Load	
Familiarity	8 Unit	10 Unit
Familiar drivers--no turns given	0	2
Unfamiliar drivers given turns	1	2

Only 1 of the 18 drivers made an error (and that on the 10-unit problem) while negotiating a familiar route, even without the message mentioning turns. In study 1, no group exceeded this performance, although a binomial test found no significant differences in error frequency.

It was concluded that familiar drivers may be given briefer messages that do not include turn direction. They generally do not encounter problems in following a route that requires as many as five turns and five street names.

CONCLUSIONS AND RECOMMENDATIONS

The results of four driver-performance studies indicated that error frequency was generally low with HAR descriptions of diversion routes. The frequency of drivers making errors reached its peak on trials with a 10-unit message load. However, it was demonstrated that performance even at this level could be improved by employing a short-form language style in the message and by adding landmarks and other route descriptors.

Recommendations are as follows:

1. Although language style was not found to be critical, a terse message style was preferred by drivers. Unnecessary wordiness is inefficient in communicating messages in a HAR system.
2. If unfamiliar drivers are diverted, the routes should not exceed four turns and four names, including the Interstate (eight-unit problems).
3. The description of the diversion route should be repeated at least once, either with internal or external redundancy or with both.
4. Prominent landmarks may be mentioned in a HAR message whenever there is a risk the driver may not see the place to turn. The number of traffic lights is useful but should be avoided whenever any of the lights are flashing.
5. When the driving population is known to be largely commuters or highly familiar with the area, the route description may be shortened by omitting turn directions.

ACKNOWLEDGMENT

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Evaluation of Driver Behavior at Signalized Intersections

ROBERT H. WORTMAN AND JUDSON S. MATTHIAS

Time-lapse photography was used to study driver behavior associated with the traffic signal change interval at a total of six intersections in the Phoenix and Tucson metropolitan areas. In addition, nighttime studies were conducted at two of these intersections. An evaluation of the time-lapse film permitted the determination of the approach speeds of the vehicles, the average deceleration rates of the stopping vehicles, the perception-reaction times of the drivers of the stopping vehicles, and the distance that the vehicle was from the intersection at the onset of the yellow interval. The distance from the intersection was measured for both the stopping vehicles as well as those that proceeded through the intersection. The results of the study indicated that the mean deceleration rates at the six sites ranged from 7.0 to 13.9 ft/s/s, and the mean value for all observations was 11.6 ft/s/s. The observed mean perception-reaction time was approximately 1.3 s while the 85th percentile times ranged from 1.5 to 2.1 s. Comparisons of intersections with yellow only versus yellow plus all-red intervals produced mixed results in terms of differences in observed behavior. Even for intersections with the same change interval design, there were cases where the observed deceleration rates were significantly different.

The failure to properly recognize and understand driver behavior in signalized-intersection design can contribute to operational and safety problems. A review of 1980 traffic accidents in Arizona reveals that approximately 3 percent of the reported accidents had disregard of a traffic signal listed as a contributing circumstance (1).

The recognition of driver behavior is particularly critical in the design of the traffic signal change interval. The change interval has two recognized purposes. The first is to advise the motorist that the red interval is about to commence, and the second is to allow vehicles that have legally entered the intersection sufficient time to clear the point of conflict prior to the release of opposing pedestrians or vehicles. The determination of the change interval considers driver perception-reaction time and vehicle deceleration rates; the use of unrealistic values for these factors would potentially affect driver compliance, safety, and even intersection capacity. In the equations used in the determination of the duration of the change interval, a perception-reaction time of 1 s and a deceleration rate of 10 ft/s/s are suggested (2). The latter value has been decreased from 15 ft/s/s, which was used in practice for a number of years.

The current Arizona Department of Transportation policy (3) indicates that 8 and 12 ft/s/s are the upper and lower limits for deceleration rates that are used in establishing the change interval duration. This range provides some degree of latitude in applying engineering judgment to the determination of the change interval; however, the policy further states that 10 ft/s/s should normally be used.

In recent years, several studies have focused on driver behavior during the yellow signal interval. For example, Williams (4) reported that a study of an intersection in New Haven, Connecticut, revealed that the average maximum deceleration rate for stopping vehicles was 9.7 ft/s/s. Other studies (5,6) have suggested that a reasonable deceleration rate would be in the magnitude of 10 ft/s/s.

Two previous studies included field measurements of driver reaction time. In an early study of the change interval problem, Gazis, Herman, and Maradudin (7) found a mean reaction time of 1.14 s based on a sample of 87 observations. A study in 1966 by Jenkins (8) contained a sample of 21 observations of reaction time. An analysis of the data sample obtained by Jenkins reveals that the mean reaction time was approximately 1.4 s.

The Transportation and Traffic Engineering Handbook (2) states that excessively short or long yellow intervals are undesirable; thus, the common practice is to use yellow intervals of 3-5 s. If a longer duration is required to clear the intersection before the cross traffic enters, an all-red period can be used in addition to the selected yellow interval. The Arizona Department of Transportation policy (3) indicates that a yellow interval of up to 6 s may be used. If a longer change interval is required, then an all-red interval should be used.

In Arizona, as in most states, it is legal for the vehicle to enter the intersection during the yellow signal indication. It is not necessary for the vehicle to have cleared the intersection prior to the onset of the red signal indication.

The purpose of this study focused on examining driver behavior that is related to the change interval. The intent was to document driver behavior parameters and possible variations in behavior that exist in Arizona. More specifically, the study objectives focused on the determination of the following items:

1. The actual deceleration rates and the range of deceleration rates that are used by drivers,
2. Possible differences in driver behavior due to the intersection environment, and
3. The effect of the use of the all-red phase on driver behavior; in addition, the study provided information about the perception-reaction times of drivers as well as measures of driver compliance in terms of the signal indications.

It should be noted that it was not feasible to conduct before-and-after studies of the effects of changes in the change interval design and duration at a particular intersection. The study was limited, therefore, to examining behavior at existing intersections with existing signal timing.

Driver behavior associated with the change interval is certainly a complex phenomenon in that there are numerous facets to the overall problem. It was not the intent of this particular study to include an evaluation of the current practice used in the determination of change intervals.

METHOD OF STUDY

Time-lapse photography was used to record the daytime driver behavior associated with the change interval at six intersections. Two of the six intersections were also observed during the nighttime period. Vehicles approaching the intersection were filmed for a few seconds prior to the onset of the yellow, during the change interval, and until the vehicle either stopped or cleared the intersection. Given the onset of the yellow signal indication, the study focused on the first vehicle to stop and the last vehicle to pass through the intersection. Where multiple approach lanes were involved, this information was included for each lane. It was not possible to determine such information such as the age, sex, experience, and route familiarity of the driver.

The camera was located so that it was possible to record the intersection and the signal indication as well as the operation of approaching vehicles within 350-400 ft of the intersection. A super 8-mm movie camera with a zoom lens and a 16-mm movie camera were used for data collection. Because of the age of the 16-mm equipment, it proved to be less reliable and more expensive to operate than the super 8-mm unit; thus, it was used only on a limited basis. Also, higher-speed film for the night observations was readily available for the super 8-mm camera; therefore, that camera was used for all of the nighttime studies. The cameras were operated at a frame speed of 18 frames/s to increase the accuracy of the time measurements.

The day filming was accomplished by using a Kodak Kodachrome color film. This made it possible to easily distinguish the signal indications as well as when brake lights were illuminated on a vehicle. For the night filming, a Kodak Ektachrome color film was used. Although it was not possible to see the entire vehicle at night, the lights on the vehicle and the traffic signals were recorded on the film.

If available, the intersection was filmed from a nearby building or structure. If such a facility was not available at a particular location, the Arizona Department of Transportation furnished a truck with an elevating platform that would extend to a height of approximately 30 ft. The camera was placed on a tripod, which resulted in a total height of about 35 ft. The truck was parked in a parking lot or vacant area approximately 400-450 ft from the intersection. Where the truck was employed for filming, it was located about 30-50 ft from the edge of the roadway. There was concern that the presence of the truck might influence the behavior of the drivers. Based on observations by the study teams at the sites, there was no evidence that the drivers were cognizant of the filming activities.

Distances from the intersection were noted by using reference points on the roadway. Generally, strips of tape were placed at 50-ft intervals along the edge of the approach. In some cases, the dashed-lane striping was measured and used as the reference for distance from the intersection.

Site Selection

The following six intersections in the Phoenix and Tucson metropolitan areas were selected for study:

1. Phoenix metropolitan area: University Drive and Rural Road (located in Tempe), Southern Avenue and McClintock Drive (located in Tempe), and US-60 and Greenfield Road (located in Mesa); and
2. Tucson metropolitan area: First Avenue and Roger Road, Sixth Street and Campbell Avenue, and Broadway Boulevard and Columbus Boulevard.

Only one approach at each of the intersections was studied, and the study approach was located on the first street that is shown in the list of intersections. Table 1 summarizes the characteristics of each of the intersection approaches that were observed.

Major difficulties were encountered in the selection of intersections that were to be used for study observations. These difficulties served to severely limit the choice of intersections that could even be considered. Even with the option of placing the camera in a nearby building or on a truck with an elevating platform, it was frequently difficult to find a suitable building or an acceptable place to locate the truck. Also, if the traffic signal at an intersection is operated as part of an areawide system, approaching vehicles frequently may not be in proximity to the intersection at the onset of the yellow interval. The same may be true for signalized intersections in outlying areas that have traffic-actuated controllers. When such conditions exist, the probability of obtaining a data sample is relatively small, and the time required for data collection is greatly increased. Jurisdictions are reluctant to temporarily isolate a signal from the control system because of liability questions.

Filming of Intersection Approaches

Because of the arrangements that were necessary for access to buildings or to use a truck with a platform, filming activities were accomplished in periods of several hours duration. For example, a study team would frequently film an approach for a 6- to 8-h period. For the night observations, filming was undertaken during the months of December and January; thus, data collection generally began about 6:00 p.m. and was terminated about 11:00 p.m.

It took about 12-16 h of filming at each site to obtain a sample size of approximately 100 stopping vehicles. Even with this filming period, there were some intersection approaches that yielded less than 100 samples when the film was analyzed. It might seem that this duration of filming is somewhat excessive for such a sample size; however, there are two explanations. First, cycle lengths were sometimes as high as 120 s. Second, there were numerous cases for which there were no vehicles in the proximity of the approach at the onset of the yellow interval. Because of the duration of filming activities, it was not possible to focus on particular periods of time during the day. The observations that were made, therefore, represent a cross section of the traffic conditions at an intersection.

Data Reduction

By using the distance reference points that were

Table 1. Characteristics of study sites.

Site	Estimated ADT[a]	Change Interval[b] (s)	Approach Configuration	Left-Turn Signalization
University Drive* and Rural Road	24 000	5Y	Two lanes plus exclusive left-turn lane	Exclusive left-turn phase
Southern Avenue* and McClintock Drive	22 100	5Y	Three lanes plus exclusive left-turn lane	Exclusive left-turn phase
US-60* and Greenfield Road	24 200	5Y+3AR	Three lanes plus exclusive left-turn lane	Exclusive left-turn phase
First Avenue* and Roger Road	21 400	3Y+2AR	Two lanes plus exclusive left-turn lane	Turns permitted on a permissive basis during through movement
Sixth Street* and Campbell Avenue	18 300	3Y+2AR	Two lanes plus exclusive right- and left-turn lanes	Turns permitted on a permissive basis during through movement except in peak hours when left-turn lane is used as a reversible lane
Broadway Boulevard* and Columbus Boulevard	35 800	3.6Y + 2AR	Three lanes plus exclusive right- and left-turn lanes	Exclusive left-turn phase plus permissive left turns during through movement

Note: ADT = average daily traffic, and *denotes street on which the observed approach was located.

[a]ADT is for the street on which the observed approach was located.
[b]Denotes design and duration of change interval. For example, 5Y + 3AR indicates 5 s of yellow plus 3 s of all-red time.

Table 2. Study site speed characteristics.

Intersection Approach	Posted Speed Limit (mph)	Approach Speed: Last Vehicle Through the Intersection: Sample Size	Mean Speed (mph)	Standard Deviation	85th Percentile (mph)	First Vehicle to Stop: Sample Size	Mean Speed (mph)	Standard Deviation	85th Percentile (mph)
University Drive	35	16	35.46	7.49	45.8	64	33.69	8.81	43.4
Southern Avenue									
Day	45	69	36.31	5.25	42.4	70	34.97	7.08	40.9
Night	45	49	40.58	6.30	47.7	64	35.77	7.17	41.8
US-60	50	87	39.66	7.52	48.0	107	35.88	7.90	44.8
First Avenue	45	152	39.84	6.42	46.5	178	39.04	7.27	46.9
Sixth Street	30	67	35.24	5.50	40.5	97	31.63	6.05	37.7
Broadway Boulevard									
Day	45	156	41.16	5.47	47.0	143	37.41	6.47	43.5
Night	45	96	37.27	5.54	44.0	116	36.32	5.43	42.3
All approaches		693	38.91	6.38	45.8	839	36.13	7.30	43.5

Table 3. Distance from intersection at beginning of yellow interval.

Intersection Approach	Last Vehicle Through Intersection: Mean Distance (ft)	Standard Deviation	First Vehicle to Stop: Mean Distance (ft)	Standard Deviation
University Drive	118.9	64.0	268.7	74.2
Southern Avenue				
Day	142.9	60.2	262.6	60.1
Night	152.6	72.8	272.0	52.2
US-60	149.6	62.2	262.5	54.4
First Avenue	136.3	46.9	238.4	48.0
Sixth Street	102.7	33.7	202.5	51.0
Broadway Boulevard				
Day	146.5	49.3	250.5	45.9
Night	114.1	46.4	240.9	49.0
All approaches	135.4	54.5	246.6	56.1

established on each of the study approaches, a grid was developed so that the location of a vehicle could be determined when the film was projected on a screen. The grid indicated the distance from the intersection. In all cases, this distance was measured from the crosswalk.

For each of the vehicles that were the first to stop after the beginning of the yellow interval, the following information was extracted from the film record:

1. Distance from the intersection at the beginning of the yellow interval,
2. Location of the vehicle when the brakes were applied (as indicated by the brake lights),
3. Location of the vehicle when it stopped,
4. Time required for the vehicle to stop,
5. Perception-reaction time (determined as the time between the beginning of the yellow interval and the application of the brakes), and
6. Type of vehicle if other than a passenger car or light truck.

Based on this information, the approach speed and the average deceleration rate was computed for each stopping vehicle.

In addition, the behavior of the last vehicle to pass through the intersection after the beginning of the yellow was determined by making the following observations:

1. Location of the vehicle at the beginning of the yellow interval,
2. Time elapsed from the onset of the yellow interval until the vehicle entered the intersection,
3. Type of vehicle if other than a passenger car or light truck, and
4. If the vehicle entered the intersection on the red signal indication.

For the vehicles that did not stop, the approach speed was also computed. In this case, the determination of speed was accomplished by using the elapsed time and the distance traveled.

RESULTS

Table 2 indicates the observed speed characteristics at the study sites. At all six study locations, the mean approach speeds of the last vehicle through the intersection were higher than the first vehicle to stop. It should be recognized that the measurement of the speed of the first vehicle was made at a point where some deceleration may have occurred. Also, speed measurements for the through vehicles were made before the onset of the yellow interval, and some of those vehicles may have been accelerating. It was not possible, however, to determine if these vehicles were accelerating from the films that were made.

Although the posted speed limits ranged from 30 to 50 mph, there was much less variation in the vehicle approach speeds. The difference between the high and low mean speed was less than 6 mph for the last vehicle through the intersection and less than 8 mph for the first vehicle to stop. This indicates that there was not that much diversity in the vehicle operations when comparing the sites.

With respect to the comparison of the intersection approaches where the day and night studies were conducted, there was no significant difference in the approach speeds for the first vehicle to stop. The approach speeds were significantly higher at night for the through vehicles in the case of the Southern Avenue site and were significantly lower at night at the Broadway Boulevard site. It might be expected that the nighttime approach speeds would consistently be higher because of the lower volumes.

Table 3 summarizes the mean distance from the intersection at the beginning of the yellow interval for the two groups of vehicles at each site. As would be expected, the distance from the intersection was considerably less for the last vehicle through the intersection. In fact, the mean distance differences ranged from approximately 100 to 150 ft for the study approaches. Figure 1 depicts the cumulative frequency distributions of the distance from the intersection for all sites.

A summary of the observed perception-reaction

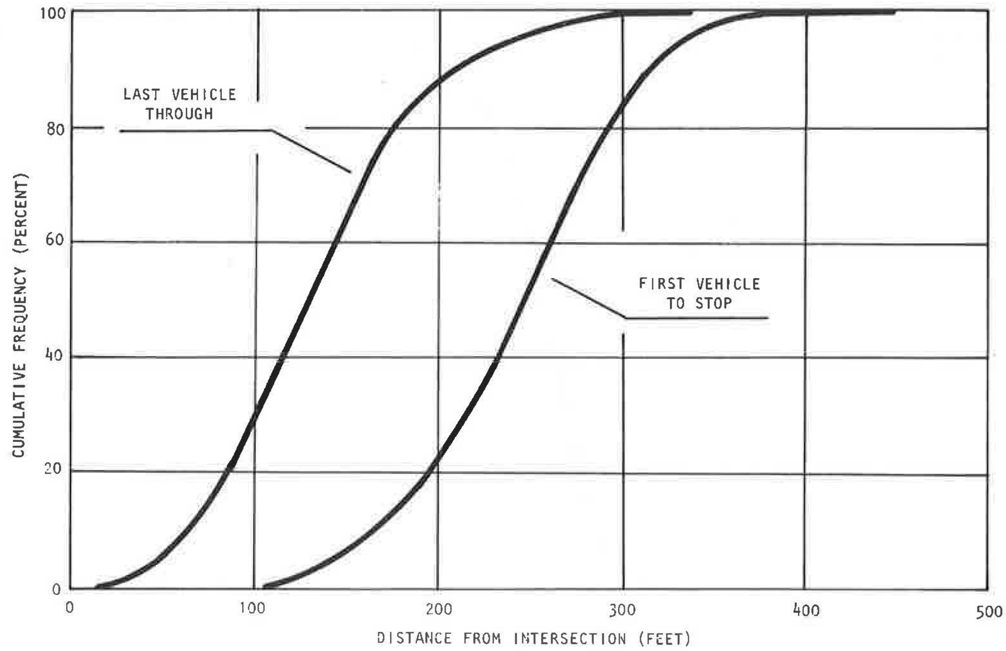

Figure 1. Cumulative frequency distribution of vehicle location at onset of yellow interval.

times is given in Table 4. As indicated previously, the perception-reaction time was determined by measuring the time between the onset of the yellow interval and the application of the brakes. These times are for the stopping vehicles only. Generally, the study team was unable to determine the perception-reaction time for the drivers who chose to proceed through the intersection. There were a few cases during the night studies where it was possible to detect that drivers began to apply the brakes but then proceeded through the intersection. In these cases, a brief flickering of the brakelights was recorded on the film. The cumulative frequency distribution curve for all approaches is shown in Figure 2.

It was theorized that the perception-reaction time is related to such factors as the location of the vehicle at the beginning of the yellow interval, the approach speed, or even possibly the deceleration rate that was used by the driver. These hypotheses were tested by using correlation and regression analyses and computing the coefficient of determination (r^2). When these variables were analyzed in terms of the relation with the length of the perception-reaction time, the following r^2-values were obtained:

Variable	r^2
Distance from intersection	0.08
Approach speed	0.09
Deceleration rate	0.01

These results would serve to indicate that there was little relation between the perception-reaction time and these variables.

Table 5 summarizes the deceleration rates that were observed at the study locations, and the cumulative frequency distribution of the deceleration rates is shown in Figure 3. It should be emphasized that these observed values reflect driver behavior under the set of conditions experienced at the study sites. Further analysis of the variation in the deceleration rates with respect to the study sites is discussed later.

The tabulation of the vehicles entering the intersection on the red signal indication is given in

Table 4. Perception-reaction times.

Intersection Approach	Mean Time (s)	Standard Deviation	85th Percentile Time (s)
University Drive	1.28	0.82	2.0
Southern Avenue			
Day	1.49	0.62	1.9
Night	1.43	0.73	2.0
US-60	1.38	0.60	2.1
First Avenue	1.24	0.51	1.8
Sixth Street	1.55	0.70	2.0
Broadway Boulevard			
Day	1.16	0.48	1.5
Night	1.09	0.44	1.5
All approaches	1.30	0.60	1.8

Table 6. Generally, the intersections with the shorter yellow interval and the all-red phase resulted in a higher percentage of vehicles entering on the red indication. This would be expected due to the shorter yellow duration. The First Avenue site had an extremely high percentage of vehicles entering on the red signal compared with the other locations. Although this intersection is situated in a more outlying area than the other two study locations in Tucson, there is no clear explanation for this percentage being so much higher than the other sites.

When comparing the variation between the day and night conditions, the percentage of entering vehicles was drastically reduced during the night conditions. This may be partly explained by the fact that there were less vehicle queues during the nighttime period. The deceleration rate at the Broadway Boulevard site also decreased during the nighttime period.

COMPARATIVE ANALYSIS OF STUDY SITES

The second major portion of the analysis effort dealt with the comparison of observed behavior at the different study sites. Basically, these comparisons were intended to test the influence of the driving environment and signal timing practices.

The mean and standard deviation for each of the

Figure 2. Cumulative frequency distribution of observed perception-reaction times.

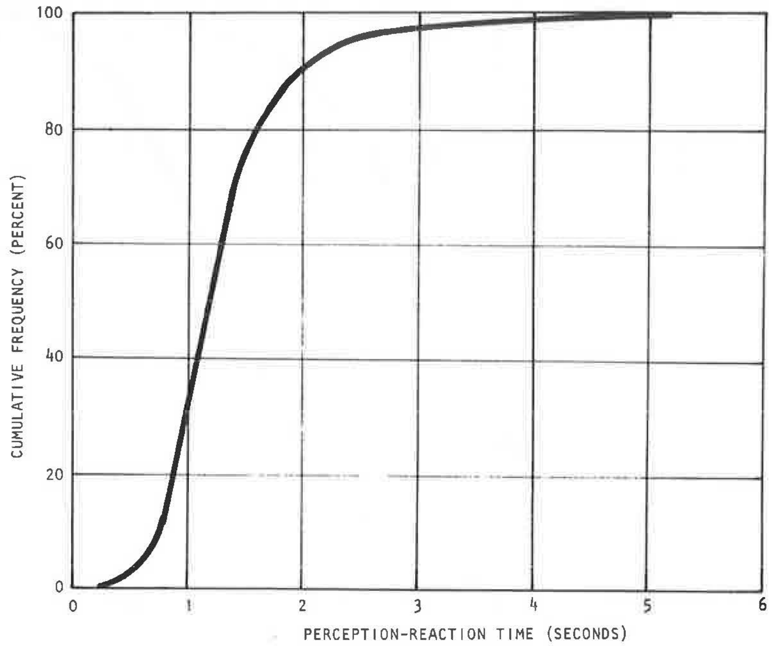

Table 5. Observed deceleration rate.

Intersection Approach	Change Interval[a]	Mean Approach Speed (mph)	Mean Rate (ft/s/s)	Standard Deviation	85th Percentile Rate (ft/s/s)
University Drive	5Y	33.69	7.00	3.80	11.5
Southern Avenue					
Day	5Y	34.97	10.73	3.02	13.9
Night	5Y	35.77	11.60	2.57	14.8
US-60	5Y + 3AR	35.88	11.79	3.43	15.8
First Avenue	3Y + 2AR	39.04	12.45	3.52	16.1
Sixth Street	3Y + 2AR	31.63	13.87	4.54	18.2
Broadway Boulevard					
Day	3.6Y + 2AR	37.41	12.83	4.12	17.2
Night	3.6Y + 2AR	36.32	9.67	3.06	12.5
All approaches		36.13	11.58		

[a] Denotes design and duration of change interval. For example, 5Y + 3AR indicates 5 s of yellow plus 3 s of all-red time.

Figure 3. Cumulative frequency distribution of observed deceleration rates.

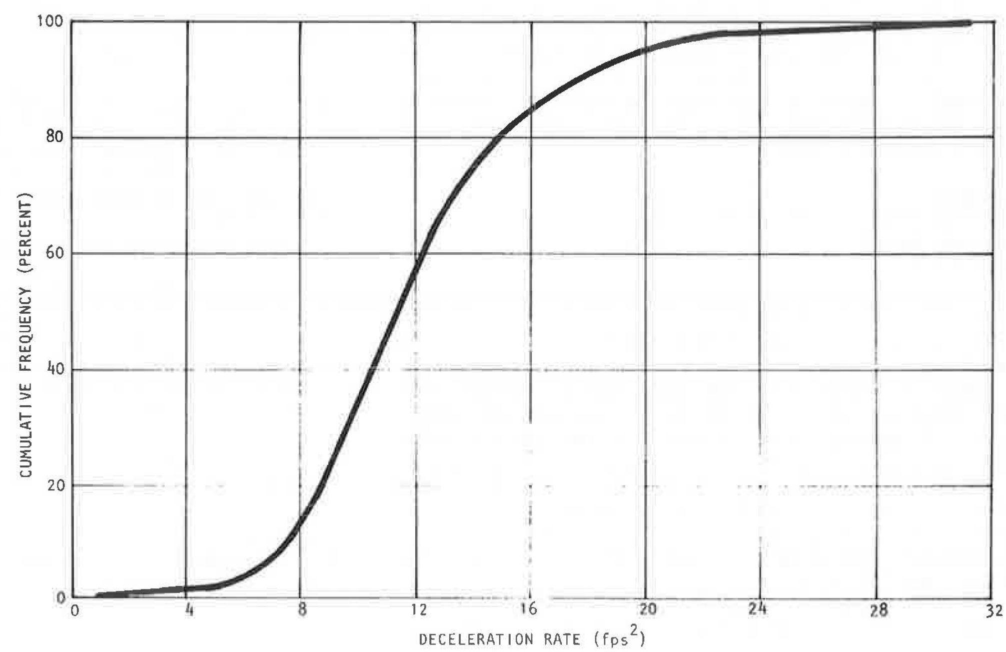

Table 6. Percentage of last vehicles through intersection on red indication.

Intersection Approach	Change Interval[a]	Sample Size of Last Vehicle Through Intersection	Percentage Entering on Red Indication
University Drive	5Y	16	0
Southern Avenue			
Day	5Y	69	2.9
Night	5Y	49	0
US-60	5Y + 3AR	87	2.3
First Avenue	3Y + 2AR	152	29.6
Sixth Street	3Y + 2AR	67	8.9
Broadway Boulevard			
Day	3.6Y + 2AR	156	8.3
Night	3.6Y + 2AR	96	1.0

[a]Denotes design and duration of change interval. For example, 5Y + 3AR indicates 5 s of yellow plus 3 s of all-red time.

Table 7. Comparison of Phoenix area sites (day observations).

Item	Sites Compared		
	US-60 and University Drive	US-60 and Southern Avenue	Southern Avenue and University Drive
First vehicle to stop			
Approach speed	N	N	N
Perception-reaction time	N	N	N
Distance from intersection	N	N	N
Deceleration rate	S	N	S
Last vehicle through intersection			
Approach speed	S	S	N
Distance from intersection	N	N	N

Note: N = difference is not significant, and S = significant difference.

Table 8. Comparison of Tucson area sites (day observations).

Item	Sites Compared		
	First Avenue and Broadway Boulevard	First Avenue and Sixth Street	Sixth Street and Broadway Boulevard
First vehicle to stop			
Approach speed	S	S	S
Perception-reaction time	N	S	S
Distance from intersection	S	S	S
Deceleration rate	N	S	N
Last vehicle through intersection			
Approach speed	N	S	S
Distance from intersection	N	S	S

Note: N = difference is not significant, and S = significant difference.

Table 9. Comparison of day and night behavior.

Item	Site	
	Southern Avenue	Broadway Boulevard
First vehicle to stop		
Approach speed	N	N
Perception-reaction time	N	N
Distance from intersection	N	N
Deceleration rate	N	S
Last vehicle through intersection		
Approach speed	S	N
Distance from intersection	N	N

Note: N = difference is not significant, and S = significant difference.

measured parameters were computed. Where data from intersections were combined for a particular analysis, these values were computed for the combined data. Potential differences in behavior were analyzed by examining differences in the means for specific groups or pairs. In all cases, the 95 percent confidence level was used for the purpose of assessing statistical significance.

Table 7 gives a summary of the results for a comparison of the sites in the Phoenix metropolitan area. For this analysis, a particular site was compared with each of the other sites in that metropolitan area. Note that there was a statistically significant difference in the deceleration rate at the University Drive site even though there was no significant difference in the other parameters for the first vehicle to stop. For the last vehicle through the intersection after the beginning of the yellow interval, there were two cases where the differences in the approach speeds were significant.

A similar comparison of the sites in Tucson yielded a somewhat different set of results (Table 8). For this group of intersections, the comparison revealed significant differences in a number of measures of behavior.

Table 9 gives the results of the comparison of the day and night studies at the two sites selected for that purpose. In this case, the observed day and night behavior at each of the two sites were compared. Again, there were some differences between the two study sites. As was previously indicated, the observed deceleration rates were significantly lower at night for the Broadway Boulevard location. The Southern Avenue site revealed no difference in the day and night comparison, except that the mean approach speed for the last vehicle through the intersection was significantly higher at night.

All three of the study locations in Tucson had change intervals with the all-red phase; thus, an analysis of the influence of the yellow only versus the yellow plus all-red intervals required a comparison of the Phoenix and Tucson area sites. For this analysis, the Southern Avenue and the University Drive study approaches in the Phoenix area were used. The data for these intersections were compared with the information for the three Tucson intersections. The results of this analysis are given in the table below (note, N = difference is not significant, and S = significant difference):

Item	Difference
First vehicle to stop	
Approach speed	S
Perception-reaction time	N
Distance from intersection	S
Deceleration rate	S
Last vehicle through intersection	
Approach speed	S
Distance from intersection	N

The mean approach speeds for both the stopping and through vehicles were significantly higher in Tucson. In contrast, however, the distance from the intersection at the beginning of the yellow interval was significantly less and the deceleration rate was higher in Tucson.

CONCLUSIONS

The results of this study reflect the behavior of drivers given the existing intersections and traffic signal timing. It is not possible to anticipate what the resulting driver behavior would be with a different set of conditions. Based on the observations and analyses that were made a part of this study, the following conclusions can be drawn:

1. The observed mean deceleration rates chosen by drivers at the various study sites ranged from 7.0 to 13.9 ft/s/s. The mean deceleration rate for all observations was 11.6 ft/s/s.
2. The day and night comparison of driver behavior revealed mixed results in that the mean deceleration rate was not significantly different during the nighttime period at one site; however, there was a decrease in the mean rate from 12.9 to 9.7 ft/s/s at the Broadway Boulevard location.
3. With respect to the perception-reaction times of drivers, the observed mean time for each of the sites ranged from 1.16 to 1.55 s. The mean for all approaches was 1.30 s. The 85th percentile value for the time measurement ranged from 1.5 to 2.1 s.

Although the nighttime studies revealed mean times that were slightly less at both sites, these differences were not statistically significant. Factors such as approach speed, distance from the intersection at the beginning of the yellow interval, and the deceleration rate that is used by the driver had little or no influence on the perception-reaction time.

4. The comparison of study sites in each of the metropolitan areas also provided mixed results. In the Phoenix area, the mean deceleration rate at the University Drive location was significantly less than at the other two sites even though most of the other measures showed no significant differences. Similar comparisons for the Tucson sites revealed differences in a number of the measured parameters.
5. In terms of the comparison of the yellow only versus the yellow plus all-red change intervals, there were no significant differences in the perception-reaction times. This was true when the intersection with an all-red interval in the Phoenix area was compared with the other sites that had a yellow only interval. It was also true when the intersections in Tucson were compared with those in Phoenix.

The analysis of the deceleration rates for the two types of change intervals did not yield consistent results. Although the deceleration rates for the Tucson sites were higher than those observed in the Phoenix area, there was a variation in the significance of the differences between intersections with the same type of change interval.

In reviewing the comparisons of the observed behavior, the use of some degree of caution should be exercised. For example, small differences in deceleration rates may be statistically significant but may have no appreciable affect on the evaluation and solution of traffic problems.

In addition, the project staff spent considerable time observing the operations of these intersections during the course of the study. Although the observed deceleration rates and the perception-reaction times varied from the values recommended by current practice, it should be recognized that the intersections appeared to be operating reasonably well with respect to the change interval.

ACKNOWLEDGMENT

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The contents of this paper reflect our views, and we are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Arizona Department of Transportation or FHWA. This paper does not constitute a standard, specification, or regulation. Trade or manufacturers' names that may appear herein are cited only because they are considered essential to the objectives of the paper. The U.S. government and the State of Arizona do not endorse products or manufacturers.

Discussion

Peter S. Parsonson

The most controversial issue in the preparation of the second edition of the Institute of Transportation Engineers (ITE) Transportation and Traffic Engineering Handbook (9) was the calculation of the yellow and all-red intervals. The final draft of the handbook, published in 1982, called for the use of a perception-reaction time of 1 s. The paper by Wortman and Matthias does not include within its scope an evaluation of this current practice; but it is important that the discussions of their paper include a comparison of their findings with the ITE handbook recommendations.

Wortman and Matthias observed more than 800 stopping drivers and found an average perception-reaction time of 1.3 s. ITE uses 1.0, so we need to consider whether the ITE procedure gives yellow times that are 0.3 s too short.

There are two reasons why this paper does not represent a challenge to the currently accepted value of 1.0 s for perception-reaction time. First, the perception-reaction times reported by Wortman and Matthias are for the first vehicle to stop after the yellow begins. For all we know, many of these vehicles were so far from the intersection when the yellow came on that it was out of the question to continue through the intersection. These drivers could react at their leisure and brake comfortably to a stop. They tell us nothing about how fast they would react when there is a real decision to be made.

Second, it is necessary to consider perception-reaction time and deceleration rate jointly, as these two variables are tied together as determinants of stopping distance and required yellow time. The Wortman and Matthias data for perception-reaction time and deceleration rate, taken together, produce calculated yellow times comparable with those derived from the ITE guidelines.

For example, consider an approach with a speed of 50 mph. The ITE values of 1 s for perception-reaction time and 10 ft/s/s for deceleration rate yield a yellow time of 4.7 s. The calculation, when using the Wortman and Matthias values of 1.3 s and 11.6 ft/s/s, gives 4.5 s, which is just a couple of tenths of a second less than the ITE result. That difference makes sense because ITE assumes a wet road, while the Wortman and Matthias data came from dry roads. The average drivers in the Wortman and Matthias study may be allowing themselves a little extra time to react because they know they can easily make up for it by braking a little more heavily on a dry road. At 35 mph, the two sets of values give an identical 3.5 s.

It is well known (10) that the minimum yellow time can be computed by dividing the stopping distance by the approach speed, as follows:

$$Y = \text{stopping distance/speed} = [\nu t + (\nu^2/2a)]/\nu = t + (\nu/2a) \quad (1)$$

If stopping distance is measured directly by observing the behavior of traffic at the onset of the yellow interval, the required yellow time can be calculated without the need to make assumptions for t and a. From 50 mph, for example, Zegeer (11) found that 90 percent of the drivers will decide to stop if they are 350 ft from the intersection when the yellow begins. A yellow time long enough for 90 percent of the drivers is as follows:

$$Y = \text{stopping distance/speed} = 350/(50 \times 1.47) = 4.76 \text{ s} \quad (2)$$

This minimum yellow is entirely empirical and is independent of assumptions for t and a. It can be used to calculate combinations of t and a that will satisfy it, as follows:

$$4.76 = t + [(50 \times 1.47)/2a] = t + (36.75/a) \quad (3)$$

A table of combinations that satisfy this relation is given below:

t	a
0.75	9.2
1.0	9.8
1.1	10.0
1.3	10.6
1.5	11.3
2.5	16.3

This table shows that provision of a yellow of 4.76 s allows the driver stopping from 350 ft a wide range of responses. A quick-reacting driver can decelerate comfortably to a stop, while one slower to react must compensate by braking more forcefully. For example, a driver who reacts in 1-s flat can decelerate comfortably at 9.8 ft/s/s, while the driver who requires 1.3 s must brake more heavily at 10.6 ft/s/s.

The point is that it is the combination of t and a that counts. The combination reported by Wortman and Matthias is in harmony with the combination suggested by ITE. The small difference probably can be accounted for as dry road versus wet road.

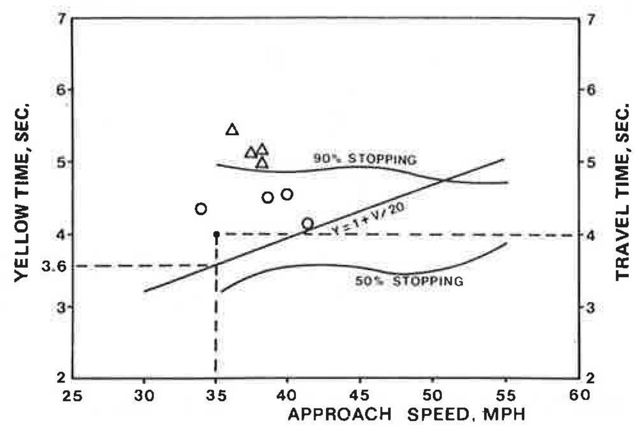
Figure 4 shows Zegeer's observed distances for 50 and 90 percent stopping, converted to travel time to the intersection by dividing by the approach speed. The data are for 2100 drivers responding to the yellow interval in Kentucky. The upper curve means that 90 percent of these drivers stopped if the yellow came on when they were about 4.8 s of travel time from the intersection. The curve is quite flat; values of just under 5 s were observed over the entire range of speeds studied. At 50 mph, 4.8 s is a yellow time long enough for 90 percent of the Kentucky drivers. These motorists seem to be indicating that they want a constant yellow of almost 5 s, regardless of the approach speed.

Figure 4 also shows a curve for yellow time calculated by using the ITE values of t = 1 and a = 10, as follows:

$$Y = t + (\nu/2a) = 1 + [\nu/(2 \times 10)] = 1 + (\nu/20) \quad (4)$$

Consider a driver approaching at 35 mph. Suppose he or she is shown a yellow calculated by using t = 1 and a = 10, which yields 3.6 s. Now suppose the vehicle is 4 s from the intersection when the yellow comes on. There is a 70 percent chance that the driver will decide to stop (obtained by interpolating between Zegeer's 50 and 90 percent curves). There is a 30 percent chance that he or she will

Figure 4. Certain relation involving yellow time, travel time, and approach speed.



decide to clear, in which case the driver will then run the red because of the deficient yellow. The driver seems to want 4.8 s--he or she needs 4.0 s--but the driver gets only 3.6 s, so he or she may well run the red. The Zegeer data suggest that yellows calculated by the ITE guidelines, or by the Wortman and Matthias data, are too short for speeds less than 50 mph if we are trying to meet the needs of the 90th percentile driver.

The eight plotted points on Figure 4 cannot be explained without further discussion of entering on the red because of yellow time deficiency. The First Avenue site was observed to have an extremely high percentage of vehicles entering on the red signal compared with other locations. The reason for this seems clearly to be that this site was much more deficient in yellow time than any of the others. Table 10 shows the deviations of the observed yellow times from those calculated by using t = 1 s and a = 10 ft/s/s for the eight sites. Figure 5 shows the percentage entering on the red plotted against these deviations expressed as a surplus or deficiency in yellow time. The figure shows that the First Avenue site can be understood if the percentage entering on the red increases exponentially (or logarithmically) with increasing deficiency of yellow. The figure also suggests that, at zero deficiency (obtained by using t = 1 and a = 10 for our calculations), we can expect about 3 percent of the vehicles to enter on the red. This might be considered an acceptable level, inasmuch as it is a common rule of thumb in the positive-guidance area that an operational deficiency exists if traffic conflicts or erratic maneuvers are exhibited by more than 3 percent of the approaching vehicles.

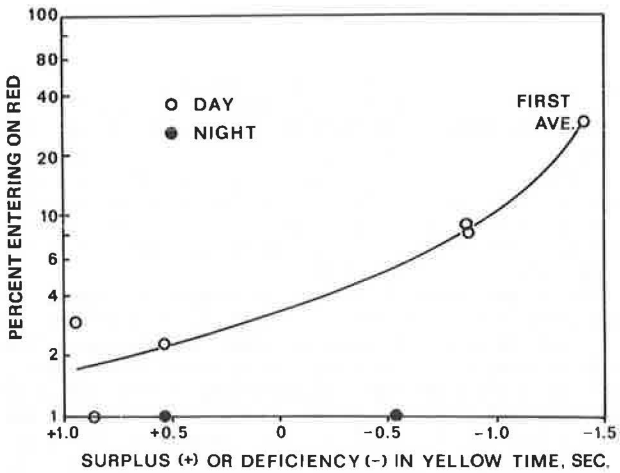
The final comment is an explanation of the eight plotted points on Figure 4. These points represent the first vehicles to stop at the eight sites reported by Wortman and Matthias. Specifically, the ordinate of each point is the mean travel time to the intersection for the first vehicles to stop. The ordinates are not the yellow times at these intersections. The points plotted as triangles are the four sites shown in Table 10 to have a surplus of yellow time, and the circles are four locations deficient in yellow time. All of these drivers decided to stop because they correctly judged that their travel times to the intersection were too great to allow entry on the yellow. The average ordinate of these points, which represents 100 percent stopping, lies close to Zegeer's 90 percent curve and indicates good agreement. It is most interesting to note that the sites deficient in yel-

Table 10. Deviations of yellow time from calculated standard.

Intersection Approach	Percentage Entering on Red	85th Percentile Approach Speed[a] (mph)	Calculated Standard Yellow Time[b] (s)	Actual Yellow Time (s)	Surplus(+) or Deficiency(-)[c] (s)
University Drive	0	44.6	4.2	5	+0.8
Southern Drive					
Day	2.9	41.6	4.1	5	+0.9
Night	0	44.7	4.4	5	+0.6
US-60	2.3	46.4	4.5	5	+0.5
First Avenue	29.6	46.7	4.4	3	-1.4
Sixth Street	8.9	39.1	3.8	3	-0.8
Broadway Boulevard					
Day	8.3	45.3	4.4	3.6	-0.8
Night	1.0	43.1	4.2	3.6	-0.6

[a] Average of last vehicle through the intersection and first vehicle to stop.
[b] Derived by using the equation $1 + (\nu/2a \pm 64.4g)$, where $a = 10$ ft/s/s and g = approach gradient.
[c] Derived by subtracting actual yellow time from calculated standard yellow time.

Figure 5. Percent entering on red versus yellow-time surplus or deficiency.



low all plotted lower than those with a surplus. That is, drivers approaching yellow-deficient signals were willing to stop from locations closer to the intersection, which required quicker reaction and/or heavier braking. The data suggest that the drivers are aware of the lengths of the yellow times at these intersections and do factor this into their decision whether to stop. This finding conflicts with the 1979 report by Stimpson (12), which indicated that the length of the yellow interval does not affect driver behavior.

Discussion

Jack A. Butler

The work of Wortman and Matthias has produced data that confirms the findings of other researchers and reinforces current trends in calculating vehicle change intervals. The conclusions of the authors, however, do not fully explain the data and fail to answer questions, stated and implied, in their report. This discussion will expand their analysis and propose further explanations of the data.

The first goal of the subject study was the determination of actual deceleration rates applied in stopping a vehicle at the monitored signal-controlled intersections. By employing field study methods to survey unsuspecting drivers, the possibility of driver behavior modification was removed.

Given that it has been shown (13) that acceptable deceleration rates fall within the range of 8-12 ft/s/s (15th to 85th percentiles), the observed values generally fall within the upper half of the acceptable range. The two deviating means can be explained by the approach geometry.

Although not included in the report, the University Drive site includes an at-grade railroad crossing within the stopping distance. Wortman disclosed in a conversation that they had observed drivers applying a higher deceleration rate on the upstream side of the crossing. The reported mean of 7.0 ft/s/s is an average that may not accurately indicate a rate used at a signalized intersection.

The upper limit on the range of observed rates of 13.9 ft/s/s at Sixth Street includes the effect of gravity on slowing the vehicle. Parsonson and Santiago (14) published a report that details an easy method to evaluate the effect of roadway slope on vehicle deceleration. Removing the deceleration due to gravity on the 2 percent uphill grade produces a rate of 12.6 ft/s/s. It is this value that should be used in discussing driver behavior, as it is the force perceived by the motorist.

All observations were made on dry pavement. Because it might be expected that wet pavement conditions would lower the study values for deceleration, Wortman and Matthias' findings tend to reinforce the use of 10 ft/s/s as a mean deceleration rate in yellow interval calculations.

The second goal of the study was the collection of nighttime data and the evaluation of various intersection environments. The data do not indicate any differences among these factors except as noted above.

The third goal, evaluation of all-red intervals, was not fully accomplished. The measure of effectiveness selected--vehicles entering on red--is indicative of yellow interval performance. It has been shown (5) that the length of the yellow interval does not effect driver behavior. Indeed, it is the yellow interval that must conform to driver behavior. A too-short yellow will naturally produce a large number of red-aspect violations. The subject study provides some confirmation of that principle. Data were not presented that allow the comparison of the all-red timing and intersection geometry. Further work is needed to accomplish this third goal.

It is possible to calculate more reasonable yellow intervals by applying the following formula (14):

$$y = t + [\nu/(2a + 64.4g)] \quad (5)$$

where

t = reaction time (s),
ν = 85th percentile approach speed (ft/s),
a = deceleration rate (ft/s/s), and

g = grade as percent divided by 100.

When this is done, a clear relation emerges between yellow time deficiency and red-aspect violations.

Table 11 lists the values relevant to this discussion; Tables 12 and 13 illustrate yellow and all-red times over a variety of speeds and intersection widths. The 85th percentile speeds were estimated by adding one standard deviation to the 50th percentile speed. The means for the last car through and the first to stop were combined, as were the standard deviations. This technique attempts to balance the effect of acceleration of vehicles that did not stop with the deceleration that occurs dur-

Table 11. Reported and derived data.

Item	University Drive	Southern Avenue		US-60	First Avenue	Sixth Street	Broadway Boulevard	
		Day	Night				Day	Night
Yellow interval (s)	5	5	5	5	3	3	3.6	3.6
All-red interval (s)	0	0	0	3	2	2	2	2
85th percentile speed (ft/s)	63	61	66	63	65	55	64	60
Calculated yellow (s)	4.2	4.1	4.3	4.2	4.3	3.6	4.2	4.0
Actual minus calculated yellow (s)	0.8	0.9	0.7	0.8	-1.3	-0.6	-0.6	-0.4
Last through vehicles entering on red (%)	0	3	0	2	30	9	8	1
Mean stopping distance (ft)	269	263	272	263	238	203	251	241
Stopping distance implied by calculated yellow (ft)	265	250	271	265	280	198	269	240
Distance traveled at 85th percentile speed during actual yellow (ft)	315	305	330	315	195	165	230	216
Mean reaction time (s)	1.3	1.5	1.4	1.4	1.2	1.6	1.2	1.1
Mean rate of deceleration (ft/s/s)	7.0	10.8	11.6	11.8	12.4	12.6[a]	12.9	9.7
Deceleration rate implied by current yellow for 85th percentile speed (ft/s/s)	7.9	7.6	8.3	7.9	16.3	12.5[a]	12.3	11.5

[a] Value has been adjusted to remove effect of approach grade.

Table 12. Time (in seconds) of yellow interval.

85th Percentile Speed	Yellow Intervals by Grade of Approach								
	Uphill (%)					Downhill (%)			
	+4	+3	+2	+1	Level	-1	-2	-3	-4
25	2.63	2.68	2.73	2.78	2.84	2.90	2.96	3.03	3.11
30	2.95	3.01	3.07	3.14	3.21	3.28	3.36	3.44	3.53
35	3.28	3.35	3.42	3.49	3.57	3.66	3.75	3.85	3.95
40	3.60	3.68	3.76	3.85	3.94	4.04	4.14	4.25	4.37
45	3.93	4.02	4.11	4.20	4.31	4.42	4.54	4.66	4.80
50	4.26	4.35	4.45	4.56	4.68	4.80	4.93	5.07	5.22
55	4.58	4.69	4.80	4.92	5.04	5.18	5.32	5.47	5.64
60	4.91	5.02	5.14	5.27	5.41	5.56	5.71	5.88	6.06
65	5.23	5.36	5.49	5.63	5.78	5.94	6.11	6.29	6.48

Note: Yellow interval table values derived from the formula:

$$y = t + [\nu/(2a + 64.4g)]$$

where

y = yellow interval (s),
t = reaction time (set at 1.0 s),
ν = 85th percentile approach speed (ft/s),
a = deceleration rate (set at 10 feet/s/s), and
g = grade of approach over the braking distance (percent divided by 100).

Table 13. Time (in seconds) of all-red interval.

85th Percentile Speed	All-Red Intervals by Width of Approach (ft)										
	20	30	40	50	60	70	80	90	100	110	120
25	1.09	1.36	1.63	1.90	2.18	2.45	2.72	2.99	3.27	3.54	3.81
30	0.91	1.13	1.36	1.59	1.81	2.04	2.27	2.49	2.72	2.95	3.17
35	0.78	0.97	1.17	1.36	1.55	1.75	1.94	2.14	2.33	2.53	2.72
40	0.68	0.85	1.02	1.19	1.36	1.53	1.70	1.87	2.04	2.21	2.38
45	0.60	0.76	0.91	1.06	1.21	1.36	1.51	1.66	1.81	1.97	2.12
50	0.54	0.68	0.82	0.95	1.09	1.22	1.36	1.50	1.63	1.77	1.90
55	0.49	0.62	0.74	0.87	0.99	1.11	1.24	1.36	1.48	1.61	1.73
60	0.45	0.57	0.68	0.79	0.91	1.02	1.13	1.25	1.36	1.47	1.59
65	0.42	0.52	0.63	0.73	0.84	0.94	1.05	1.15	1.26	1.36	1.47

Note: All-red interval table values derived from the formula:

$$r = (w + l)/\nu$$

where

r = all-red interval (s),
w = width of intersection (ft),
l = length of vehicle (set at 20 ft), and
ν = speed of vehicle (ft/s).

ing the driver reaction time. The values for t and a are 1.0 s and 10 ft/s/s, respectively.

One also finds a close agreement between the predicted stopping distance and that that was actually observed. The longer reaction times were offset by higher deceleration rates. The only exception is First Avenue, where the observed mean is 15 percent less than expected. This is also the approach with the highest red-aspect violation rate (30 percent) and the greatest yellow time deficiency. It may be concluded that many drivers chose not to apply the high rate of deceleration necessary on this approach to keep from entering on red.

A similar, but less severe, yellow time deficiency and red-aspect violation relation exists at the Sixth Street and Broadway Boulevard sites. The lower violation rate observed at night for Broadway Boulevard is directly related to the lower approach speed at night.

The yellow deficiency can be further refined by determining the deceleration rates implied by the existing signal timing at the 85th percentile approach speed. These values all fall within the acceptable range except for those locations that experience the highest red-aspect violation rates. At the First Avenue site, a deceleration rate of 16.3 ft/s/s would be necessary given a 1.0-s reaction time. The higher-speed drivers have obviously indicated their dislike of such a high rate by entering on red.

Taken together, the data show that drivers reject deceleration rates significantly greater than 12 ft/s/s and that there is an evident relation between the length of the yellow and red-aspect violation. Implied in this is the observed absence of drivers who adjust their behavior significantly to conform to yellow times.

It may be further stated that driver action was not affected by the presence of an all-red interval, which gives support to the findings of Benioff and others (15) that an increase in red-aspect violation does not accompany the use of such intervals. Beyond that, it is not possible to examine the given all-red times, since the critical factor of intersection width was omitted. Absent as well was a discussion of right-angle and other accidents that would provide insight on the effectiveness of all-red intervals.

The final goal of the subject study was the recording of driver reaction times. The authors admit in the paper that they had no way of determining the reaction time of drivers who chose not to stop. It may be further suspected that they had no way of determining the true reaction time; i.e., did the drivers react in a leisurely manner.

In spite of those shortcomings, the data seem to refute the contention of the American Association of State Highway Officials (AASHO) (16) that the 95th percentile alert reaction time is 1.0 s. The results also place an upper limit on the range of driver reaction times.

As Parsonson shows in his discussion, for any given speed there is a range of reaction times and deceleration rates that produce the same stopping distance. Because the various sites had similar approach speeds, it is not possible to select the correct combination. Further research is needed to resolve the issue, including the question of the applicability of observed values to design calculations.

Committee 4A-16 (Use and Timing of Vehicle Change Intervals) of ITE will soon begin field experiments to evaluate the effectiveness of a range of reaction times and deceleration rates in calculating the yellow interval length.

The work of Wortman, Matthias, and their team members comes at a time when a decision must be made on selecting a realistic change interval calculation methodology. In that context, their report provides considerable guidance.

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Driver Perception-Reaction Time: Are Revisions to Current Specification Values in Order?

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The appropriateness of current specification values for the driver characteristic perception-reaction time is examined for several geometric design and traffic operations standards: stopping-sight distance, lateral clearance to sight obstructions on horizontal curves, intersection sight distance, and vehicle change interval. The analysis focuses on three issues. First, a brief review of state-of-the-art knowledge relative to the driver characteristic is presented. The review compares field results with aggregated simulation tests of the discrete components of perception-reaction time. The second issue that is addressed is the sensitivity of the design or operations standard to incremental changes in perception-reaction time. The third issue is a determination of the actual maximum allowable perception-reaction time for the various standards. The findings of the research indicate that the specification values for perception-reaction time are too low for the stopping-sight-distance design standards and the vehicle-clearance-interval standard. Also recommended is that the perception-reaction action for case III intersection sight distance be redefined.

The driver characteristic perception-reaction time is considered a factor in a variety of highway design and operations standards. This paper examines the current specification value for the characteristic for several standards: stopping-sight distance, lateral clearance to sight observation on horizontal curves, intersection sight distance, and vehicle clearance interval.

STOPPING-SIGHT DISTANCE

Driver Characteristic

The current American Association of State Highway and Transportation Officials (AASHTO) standard for stopping-sight distance is in part based on a driver characteristic of brake reaction (P). More precisely, it should be identified as the perception-brake-reaction time. The American Association of State Highway Officials (AASHO) (1) 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 s. As specified in the AASHO Policy on Geometric Design of Rural Highways (1), this value was determined from an assumed perception time of 1.5 s and a brake-reaction time of 1.0 s. 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." The values were based on the results of an assortment of laboratory and field-controlled studies that used alerted drivers (2-4).

The 2.5-s time was again selected as the specification in review drafts of the updated AASHTO manual, A Policy on Geometric Design of Highways and Streets (5). [In this draft version, there is an inconsistency in the terminology and definition of this characteristic. Only brake-reaction time is defined (interval between the driver's recognition of an object and driver's application of brakes), but yet in arriving at the 2.5-s specification, the perception time was indirectly considered.] The 2.5-s specification appears to be based on the results of the Johansson and Rumar study (6) that 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 (tests), brake-reaction time was estimated to be 1.5 s. On what basis the additional 1 s was added to arrive at 2.5 s is not clearly stated in the AASHTO manual, 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 subjects, regardless of whether they were alerted or not, knew they were to apply the brakes on hearing a signal (a horn) in the car. As stated in the AASHTO manual, the 2.5 s is supposed to be "large enough to include the reaction time required for <u>nearly all</u> 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: (a) experiments that measure the entire perception-brake-reaction time or (b) by simply adding the individual values experimentally determined for each of the components, i.e., perception, decision, and limb movement. The first method is preferred because it is more realistic. The processes of detection, perception, decisionmaking, and physical response are often overlapping and cannot simply be added as step-by-step tasks. For instance, the driver can take his or her foot off the accelerator while he or she decides whether or not to stop.

There are numerous studies that 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 (7). Table 1 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 (primarily means) are considered only brake-reaction times under expected conditions. Taoka refers to it as "simply laboratory response time", which is not indicative of actual driving situations.

The second group are results of field driver response experiments that attempt to duplicate actual conditions. All of the studies have deficiencies inherent in their procedure that make 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 an object.

For a laboratory study, latency would be defined as the delay between the time the stimulus is presented and the time the eyes begin to move to the

Table 1. Summary of studies on brake-reaction time.

Investigation	Sample Size	Mean	SD	Percentile Values							Comments
				50	75	80	85	90	95	99	
Laboratory/Field-Controlled Studies											
Greenshields (8), 1936	1461	0.496	0.0913								Laboratory
	13	0.86									Automobile
	27	0.74									
Forbes and Katz (9), 1936; and DeSilva and Forbes (10), 1937	907	0.64									
Jones and others (9), 1936	889 truck drivers	0.697	0.121								
Konz and Daccarett (11), 1967	12	0.59									
	40	0.47									
On-the-Road Studies											
Moss and Allen (12), 1925	46	0.54							0.88	1.01	Response to auditory detection
Massachusetts Institute of Technology (MIT) (2), 1934	144 men 36 women	0.66		0.60				0.85	1.0		Response to brake light of car ahead
Drew (13), 1968	1000	0.57 (men) 0.62 (women)									Alerted condition
Norman (3), 1953	53	0.73		0.73	0.77	0.83	0.85	0.89	0.96	1.1	Alerted condition
Johansson and Rumar (6), 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
Mortimer (14), 1970	80	1.30		1.42		1.88			2.56	3.29	Response to brake light of car ahead
Sivak and others (15), 1981	311	1.23	0.62	1.04	1.55	1.70	1.88	2.15	2.36	2.68	Response to brake light

stimulus. This has relevance to the highway when the object is in the peripheral vision of the motorist either because the object is off to 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 (day dreaming, 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 (16) 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 others (17), who 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° curve, which is not unrealistic for driving situations, are as follows:

Percentile	Latency (s)
50	0.24
75	0.27
80	0.29
85	0.31
90	0.33
95	0.35
99	0.45

These data are based on only three subjects, however.

Eye-movement times for a target 20° off the visual axis averages about 0.09 s according to White and others (18). This value is compatible with the 0.15-0.33 s cited by Matson, Smith, and Hurd (19) as the time for moving the eye to fixate to the left or right in scanning an intersection scene, which is a situation with a much wider angle than 20°.

The time it takes to bring the object into focus on the retina can be considered minimum fixation time. Data on this component are skimpy and they are qualified by their own experimentation apparatus, procedure, and purpose. Matson, Smith, and Hurd (19) cite a range of 0.1-0.3 s for fixation time, and Mourant and others (20) reported a mean fixation time of 0.27 s 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 in, which was arbitrarily selected by AASHO 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" (1). Objects this low would be animals, rocks, or other debris. More common objects, particularly at intersections, would be pedestrians and vehicles, both of which exceed the 6-in object height.

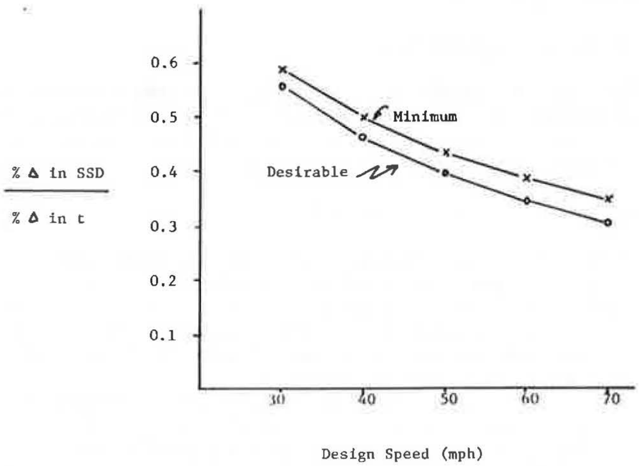
The fact that recognition time is a mental process makes it nearly impossible to measure it alone. The recognition component cannot be isolated from the total information-gathering process and, consequently, is measured only as part of the total perception phase. Data that could be used to approximate this component are available from the work of Ells and Dewar (21) and Ells and others (22). Ells and others (22) found the mean response time of 12 subjects responding to sign targets after being detected to be from 0.42 to 0.48 s. In another similar study, Ells and Dewar (21) found this to be about 0.6-0.7 s.

Table 2. Estimated perception-brake-reaction time for various percentiles of driving population.

Element	Perception-Brake-Reaction Time (s) at Following Percentile of Drivers					
	50	75	85	90	95	99
Perception						
Latency	0.24	0.27	0.31	0.33	0.35	0.45
Eye movement	0.09	0.09	0.09	0.09	0.09	0.09
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
Decision	0.50	0.75	0.85	0.90	0.95	1.00
Brake reaction	0.85	1.11	1.24	1.42	1.63	2.16
Total[a]	2.3	2.9	3.2	3.5	3.8	4.6

[a]Rounded to highest tenth of a second.

Figure 1. Sensitivity indices related to design speeds for minimum and desirable stopping-sight distance.



Although it has not always been recognized, there can be a decision process involved in the perception-brake-reaction 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. That 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 are that of Lunenfeld (23), which state that 85th percentile driver decision times for both expected and unexpected situations would be as follows:

Information Content (bits)	Decision Time (s)	
	Expected	Unexpected
0	0	0
1	0.7	1.0
2	1.3	1.6
3	2.0	2.6

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

The last component is brake reaction. The values suggested are those from the Johansson and Rumar study (6) under an unalerted condition.

The totals for the estimated percentile values range from 2.3 s for the 50th percentile to 4.6 s for the 99th percentile (see Table 2). The suggested 85th percentile of 3.2 s is 28 percent greater than the current specification value of 2.5 s.

These values 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 a human is capable of time-sharing sensory information processing and psychomotor tasks. Nonetheless, although higher, they are not unrealistic when compared with the perception-brake-reaction times cited by Mortimer (14) and Sivak (15) (see Table 1).

It is worthwhile noting that recent Canadian research, as reported by Scott (24), 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 s at a speed of 25 mph to 3.5 s at a speed of 85 mph.

Sensitivity of Standard to Driver Characteristic

One way to state the sensitivity of the standard with respect to a change in the driver characteristic specification is to express the percentage of change in the standard (i.e., stopping-sight distance) that results from a one percent change in the specification of the driver characteristic, assuming all other independent variables remain constant. This value 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. The formula that applies is

$$(dSSD/dP)/(SSD/P) = 1.47\,PV/[1.47\,PV + V^2/30(f \pm g)] \quad (1)$$

where

SSD = stopping-sight distance (ft),
P = perception-reaction time (s),
V = velocity of vehicle (mph),
f = coefficient of friction between tires and roadway surface, and
g = grade of roadway.

Application of Equation 1 for both minimum and desirable stopping-sight distances where there is no grade yields the sensitivity indices illustrated in Figure 1. At the design speed of 30 mph, a 1 percent change in the brake-reaction time will yield a 0.580 and a 0.563 percent change in the minimum and desirable stopping-sight distance, respectively. This percentage change decreases with increasing design speed partly due to the lower coefficient of friction values. The stopping-sight distance is less sensitive to a change in the brake-reaction time at higher speeds because the braking distance component $[V^2/30(f \pm g)]$ accounts for a greater proportion of the total distance as speed increases. The other observation is that the minimum values are more sensitive to a change in the perception-brake-reaction time than are the desirable values.

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 s for perception-reaction time. The minimum value for a design speed of 50 mph is less than the actual computed value (375 versus 376.4 ft). However, the effect of this shortfall is that the maximum allowable perception-reaction time is reduced only to 2.48 s.

Truck stopping-sight distances differ from automobile stopping-sight distances due to the relative efficiencies of the automobile and truck braking systems. Based on the Uniform Vehicle Code (25) performance standards for truck and automobile braking systems, it can be assumed that the average truck braking distance is 60 percent longer than the automobile braking distance. If that is the case, truck drivers are not provided 2.5 s of perception-reaction time. In fact, at higher design speeds, truck braking distance exceeds the total sight distance provided. The table below lists the computed truck perception-reaction time based on the AASHTO rounded design standards:

Design Speed (mph)	Minimum SSD (s)	Desirable SSD (s)
30	--	1.43
40	1.12	0.99
50	0.48	0.42
60	0.36	0
70	0	0

LATERAL CLEARANCE TO SIGHT OBSTRUCTION ON HORIZONTAL CIRCULAR CURVES

Current Standard

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. AASHTO (5) provides design standards for the provision of stopping-sight distance based on the following formulas:

$$m = (5730/D)[1 - \cos(SD/200)] \tag{2}$$

or

$$m = R[1 - \cos(28.65S/R)] \tag{3}$$

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),
- D = degree of curvature of the centerline of the inside lane (degrees) = 5730/R,
- S = sight distance measured along the centerline of the inside lane (ft), and
- R = radius of the curve measured to the centerline of the inside lane (ft).

Driver Characteristic

The AASHTO standard for lateral clearance to sight obstructions on horizontal circular curves is simply an application of the 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 s.

It should be noted that, on horizontal curves, an object situated on the roadway surface at the stopping-sight distance is not directly in front of the vehicle. Instead, it is off to one side at an angle, as shown in Figure 2. For example, for a driver of a vehicle traveling at 30 mph on a 10° horizontal curve, the stopping-sight distance on the roadway surface is 20° off center (i.e., the driver's field of view must be rotated 20° from straight ahead in order to look directly at the stopping-sight-distance point). At the maximum design degrees of curvature, the off-center angles range between 30° and 50°. Visual acuity has been observed to decrease significantly outside of an individual's 10° field of view (5° off center). Therefore, in order for a driver to sight an object in the roadway at the stopping-sight distance, the driver's field of view must be shifted away from straight ahead. It must be expected that all drivers do shift their field of view to encompass more of the roadway surface and do have sufficient eye movements to bring the stopping-sight-distance point within foveal view. Based on this expectation, it is assumed that the perception time estimates developed earlier under the section on Stopping-Sight Distance are likewise valid estimates for horizontal curve applications.

Sensitivity of Standard

The direct sensitivity of middle ordinate distance to changes in perception-reaction time is calculated by the following equation, which is a function of the first derivative of Equations 2 and 3:

$$(dm/dRT)(1/m) = (VD/7814)[\cos(SD/22\,920)] \tag{4}$$

where the trigonometric function is expressed in radians. The instantaneous percentage change in middle ordinate distance as a result of changes in perception-reaction time ranges between 44 and 24 percent per one-tenth second change in perception-reaction time (refer to Table 3).

The sensitivity of the degree of curvature to perception-reaction time is likewise given in Table 3. The table lists only the sensitivities for horizontal curves designed at the maximum degree of curvature. For curves designed at less than the maximum, the sensitivity rates are roughly proportionately smaller. The table shows that, the higher the degree of curvature (and thus the lower the design speed), the greater is its sensitivity to changes in the driver characteristic perception-reaction time. In comparison with the sensitivity of the middle ordinate distance, the sensitivity of the degree of curvature is slightly greater.

Critique of Standard

The AASHTO design standards for lateral clearance on horizontal curves fail to consider three important factors. First, it has been demonstrated earlier in this paper that trucks do not stop at the same deceleration rate as do automobiles.

On vertical curves, the higher driver eye height for trucks compensates to some degree for the relative inefficiency of the truck braking system. However, on horizontal curves, the additional eye height for drivers of trucks does not necessarily provide them with additional sight distance. Therefore, truck stopping-sight distance is not provided for in the AASHTO design standards.

The second factor not addressed directly in the AASHTO design standards is limited visibility conditions (e.g., nighttime). The vehicle's headlight beam must reach the stopping-sight-distance location in order for the driver of the vehicle to have an opportunity to sight an object in the roadway. There are potentially two different reasons why the headlight beam may not reach the necessary distance. Either the horizontal spread of the head-

Figure 2. Angular location of object at stopping-sight distance on horizontal circular curve.

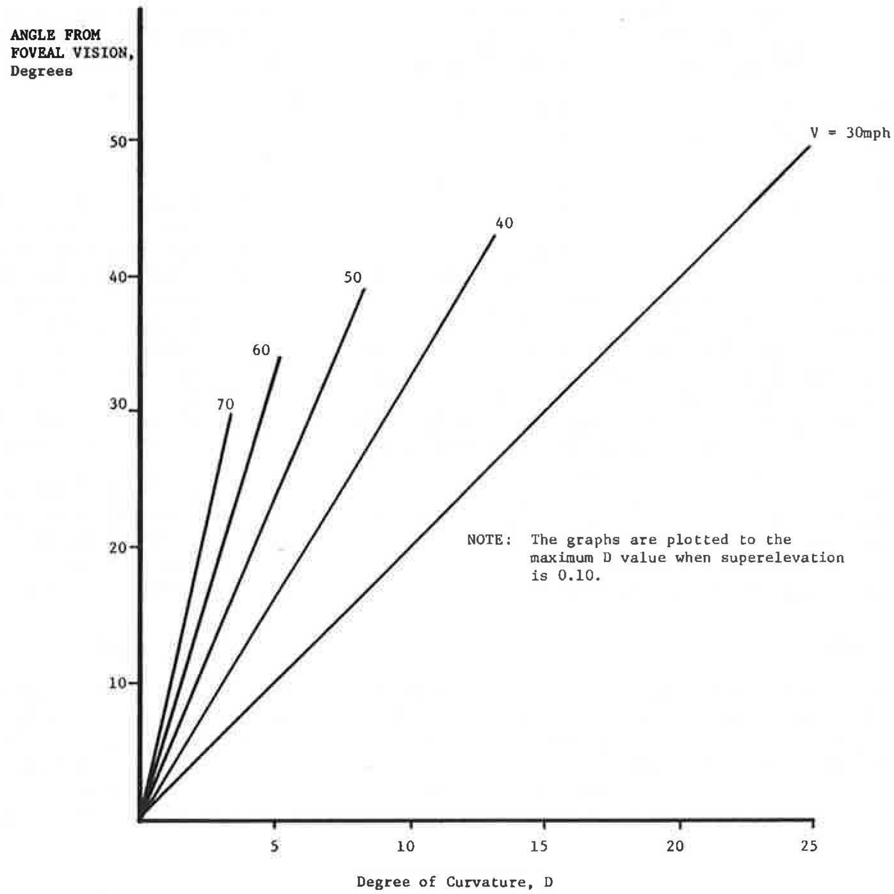


Table 3. Sensitivity of horizontal curve characteristics to changes in perception-reaction time.

Design Speed (mph)	Minimum m at Maximum D (ft)	[(dm/dRT)(1/m)] (%/0.1 s)	Maximum D (°)	[(dD/dRT)(1/D)] (%/0.1 s)
30	20.4	4.42	24.75	-4.55
40				
Minimum	20.4	3.92	13.25	-3.98
Desirable	28.0	3.71	13.25	-3.79
50				
Minimum	25.3	3.41	8.25	-3.46
Desirable	37.9	3.15	8.25	-3.21
60				
Minimum	28.6	3.03	5.25	-3.06
Desirable	45.7	2.76	5.25	-2.80
70				
Minimum	28.6	2.77	3.5	-2.78
Desirable	53.6	2.43	3.5	-2.46

light beam is not sufficient or the characteristics of the potential sight obstruction may permit a driver with an eye height of 42 in to see in the daylight over the obstruction but do not permit a light beam from a 2-ft-high headlamp to pass over the obstruction and strike the roadway surface at the stopping-sight distance.

The third factor not addressed by the AASHTO standards is the reduced coefficient of friction between the vehicle tires and the roadway surface when a vehicle is traveling on a horizontal curve. Newman and others (26) state that a vehicle braking on a horizontal curve is not afforded the full friction force observed in skid tests. Instead, the actual friction available is a reduced value, which "reflects the amount of side friction used for cornering, and can be calculated as the vector resultant of both total available friction and cornering friction." The table below compares the actual required stopping-sight distance (as predicted by Neuman) with the current design values and compares the actual minimum and current design values for middle ordinate distance:

Design Speed (mph)	Stopping-Sight Distance (ft)		Middle Ordinate Distance (ft)	
	Design	Actual	Design	Actual
30				
Minimum	200	183	21.3	17.8
Desirable	200	206	21.3	22.6
40				
Minimum	275	274	21.7	21.5
Desirable	325	334	30.2	31.9
50				
Minimum	375	386	25.2	26.7
Desirable	475	498	40.2	44.1

Design Speed (mph)	Stopping-Sight Distance (ft)		Middle Ordinate Distance (ft)	
	Design	Actual	Design	Actual
60				
Minimum	525	509	31.4	29.6
Desirable	650	675	48.0	51.7
70				
Minimum	625	616	29.7	28.9
Desirable	850	880	54.9	58.8

Figure 3 illustrates the actual maximum allowable perception-reaction times on horizontal curves designed for desirable stopping-sight distances. The values are plotted to the maximum degree of curvature when the superelevation is 0.10. The figure shows that none of the desirable design values permit the specification for perception-reaction time (2.5 s). Even if the estimated median value (2.3 s) is taken, the desirable stopping sight distance for 50-, 60-, and 70-mph design speeds are not sufficient. Therefore, significant portions of the driving population are being excluded by the current design standards.

INTERSECTION SIGHT DISTANCE--CASE III

Current Standard

Case III intersection sight distance applies to intersections controlled by stop signs on minor roadway approaches. The current 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. The AASHTO formulation for computing the required sight distance is as follows:

$$D = 1.467V(J + t_a) \tag{5}$$

where

- D = minimum or desirable sight distance along the major highway from the intersection (ft),
- V = design speed on the major highway (mph),
- J = sum of the perception time and the time required to actuate the clutch or actuate an automatic shift (s),
- t_a = time required to accelerate and traverse the distance S to clear the major highway pavement (s),
- S = distance that the crossing vehicle must travel to clear the major highway (ft) = D + W + L,
- D = distance from near edge of pavement to the front of a stopped vehicle (ft),
- W = pavement width along path of crossing vehicle (ft), and
- L = overall length of vehicle (ft).

Driver Characteristic

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

Figure 3. Maximum allowable perception-reaction time on horizontal curves based on desirable stopping-sight-distance standards.

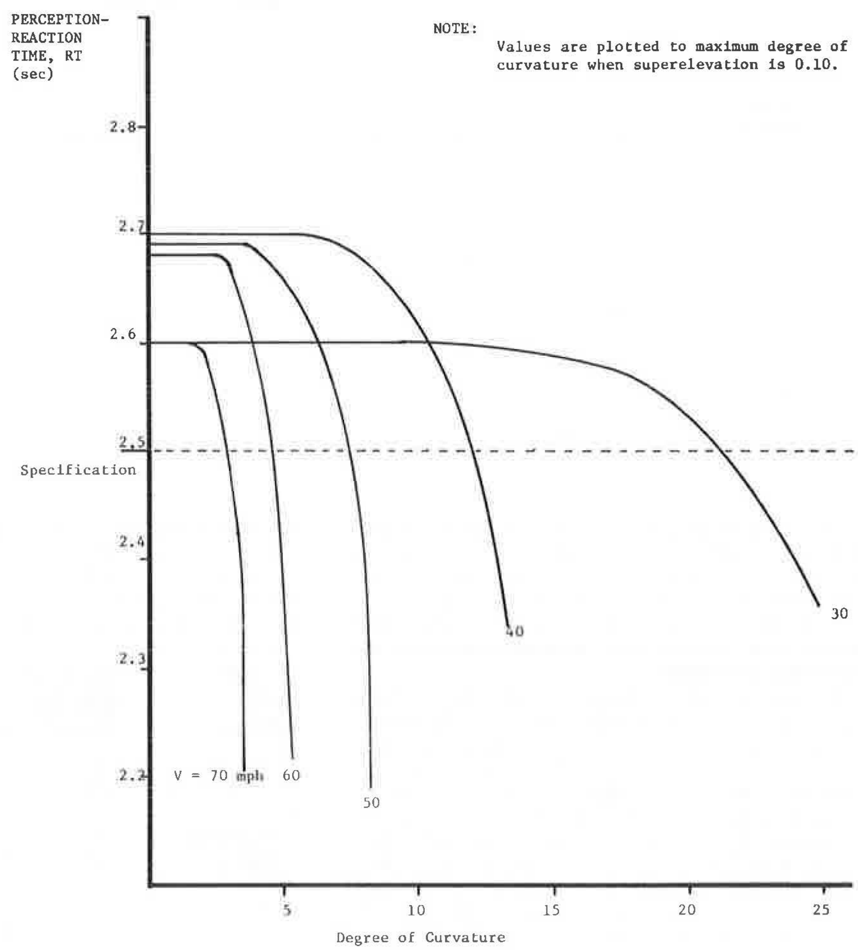
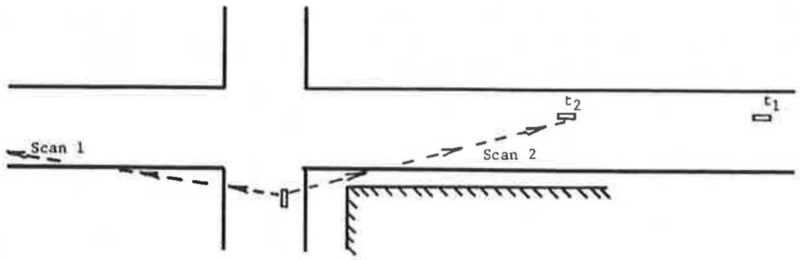
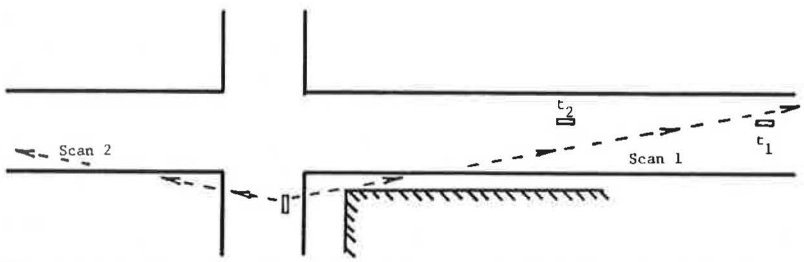


Figure 4. Case III intersection sight distance: driver of stopped vehicle scanning approaching roadways.



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.

time from the driver's first look for possible oncoming traffic to the instant the car begins to move." A value of 2.0 s is the current specification, which represents "the time taken by a small percentage of slower drivers" (5). The historical basis for this value cannot be determined.

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 and eye movement to scan intersection,
2. Fixation and 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 and eye movements on which the decision to proceed or stay is made is the last one. If the driver scans one direction (as illustrated in Figure 4) 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 4, the driver has performed the two scans and has decided to proceed, even though another scan to the right would reveal an approaching vehicle. The distance that 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 or her head and/or eyes to the left, to decide to proceed, and to move his or her foot to the accelerator. Thus, the head- and 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 at intersections. In order to develop an estimated distribution of values for the driving population, it is necessary to assign values to individual elements of perception-reaction time and sum them.

Robinson (27) 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 s. Johansson and Rumar (6) measured the brake-reaction time for drivers in an alerted condition. Brake-reaction time is appropriate in this application because accelerator reaction is simply a motor movement equal and opposite to brake reaction; the driver is in an alerted condition due to the intersection scan. Johansson and Rumar found values ranging from 0.63 s at the 50th percentile to 1.21 s at the 95th percentile. Interestingly, if the 85th percentile value of 0.92 s is added to Robinson's 1.1 s described above, a total perception-reaction time of 2.02 s results (only 0.02 s higher than the current specification).

The 1965 edition of the Traffic Engineering Handbook (28) provides the following data as the total time required for a driver to scan one leg of an intersection:

Item	Time (s)
Shift (head and eye movement)	0.15 to 0.33
Fixate on object	0.10 to 0.30
Total	0.25 to 0.63

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

Item	Time (s)
Head and eye movement	0.24
Fixation	0.20
Decision	0.85
Reaction	0.92
Total	2.21

Sensitivity of Standard

The percentage change in minimum or desirable sight distance for a unit change in the driver characteristic perception-reaction time ranges from approximately 0.14 percent/s for passenger vehicles to 0.08 percent/s for WB-50 vehicles. The incremental change in intersection sight distance as a function of unit changes in perception-reaction time ranges from 4.4 ft/0.1 s for a 30-mph design speed to 10.3 ft/0.1 s for a 70-mph design speed.

VEHICLE CHANGE INTERVAL

Driver Characteristic

For the driver who sights a traffic signal that has just turned yellow and who decides to stop for the imminent red signal, two driver characteristics are involved: perception-reaction time and comfortable deceleration rate. The following analysis addresses only the former characteristic.

The current specification for the driver characteristic perception-brake-reaction time is 1.0 s as stated in the Transportation and Traffic Engineering Handbook (29) and the Traffic Control Devices Handbook (30). The use of the 1.0-s value can perhaps be traced back to a 1934 MIT research effort (2), which found that 95 percent of the sampled drivers had brake-reaction times of 1 s or less when in an alerted condition. Subsequent studies in the early 1960s of driver reactions to the amber signal by Gazis, Herman, and Maradudin (31) and by Olson and Rothery (32) continued the use of the 1.0-s specification. However, a recent field study by Wortman and Matthias (33) 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 s. In fact, at most of the intersections, the 85th percentile value approached 2 s.

Experimentally derived data on driver perception-reaction time were discussed earlier under the section on Stopping-Sight Distance. Applicable to the yellow signal case is the data presented by Johansson and Rumar (6) 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 on hearing the stimulus, no appreciable decision time would be expected. The Johansson and Rumar data distribution is as follows:

Percentile	Alerted Brake-Reaction Time (s)
50th	0.63
85th	0.93
95th	1.21

It should be noted, however, that this distribution depicts a pure case of brake reaction. That is, the decision is made instantaneously on perceiving the circumstances (no decision time) and perception of the situation occurs simultaneously with the onset of the situation (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 to yellow and decide whether to continue through the intersection or to stop prior to the intersection. Detection and/or 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 (34).] 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 who approaches an uncontrolled intersection and who must judge relative speeds of potentially conflicting vehicles. However, we will assume the same decision time distribution presented earlier under Stopping-Sight Distance. 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:

Percentile	Perception-Brake-Reaction Time (s)
50th	1.13
85th	1.77
95th	2.16

These empirically derived values compare quite favorably with the observed values documented by Wortman and Matthias (33). For example, the mean value observed was 1.30 s as compared with the 1.13-s median value derived above. Wortman and Matthias' average 85th percentile value for all study intersections was 1.8 s; the estimate above is 1.77 s.

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.

Sensitivity of Standard

The effect of increasing the perception-reaction time on the vehicle clearance interval is 1:1. That is, a 1-s increase in the specification for perception-reaction time necessitates a 1-s increase in the vehicle clearance interval.

RECOMMENDATIONS

The analyses presented above are only a cursory review of the interrelations between the driver characteristic perception-reaction time and several highway design and operations standards. A more thorough analysis is presented in the upcoming Federal Highway Administration (FHWA) research report, Driver Characteristics Impacting on Highway Design and Operation.

Throughout the above analyses, a number of observations were made regarding perception-reaction time and the various standards. These comments are summarized below.

The stopping-sight-distance standard does not adequately account for the braking inefficiencies of trucks. Based on the calculations presented earlier, it would appear that trucks are unable to stop within current standard distances. Further research into truck operating characteristics, especially its braking capabilities, would enable the improvement of the stopping distance formulation to more accurately depict the action of a truck stopping.

The analyses contained in this report indicate that several specification values for perception-reaction time may in fact be too low. Because these conclusions are based principally on aggregated simulation results, it is recommended that extensive field testing be undertaken to establish definitive and documentable specification values.

The current specification value for perception-reaction time in the vehicle clearance interval has been criticized as being too low. A concerted effort should be made to establish scientifically developed specification values for both perception-reaction time and vehicle deceleration rate.

The driver characteristic perception-reaction time in the case III intersection sight distance standard should be redefined. It is recommended, however, that the specification value be kept at 2.0 s.

ACKNOWLEDGMENT

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Abridgment

Corrections to Driver Characteristic Specifications and Standard Formulations for Intersection Sight Distance

KEVIN G. HOOPER AND HUGH W. McGEE

This report documents an evaluation of the American Association of State Highway and Transportation Officials standards for intersection sight distance and how they are affected by driver characteristics. The study involved the development of a population profile for the driver characteristic perception-reaction time, and the calculation of the sensitivity of each standard to realistic changes in the driver characteristic. The study found that, for case I intersection sight distance, the driver is not provided sufficient time or distance to take evasive action from an opposing vehicle, and for case II, adequate sight distance in order to stop before the intersection is not provided despite the intent of the standard to enable such an action. Proper formulations are developed in the paper and proposed as revisions. The effect of these revisions on current standard intersection sight distances is described and quantified. In addition, recommendations are made to increase the perception-reaction-time value for case I from 2.0 to 3.4 s and for case II from 2.5 to 3.4 s.

The 1965 American Association of State Highway Officials (AASHO) Blue Book, A Policy on Geometric Design of Rural Highways (1), and its draft revised versions (2) present standards for adequate sight distance at intersections. Abridged analyses of the interrelations between characteristics and the sight-distance standards for cases I and II follow. Included in these analyses are investigations into the appropriateness of the current standard formulations.

CASE I: ENABLING VEHICLES TO ADJUST SPEED

Current Standard

At an intersection where no approach leg is controlled by stop signs, yield signs, or traffic signals, a driver of a vehicle who approaches the intersection must be provided adequate sight distance both to perceive the potentially conflicting movement of a crossing vehicle and to take the necessary countermeasure. The American Association of State Highway and Transportation Officials (AASHTO) (2) formula for computing the minimum allowable sight distance on each leg is as follows:

$$D = 1.467V(\mathrm{PRAT}) \qquad (1)$$

where

D = minimum sight triangle distance (ft),
V = vehicle velocity (mph), and
PRAT = perception-reaction-action time (s).

The formulation assumes that the appropriate minimum distance from an intersection, at the point where the driver first observes a vehicle approaching on an intersecting road, is that which is covered during both the driver's perception and reaction time (which includes 1 s in which the speed of the vehicle is adjusted by the driver's reaction). AASHTO recommends the use of between 2.5 and 3.0 s as the value for the perception-reaction-action time: 1.5-2.0 s for perception and reaction and 1.0 s for the action (acceleration or deceleration).

Driver Characteristic

The perception-reaction process in this case is the ability of a driver to perceive a vehicle moving across his or her path, judge its trajectory in relation to his or 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 overall task. In the absence of any empirical research, estimates of the actual distribution of perception-reaction times for the driving population have to 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 (i.e., before the vehicle actually accelerates or decelerates), the following steps would likely be considered:

1. Driver picks up (through peripheral vision) an object moving toward the intersection;
2. After a latency period, eye or head movement or both detects the object;

3. Object is recognized as vehicle;
4. Opposing vehicle's speed and time to reach intersection are estimated;
5. Decision is made on whether deceleration or acceleration is required; and
6. Decided action is initiated (e.g., foot moves to brake pedal).

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. Values for each of the above steps can be approximated by using current research literature, as is detailed by McGee and Hooper (3). The resulting estimated total values for various percentiles of the driving population are as follows: current specification, 2.0 s; 50th percentile, 2.6 s; 85th percentile, 3.4 s; and 95th percentile, 4.0 s.

CASE II: ENABLING VEHICLES TO STOP

Current Standard

The second case of intersection sight-distance requirements cited by AASHTO (2) deals with the situation in which "it is assumed 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." Simply stated, the AASHTO policy requires that a driver of a vehicle moving toward an uncontrolled intersection be able to see a vehicle approaching the intersection from another leg when each vehicle is situated at its stopping-sight distance from the intersection.

The AASHTO (2) formulation for stopping-sight distance is as follows:

$$SSD = 1.47(RT)V + V^2/30(f \pm g) \quad (2)$$

where

SSD = stopping-sight distance (ft),
RT = perception-brake-reaction time (s),
V = initial vehicle velocity (mph),
f = coefficient of friction between tires and roadway, and
g = grade of roadway (ft/ft).

Driver Characteristic

The case II intersection sight-distance standard is based directly on the driver characteristic perception-brake-reaction time. The AASHTO specification for stopping-sight distance perception-brake-reaction time is 2.5 s. However, in this application, the driver is required to perform a more complex set of actions than is required in sighting and stopping for a stationary object in the roadway. The driver needs more time to pick up the other moving vehicle through peripheral vision and to move the head or eyes or both to detect the object. The decisionmaking component of the perception-brake-reaction task is also more complex because the driver must first evaluate the velocity of the other vehicle and the potential for a collision before deciding to take a particular evasive action. The actual perception-decision-reaction process that a driver must follow in a case II situation is similar to the process described earlier for case I situations. Therefore, the case II perception-reaction time approximates the values listed previously for case I. The values range from an estimated 2.6 s for the 50th percentile to an estimated 4.0 s for the 95th percentile. The suggested estimated 85th percentile value of 3.4 s is well above the specification value of 2.5 s.

The computed case II intersection sight distances are listed in Table 1 for the current specification value and for the estimated 50th, 85th, and 95th percentile drivers. The percentage differences between these values and the current rounded stopping-sight distance standard values are also given in Table 1. For example, assuming a perception-brake-reaction time value of 3.4 s, the calculated desirable intersection sight distance would be 18 percent greater than the current standard value at 30 mph and nearly 10 percent greater at 70 mph.

It should be emphasized that these values for perception-brake-reaction time 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 because they were based on relatively small sample sizes and less-than-actual driving conditions.

Critique of Standard

The AASHO (1) definition of the conditions that describe 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 vehi-

Table 1. Computed case II intersection sight distances based on various values of perception-brake-reaction time.

Design Speed (mph)	Design Stopping-Sight Distance[a] SSD (ft)	Condition	RT = 2.5 s (specification) SSD (ft)	RT = 2.5 s Increase Above Standard (%)	RT = 2.6 s (50th percentile) SSD (ft)	RT = 2.6 s Increase Above Standard (%)	RT = 3.4 s (85th percentile) SSD (ft)	RT = 3.4 s Increase Above Standard (%)	RT = 4.0 s (95th percentile) SSD (ft)	RT = 4.0 s Increase Above Standard (%)
30	200	Minimum	177	-12	181	-9.5	214	7.0	239	20
	200	Desirable	196	-2.0	200	0	235	18	262	31
40	275	Minimum	267	-2.9	272	-1.1	315	15	346	26
	325	Desirable	313	-3.7	319	-1.8	366	13	401	23
50	375	Minimum	376	0.3	383	2.1	435	16	473	26
	475	Desirable	461	-2.9	468	-1.5	527	11	571	20
60	525	Minimum	501	-4.6	509	3.0	570	8.6	616	17
	650	Desirable	634	-2.5	643	-1.1	713	9.7	766	18
70	625	Minimum	613	-1.9	622	-0.5	690	10	741	19
	850	Desirable	840	-1.2	850	0	932	9.6	994	17

Note: Computed Stopping-Sight Distance at Following RT Values.

[a]Design values from AASHO (1).

Figure 1. Illustration of inadequacy of case II formulation.

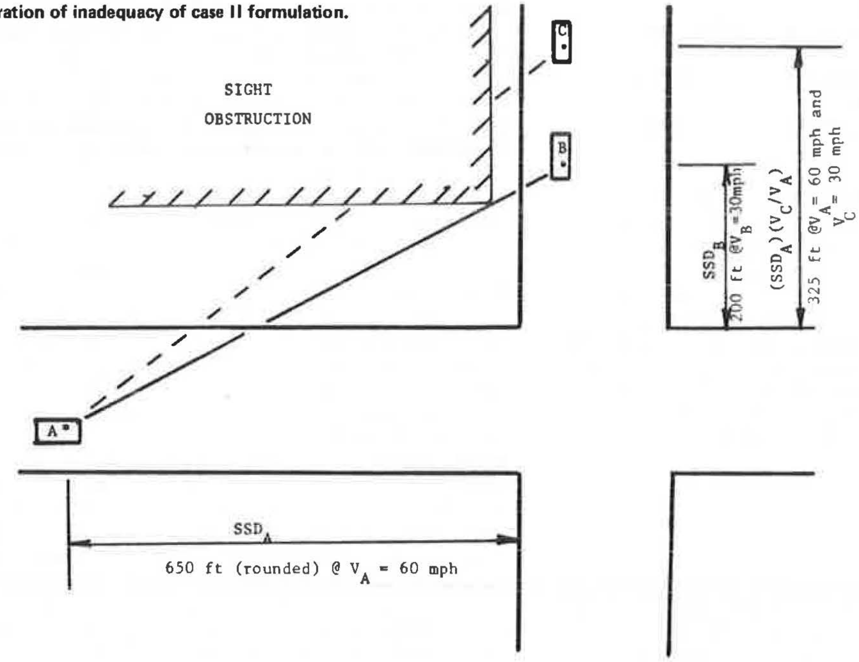

Table 2. Minimum distance of vehicle B from intersection based on suggested revised methodology for case II intersection sight distance.

Design Speed of Vehicle A (mph)	Minimum Distance of Vehicle B from Intersection[a] (ft) at Following Velocities					AASHTO SSD[b] (ft)	
	V_B = 30 mph	V_B = 40 mph	V_B = 50 mph	V_B = 60 mph	V_B = 70 mph	V_A	V_B
30	196	261	326	391	457	200	200
40	235	313	392	470	548	325	325
50	277	369	461	553	646	475	475
60	317	423	528	634	739	650	650
70	360	480	600	720	840	850	850

[a] Calculated with the suggested revised formulation: $D_B = [1.467(RT)V_A + V_A^2/30(f \pm g)]\,(V_B/V_A)$. Derivation and explanation of the formula is provided in the text. Values for V_A, V_B, and SSD are based on full (desirable) velocities. Perception-brake-reaction time is assumed to be 2.5 s—the specification for these calculations.
[b] The listed stopping-sight-distance values are the "rounded for design" values provided by AASHTO (2).

cle before reaching the intersection." There are three different relative approach patterns for two vehicles at an uncontrolled intersection: either vehicle A arrives first, or vehicle B arrives first, or they arrive simultaneously. According to the above definition, the case II sight distance should enable a driver confronted with the collision scenario to bring the vehicle to a complete stop before reaching the intersection. As is explained below, the AASHTO standards for stopping-sight distance do not provide this adequate sight distance in some cases. 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, we are back to case I (enabling vehicles to adjust speed).

In terms of the sketch in Figure 1, the AASHTO case II intersection sight distance requires that the driver of vehicle A be able to see vehicle B, each of which is situated at its respective stopping-sight distance from the intersection. Immediately on sighting vehicle B, the driver of vehicle A reacts and brings vehicle A to a stop before reaching the intersection. However, if vehicle B did not exist, the driver of vehicle A would pass the SSD_A point without perceiving the presence of a vehicle on the other leg of the intersection. For example, vehicle C cannot be seen by the driver of vehicle A; if both vehicle A and vehicle C proceed at a constant speed (60 and 30 mph, respectively, in Figure 1), they will arrive at the intersection simultaneously. Thus, in reality, the situation is a version of case I, where a vehicle is not given enough distance to stop but instead is provided distance to only decelerate to avoid the other vehicle.

It is recommended that the case II methodology be revised so that the driver of a vehicle located at its stopping-sight distance from an uncontrolled intersection be provided a direct line of sight to an approaching vehicle located the greater of the two distances described below:

1. The stopping-sight distance that corresponds to the latter vehicle's velocity, or
2. The distance at which the latter vehicle would be in order for the two vehicles to collide if the speeds of both were maintained.

In general, the distance for the slower-approaching vehicle would need to be increased. Table 2 lists computed minimum distances for various combinations of approach velocities.

ACKNOWLEDGMENT

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Operation. The contents reflect our views and we are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the U.S. Department of Transportation.

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Visual Complexity and Sign Brightness in Detection and Recognition of Traffic Signs

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The effects of sign luminance on the detection and recognition of traffic-control devices are mediated through contrast with the immediate surround. In addition, complex visual scenes are known to degrade visual performance with targets well above the visual threshold. A laboratory study was conducted to determine ways of measuring visual complexity and to assess the capability of changes in sign luminance to offset decrements in performance that result from added complexity. Positive results were found for warning, construction, and stop signs but not for the black-on-white regulatory sign. Regression equations that use complexity factors, contrast, and target variables suggested that, in complex scenes, complexity is a more significant determinant of sign detection than brightness or contrast. A field study was also conducted to determine if these findings could be observed in terms of real-world driver performance. The effects of visual complexity were observed in the field, and increasing sign brightness improved sign detection and recognition under specific conditions.

The role of sign brightness and the visual complexity of the nighttime highway environment in the detection and recognition of traffic signs was studied in both laboratory and field situations. The laboratory study permitted control over a large number of highway scenes that varied in the amount of visual clutter. The field study was undertaken to determine whether sign brightness and visual complexity had an observable effect on driver behavior.

LABORATORY STUDY

The primary objective of the laboratory study was the development of a metric for visual complexity based on target-independent characteristics of the visual field. In this regard, the study addressed the question of whether a sign location that causes sign-recognition problems can be identified from measurements or observations of the location itself. A secondary objective of the laboratory study was to determine whether increases in sign brightness offset decrements in visual performance that result from the visual complexity of the location.

The detection of a visual target, such as a traffic sign, is influenced by the characteristics of the target and by the contrast of these target characteristics with similar dimensions of the surround. For example, the attention-getting value of a target increases as

1. The target's brightness increases (1,2),
2. The brightness contrast between the target and its surround increases (2-7),
3. The brightness contrast between different parts of the target increases (e.g., sign legend to background) (2,5),
4. The target's size increases relative to other stimuli in the visual field (2,8,9),
5. The shape of the target contrasts with noise items (10), and
6. The target's hue contrasts with noise (5,11).

In addition to the effects of target characteristics, the characteristics of a target's surround also influence the likelihood of target detection. Specifically, several basic studies suggest that target conspicuity increases as

1. The number of noise elements in the visual field decreases (12-18),
2. The overall density of noise items in the visual field decreases (19-21),
3. The density of noise items immediately adjacent to the target decreases (22),
4. The distance between the target and noise increases (15-17,23),
5. The target is located further from the center of the visual field than the noise (versus when the target is located closer to the center of the visual field than the noise) (23-27),
6. The number of irrelevant classes of stimuli in the visual field decreases (i.e., as the visual field becomes more homogenous) (28), and
7. The variability within each irrelevant class of stimuli decreases (21).

Because the majority of the studies listed above reflect basic research efforts that often use abstract targets located within relatively sterile visual matrices, operational definitions that facilitate measurement of these dimensions in complex highway scenes have not been established. One applied study (29) that did use photographs of actual road scenes as stimuli found that background complexity had a substantial negative effect on detection. The components of background complexity, however, were not evaluated.

The experimental methodology of the laboratory study attempted to simulate real-world variation in visual complexity via photographic stimuli. In order to simulate a driver's search for signing in-

Table 1. Overview of variables analyzed.

Item	Visual	Photometric
Scene	Twenty-two variables that measure uniformity and brightness of scene; number, size, and location of light sources; presence of recognizable detail and distracting things; general descriptions; e.g., number of point sources of light, number of traffic signs, wet or dry pavement, and land use	Scene illuminance measured at observer's eye
Surround	Thirteen variables that measure uniformity and brightness of surround, presence of other signs, and number and size of light sources, e.g., number of bright point sources, number of large bright sources of light, uniformity of surround, and brightness of surround	Minimum surround luminance, maximum surround luminance, and average surround luminance
Contrast	Three variables that measure the brightness of the sign relative to the area immediately adjacent to it, e.g., proportion of perimeter darker than sign, proportion of perimeter lighter than sign, and proportion of perimeter equal to sign brightness	Minimum external contrast, maximum external contrast, and average external contrast
Sign	Three independent variables: device type, sign brightness, and distance	Luminance of sign legend, luminance of sign background, and integrated target luminance

Figure 1. Factorial arrangement of distance by brightness by device.

	Brightness							
	Dim				Bright			
	Sign Type				Sign Type			
Distance (feet)	STOP	SPED	CROS	DTOR	STOP	SPED	CROS	DTOR
250								
400								
600								
800								

formation, uncertainty about what sign would appear was created by using four distinctly different signs, and uncertainty about where to look for a sign was maintained by varying scenes and the placement of signs within scenes.

Variables

The independent variables and their respective values are listed below:

1. Type of device: (a) DETOUR (DTOR)--black on orange; (b) SPEED ZONE AHEAD (SPED)--black on white; (c) STOP--white on red; and (d) PEDESTRIAN CROSSING (CROS)--black on yellow;
2. Sign brightness: low and high; and
3. Distance from observer: 250, 400, 600, and 800 ft.

(Note, for sign brightness for each sign type at a single distance, the luminance of target signs was controlled such that bright signs were typically twice as bright as dim signs and the standard deviation for a given brightness was kept below 25 percent of the mean.) Device type was varied to explore the effect of visual complexity on those traffic signs with colors and shapes that are used most frequently or in the most critical situations. Sign brightness was manipulated in order to identify the conditions under which increased brightness is likely to improve sign recognition. Distance was included to create the uncertainty necessary in a study of conspicuity and to assess the effects of distance or size.

Because the role of visual complexity in detection and recognition is not well understood, target and contrast variables were also included in the design so that the relative importance of each and their interactions could be evaluated. Four categories of variables were measured in each visual stimulus: target, contrast, surround, and scene. Because it was not known what size area should be referred to as surround, the target's surround was defined in two ways. Specifically, the surround was defined as a circular spatial area that surrounds the center of the target sign, which subtends a radius of 1° and 2° of visual angle, respectively. Measurements in all categories were made both visually and photometrically to provide a most comprehensive assessment. The pool of variables available for analysis can be subdivided as shown in Table 1.

Experimental Design

The design of this study was complicated by the need to assess both the effects of the independent variables, which were controlled, and the variables that describe visual complexity, which were largely uncontrolled. The first of these objectives was amenable to analysis of variance, while the second was suited primarily to methods of correlation and regression.

Figure 1 shows the factorial arrangement of the three independent variables. The contents of each of these cells were observations of either scenes or subjects, depending on the analysis to be performed. When the unit of observation was subjects, scores were the proportion of correct responses over 20 scenes. When the units of observations were scenes, scores were the proportion of correct responses over 40 subjects.

The complete factorial of brightness (2) by devices (4) by distance (4) resulted in 32 cells. Eighty scenes were grouped into four sets of 20, and each set of scenes was crossed with only 8 of the 32 cells. This resulted in a total of 640 stimuli. An image of each scene without a target sign was also included, bringing the total stimuli to 720. In short, then, each roadway scene was presented nine times: eight times with a target sign and one time without a sign. Of the eight instances of the scene with signs, there were two signs at both levels of brightness at two distances.

Procedure

Subjects attended three experimental sessions, scheduled on different days, during which they were individually tested. During each session, which

Table 2. Multiple correlations of visually and photometrically determined scene, surround, contrast, and target brightness measures with criterion of sign recognition.

Measure	STOP		DTOR		CROS		SPED	
	Visual	Photometric	Visual	Photometric	Visual	Photometric	Visual	Photometric
Scene	0.70	0.06[a]	0.53	-0.32[a]	0.66	-0.31[a]	0.72	-0.26[a]
Surround	0.48	0.12	0.60	0.26	0.44	0.10	0.61	0.19
Contrast	0.28	0.24	0.38	0.39	0.27	0.12	0.52	0.42
Target brightness	0.27	0.59	0.26	0.52	0.15	0.49	0.31	0.54
Distance[b]	-0.61	-	-0.54	-	-0.52	-	-0.45	-

[a] Zero-order correlation of scene illuminance with criterion.
[b] Zero-order correlation.

lasted about 90 min with two 5-min rest periods, a subject responded to 240 projected stimuli so that data were collected for all 720 different stimuli after three sessions. Each session was preceded by both training and practice with the actual task. The task required the subject to view nighttime road scenes and to report, by using specific labels, their recognition of any of nine targets summarized in the table below:

Category	Target	Label
Road	Road curvature	Curve
	Solid center and/or edgeline	Solid
	Dashed lane line	Dashed
Traffic	Traffic moving in same direction	Same
	Traffic moving in opposite direction	Opposite
Signs	STOP sign	Stop
	PEDESTRIAN CROSSING	Cross
	DETOUR	Detour
	SPEED ZONE AHEAD	Speed zone

Subjects were shown stimuli for 3-s durations with a 15-s interstimulus interval during which blank images were projected to maintain constant dark adaptation. A quiet buzzer alerted subjects to the onset of the next trial. Subjects reported targets in different orders, which may have reflected personal search strategies or degrees of certainty.

Apparatus and Stimuli

The stimuli were composite 2.4x2.8-in color transparencies made from separate original transparencies of the scene and the sign. The procedure developed permitted any sign to be inserted into any location of any scene.

A 5x6.7-ft glass-beaded screen was located on one wall of the room while the projection equipment was isolated to limit the sound and light contamination of the experimental situation. The subject was seated 11.9 ft from the screen and the projected image constituted a 30°x24° visual field.

Subjects

A total of 40 volunteer subjects participated in the study and were reimbursed for completion of all three sessions. They were solicited from a larger sample of subjects who had been vision tested within the previous year. All subjects were required to have a driver's license and to wear corrective lenses if their license required it. The sample was fairly evenly divided as to sex but stratified on age based on the nighttime driving patterns obtained from data provided by the Federal Highway Administration (FHWA).

Results and Discussion

An analysis of variance revealed that all main effects and interactions (except sign by brightness) were significant. Obviously, bright signs were easiest to recognize, as were signs located at the nearer distances that subtended larger visual angles. Of more interest was the fact that the PEDESTRIAN CROSSING sign was easiest to recognize ($P = 0.89$) and the SPEED ZONE sign most difficult ($P = 0.57$). If response bias, which favored the SPEED ZONE sign, had been factored out, the difference between these proportions would have been even greater. The distance-by-brightness interaction showed that brightness had the greatest effect at the far distances, namely, 600 and 800 ft. The sign-type-by-distance interaction showed that the PEDESTRIAN CROSSING sign was affected least by distance and the DETOUR sign was affected most.

Regression equations were computed to evaluate the relative effectiveness of different determinants of sign recognition for each of four groups of variables measured visually and photometrically. The multiple R's (based on all variables in each group to a limit of 20, which produced asymptotic values) obtained for scene, surround, contrast, and target brightness and the zero-order R's for distance and photometrically determined scene illuminance are given in Table 2.

It should be remembered that the visual and photometric measurements within a category were not designed to measure the same thing, only the same concept or domain. Higher correlations do not imply that one method of measurement is more reliable. The differences are more likely to be attributable to differences in validity, since the underlying variables were generally different in substance as well as different in the method of measurement. For example, where visual assessments resulted in a higher correlation than photometric measurements, the difference is probably attributable to visual assessments that capitalize on the ingenuity and flexibility of the subjective process involved. Obviously, the 22 different visual measures of scenes are measuring more variance in the scene than a single measure of scene illuminance.

Inspection of Table 2 suggests that visually determined measures of the scene and surround were better predictors of detection than were photometrically determined measures. There was not much difference in visual and photometric measurements of contrast, while photometrically determined measures of target brightness were superior to visually determined brightness.

In general, the determinants of detection in order of importance were scene, surround, target brightness, and external contrast. Although surround factors were more important than target brightness (measured photometrically) for DETOUR and SPEED ZONE signs, the reverse was true for the STOP and PEDESTRIAN CROSSING signs. In any case, however, these differences were not large. Scene effects were most important for all but the DETOUR sign, where the difference between the effects of scene and surround factors was small (R = 0.53 versus 0.60).

The zero-order correlation of illuminance with the criterion revealed that brighter scenes resulted in poorer performance for all but the STOP sign, where the correlation was not significant. This may be attributable to the fact that the STOP sign was the darkest target used, and therefore the direction of contrast in bright scenes was not unfavorable to its recognition.

The zero-order correlations with distance (which also measured target size) are in Table 2 for reference. In general, this variable accounted for less variance than scene variables but more variance than the surround. In absolute terms, distance or size accounted for between 20 (SPEED ZONE) and 36 percent (STOP) of the variance in detection. (The variance accounted for is given by the correlation or multiple R^2.)

Because the determinants of detection (i.e., scene, surround, contrast, and brightness) are not independent of each other, multiple regressions were computed to evaluate the increments in predicted variance of using surround, contrast, and target variables in addition to scene variables. The R^2 for these equations is given in Table 3.

The predicted variance for scene variables ranged from 28 (DETOUR) to 52 percent (SPEED ZONE). About half of the difference in the predicted variance between these two signs was eliminated by the inclusion of surround variables in the regression equation. It is the R^2 for scene and surround that shows the proportion of variance in detection probability associated with scene complexity. For all signs, visual complexity (scene plus surround variables) accounted for more than 50 percent of the variance in the detection criterion.

The inclusion of contrast variables added little to the overall predictive validity; the greatest effect was a 4 percent increase (from 53 to 57 percent) for the DETOUR sign. The effect of adding target brightness was greater, ranging from a 4 percent increment for the PEDESTRIAN CROSSING sign to 10 percent for the STOP and SPEED ZONE signs. Brightness probably would have had even more importance if it had been put into the equation before contrast. The inclusion of distance had a modest effect of 7 percent on the DETOUR sign and 6 percent for the STOP and PEDESTRIAN CROSSING signs, but it had no effect for the SPEED ZONE sign.

Although these data provide strong support for the relative importance of scene complexity as a determinent of detection, one might still question whether or not the combined predictive validity of target brightness and external contrast might not be as great. Table 4 compares the predictive validity of visual complexity with that of sign brightness and external contrast at 400 and 600 ft. Data for 800 ft were excluded because photometric measurements were generally not possible with such small targets. The 200-ft data were eliminated to maintain balance in the analysis with respect to scenes. Photometric measures were used for brightness and visual measures for contrast, since (as indicated in Table 2) these had the highest validities.

The predictive validity (R^2) of visual complexity (scene and surround) appears to be consistently higher at the farthest distance. These validities also appear stable across sign types, ranging from 0.52 to 0.68 at 400 ft and 0.62 to 0.72 at 600 ft. The validities for brightness and contrast showed greater variability between signs. In all but one instance (SPEED ZONE at 400 ft), visual complexity had a greater validity than brightness and contrast. In general, the differences in validities were greatest at 600 ft, since (with the exception of the DETOUR sign) the validity of visual complexity increased with distance, while the validity of brightness and contrast decreased or remained the same.

Although the magnitude of these R's would suggest that visual complexity is of overwhelming importance to sign recognition, the issues of reliability and generalizability must be considered. Because of the empirical approach taken in this research, the reliability of the regression equations cannot be estimated. Even more important is the fact that complex scenes were overrepresented, which almost certainly accounts for the fact that visual complexity predicted performance better than target brightness and contrast. Additional research seems to be called for to determine the interaction of visual complexity and sign brightness within a sample that includes the full range of visual complexity.

The simple correlations of individual scene and surround variables are potentially useful to understanding the dynamics of visual complexity. Variables whose zero-order correlations with the criterion were significant for at least three of the four target types were as follows:

1. Parked vehicles on right,
2. Type of area (e.g., commercial versus rural),
3. Total area of large bright sources of light in area left of target,
4. Number of traffic signs,
5. Number of bright point sources of light,
6. Number of different (contrasting) surfaces touching target sign,
7. Proportion of perimeter darker than sign, and

Table 3. R^2 for each of four target types by using 20 variables from five groups of measurement categories.

Measurement Category	DTOR	CROS	STOP	SPED
Scene	0.28	0.44	0.49	0.52
Scene + surround (visual complexity)	0.53	0.61	0.57	0.66
Scene + surround + contrast	0.57	0.62	0.60	0.66
Scene + surround + contrast + brightness[a]	0.63	0.66	0.70	0.76
Scene + surround + contrast + brightness[a] + distance	0.70	0.72	0.76	0.76

[a] Brightness was represented by the four photometric measurements of target brightness.

Table 4. R^2 for equations of visual complexity versus brightness (photometric) and contrast (visually measured) at two distances for all signs.

	R^2 for Following Signs and Distances							
	SPED		STOP		CROS		DTOR	
Type of Sign	400 ft	600 ft	400 ft	600 ft	400 ft	600 ft	400 ft	600 ft
Bright + contrast	0.76	0.69	0.43	0.33	0.22	0.23	0.43	0.58
Visual complexity (7 variables)	0.68	0.72	0.52	0.62	0.58	0.69	0.63	0.64

8. Proportion of perimeter equal to sign brightness.

The relation of most of these variables with the criterion was consistent with normal expectations. Two exceptions were total area of large bright objects and number of different surfaces touching target. The number of large bright objects to the left aided detection. This may have occurred because eye fixations were diverted away from the large bright lights and toward the area where the sign was located. Likewise, the number of surfaces touching the target was positively correlated with performance. This finding may be an artifact of the correlation of this variable with distance or it may be a result of the fact that, with more surfaces, there are more chances of high brightness contrast over some part of the sign's perimeter. Further study is necessary to answer these questions.

FIELD STUDY

Both the laboratory and field research efforts of this project were designed to assess traffic sign recognition as a function of both device luminance and visual complexity. The field study represented an attempt to measure the effect of luminance on sign detection in visually different roadway settings under real-world conditions. Two specific hypotheses were tested by the field study. The first of these was that the probability of a driver detecting a traffic sign increases as the luminance of the device increases. The second hypothesis tested was that the probability of a driver detecting a traffic sign increases as the visual complexity of the roadway environment decreases.

In order to draw conclusions about luminance effects on detection in the real world, an attempt was made to collect the field data under conditions that were as naturalistic as possible. Inconspicuous techniques were used to measure traffic performance in response to a controlled treatment condition. The field data-collection procedure involved the unobtrusive measurement of changes in the speed of subject vehicles in response to a diamond-shaped yellow warning sign with the legend SPEED TRAP. To the extent that this procedure minimized experiment-induced sensitivity to the target sign among the subject drivers, the field data should be representative of the behavior of real-world drivers under natural conditions.

Although this approach may have maximized the external validity of the field study results, vehicle speed profiles had to be used as a surrogate measure of sign recognition. Vehicle behavior can be used as an index of recognition only if the recognition of a particular sign consistently stimulates an uninterrupted sequence of driver recognition, decision, and reaction. The SPEED TRAP sign was chosen for use in the field study on the basis of the assumption that drivers who exceed the speed limit are sufficiently motivated to recognize such messages and then decrease vehicle speed.

In spite of its face validity, it was considered necessary to test this assumption by actually demonstrating that speeding drivers do, in fact, slow down after recognizing the SPEED TRAP sign. A pilot study was conducted at two highway sites to determine whether the sign elicited an observable response. The results of the pilot provided three observations. First, speeding drivers did decelerate when the SPEED TRAP sign was deployed. Second, the frequency of decelerations was greater when an array of tape switches (used to record vehicle speeds) was deployed together with an unmarked, shoulder-parked passenger car; and, third, drivers did not respond to the tape switches and car when the sign was not present. These findings were interpreted as indicating that the presence of the car lends credibility to the message of the sign, thereby sufficiently motivating drivers to respond to the device. On the basis of this pilot study, it was concluded that the SPEED TRAP sign could justifiably be used in a methodology that purported to measure vehicle speed profiles as an indication of target sign detection.

Site Selection

The only practical method available to vary visual complexity was by site selection. An attempt was made to select three different highway sites that were as closely matched as possible in terms of both roadway geometrics and the operational traffic situation but systematically different in terms of level of complexity. The effort to match the sites, however, was limited to some extent by requirements imposed by both the instrumentation used for data collection [e.g., traffic evaluator system (TES)] and the experimental methodology itself.

The limitations imposed by the experimental methodology and the requirements of TES deployment reduced the number of candidate sites so that visual complexity played a less-than-optimal role in the selection of sites. The overall strategy employed in the site selection process was to weigh individual site characteristics in terms of apparent relevance to the hypotheses being tested and then to select the three sites that were best matched on the relevant variables.

Apparatus

The apparatus used in the field study consisted of the SPEED TRAP sign, the TES used to record vehicle trajectory data, and the shoulder-parked passenger car that was deployed to lend credibility to the message of the target sign. TES was used to record vehicle speeds at each site. TES (30) is a hardwire system that records momentary closures in electronic circuits that are actuated by wheel-hits on a series of tape switches deployed on the surface of the road throughout the site.

The target sign was a standard 36-in yellow diamond warning sign with a black, nonstandard legend: SPEED TRAP. The sign was displayed on a portable sign mount that was designed to be sturdy, inconspicuous, and similar to the typical standards for shoulder-mounted signs.

At each site, the target sign was positioned in the middle of a 1200-ft course that was instrumented via the TES. TES was used to develop a speed profile that consisted of eight speed measurements, each of which represented the vehicle's average velocity over the 150-ft interval between adjacent TES traps. The first of these traps was located 600 ft upstream of the target sign, and the ninth trap was 600 ft downstream of the target sign. A trap consisted of a parallel pair of tape switches affixed to the road surface perpendicular to traffic flow and spaced precisely 4 ft apart. The basic data that are recorded with this system are the arrival times of an axle over a particular switch. With this output, existing computer programs were used to generate vehicle speed profiles over the measured course and to identify vehicle types on the basis of number of axles and length of the wheelbase. Headways and spot speeds at each trap were also calculated.

Data-Collection Procedure

Data were collected on consecutive weekday nights at each of the three sites. The first night of data collection at each site was used to gather control data, which were used to identify the typical speed pattern through each site--uninfluenced by the SPEED TRAP sign. The only difference between the control and treatment conditions was the absence of the target sign on the control nights. The time of night during which data were collected varied minimally from one night to the next; these times ranged from 6:00 p.m. at the earliest to 2:30 a.m. at the latest. Data collection was limited, however, to hours of full dark; that is, data were never gathered during the twilight conditions of dusk. Finally, each night's data were recorded under clear, dry conditions.

Subjects

The subject sample for the field study was restricted to motorists exceeding the posted speed limit because only these drivers could be expected to have been sufficiently motivated to detect, recognize, and respond to the message of the SPEED TRAP sign. The posted speed limit was 45 mph at the low-complexity site and 40 mph at the other sites. In the State of Pennsylvania, it is a fairly common belief among motorists that speeding citations are almost certainly given to those drivers that are in violation of the posted limit by 6 mph or more. Because of the widespread nature of this belief, the sample was restricted to vehicles whose measured speed over the first 150 ft of the course was more than 6 mph above the limit. Because the subject sample consisted of nighttime motorists that were speeding, young males were probably overrepresented in comparison with the driving population as a whole.

Because decreases in speed that were due to influences other than the SPEED TRAP sign constituted a source of error variance, the subject sample was further limited to vehicles that appeared to be uninfluenced by other vehicles on the road. Specifically, each subject vehicle was required to maintain at least an 8-s clear headway throughout the 1200-ft measured course. A vehicle traveling 55 mph, for example, had to have a headway distance of at least 645 ft. Another extraneous source of speed reductions among stream vehicles may have been activity on either the shoulder or the road itself. Examples of such activity included vehicles poised to merge into the mainstream, pedestrians walking near the edge of the road, and marked police cars passing through the site. These kinds of events were coded manually by members of the field crew during the hours of data collection, and subject vehicles that may have been influenced by such activities were subsequently deleted from the sample.

Because of the greater observation angles and slower performance characteristics associated with larger trucks, the sample was limited to passenger cars, vans, and pick-ups, with no distinctions made among these subgroups. Further, to eliminate those vehicles whose speed-related behaviors may have been influenced by lane-change maneuvers, only those vehicles that did not change lanes within the study site were included.

Independent Variables

The study objectives dictated that both sign luminance and visual complexity be systematically varied as independent variables. Three levels of sign luminance were presented at each of three data-collection sites that differed in terms of visual complexity.

Sign luminance was varied by using three target signs. The high-luminance condition employed a sign made from new type III sheeting (181 cd/footcandle/ft^2). The medium-luminance sign was made from new type II sheeting. The low-luminance sign featured new type II sheeting that was artificially degraded by stretching standard hardware cloth (16 squares/in) across the face of the sign (33 cd/footcandle/ft^2).

Because legibility as well as detection varies as a function of specific luminance, it was considered necessary to estimate the legibility distances associated with each of the luminance conditions (31). Because the legibility distance associated with the high-luminance sign was estimated to be only 21 ft or 7 percent greater than that for the low-luminance device, and because this 21-ft difference in predicted legibility distance comprised only 14 percent of the 150-ft interval separating adjacent TES traps, it was not likely that differences in legibility distance across devices would produce any measured differences in the initial locations of vehicle speed reductions.

The other independent variable--visual complexity--was varied by selecting three matched data-collection sites that differed primarily in terms of the visual environment adjacent to the roadway. The visual complexity of each site was determined by first rating the site on each of the visual dimensions of complexity defined in the laboratory study and then using these coded variable values as input to one of the regression equations.

In general, the procedure used to rate the visual dimensions of complexity was the same as that used in the laboratory study. Specifically, a 35-mm color slide of each site, taken from a location 400 ft upstream of the medium-luminance target sign, was projected onto a 5-ft x 6-ft 8-in glass-beaded screen and evaluated along each complexity dimension by two trained independent evaluators. Notes taken at the site were used to assist this rating procedure. Disagreements in initial ratings were resolved by a discussion of the rationales used by each of the evaluators in arriving at their respective ratings.

After the complexity variables were rated, the level-of-complexity characteristic of each site was determined by using the coded variable values as input to the regression equation developed for the yellow diamond PEDESTRIAN CROSSING sign via the laboratory study.

Experimental Design

The field study was designed to provide information relevant to specific hypotheses that stated that both the probability and distance of sign detection increase as sign luminance increases and as visual complexity decreases. In addition, this design also allowed for an exploration of the potential interactive affects of luminance and complexity on both the probability and location of sign recognition. These hypotheses were examined by recording data that indicated the incidence and location of vehicle speed reductions elicited by the SPEED TRAP sign. The percentage and location of these vehicle decelerations were assumed to be indicative of the probability and location of target sign recognitions, respectively.

The level of visual complexity was controlled by selecting three highway sites that were matched to the extent possible except with regard to the nature of the adjacent visual environment, which differed systematically to provide low-, medium-, and high-complexity conditions. As such, visual complexity was a between-site variable with only one site representing each of three levels of complexity. Because this design used only one instance of each

Table 5. Sign effects within complexity levels for lane 1.

Complexity	Sign Present?	Speed Reduction? Yes	No	Yes (%)	Chi-Square
Low	Yes	136	230	37.2	
	No	55	142	27.9	
	Total	191	372		4.88[a]
Medium	Yes	181	204	47.7	
	No	81	128	38.8	
	Total	262	332		4.40[a]
High	Yes	118	58	67.0	
	No	96	38	71.6	
	Total	214	96		0.75[b]

[a]Significant. [b]Not significant.

Table 6. Brightness effects within complexity levels.

Lane	Complexity	Percentage Speed Reduction at Following Brightness Levels: Low	Medium	High	Chi-Square (low versus high)
1	Low	37.2	35.6	38.3	0.03[a]
	Medium	42.2	45.0	54.5	3.88[b]
	High	73.1	68.8	64.0	0.74[a]
2	Low	15.9	13.9	18.2	0.27[a]
	Medium	40.8	40.7	47.0	1.38[a]
	High	63.5	64.6	57.6	0.81[a]

[a]Not significant. [b]Significant.

complexity level to assess the effect of complexity, any other between-site differences may have confounded the influence of complexity. For this reason, any conclusions about this independent variable should be interpreted cautiously.

The level of sign luminance, however, was a within-site variable. Each of three luminance conditions--low, medium, and high--was presented at each site. The target sign was deployed on two nights per site, and the three luminance conditions were balanced across nights to preclude confounding with day of the week. Data were collected for approximately 6 h each night, and an attempt was made to divide this period into two or more equal blocks of time. Each of the three luminance conditions was presented for equal intervals of time within each of these blocks, and the order of presentation of the three levels of luminance was counterbalanced across blocks to avoid confounding luminance effects with time of night.

Findings

The dependent variable--incidence of speed reductions--was derived by evaluating the speed profile of each subject vehicle as it traversed the 1200-ft measured course. Eight interval speeds, obtained from nine TES traps, comprised the raw data that were used for the dependent measure.

The major area of inquiry was the effect of the SPEED TRAP sign on the incidence of speed reductions. For this purpose, a number of ways of measuring speed change was considered. The selected measure was the exit speed (the average speed between traps eight and nine) minus the entry speed (the average speed between traps one and two). This measure had the advantages of simplicity and ease of interpretation. In addition, it was preferred over shorter-term measures because it tended to avoid unimportant speed fluctuations and concentrate on those speed changes that were more likely to reflect driver intentions.

An initial analysis was done simply to determine if the presence of the SPEED TRAP sign was conducive to speed reductions. In this analysis, scene complexity and sign brightness were ignored; that is, the data were collapsed (i.e., summed) over the levels of complexity and brightness. In this analysis, treatment data (data collected with the sign present) were compared with control data (those collected with the sign removed) so that the focus was on the effect of the sign itself.

The results of this analysis failed to show statistically significant sign effects for either lane. That is, while the proportion of vehicles that reduced speed was higher when the SPEED TRAP sign was in place, the magnitude of the increase was not sufficiently large to preclude the results being due to chance alone.

However, a second analysis (summarized in Table 5), in which the sign effect was examined within data-collection sites (i.e., within levels of complexity), shows that for lane 1 the sign had an observable effect at the low- and medium-complexity sites. In both instances, the sign had the effect of increasing the relative frequency of speed reductions by 9 percent. The likelihood of a speed reduction was increased by 33 percent at the low-complexity site and by 23 percent at the medium-complexity site.

None of the results for lane 2 traffic showed statistically significant findings. Any explanations of this are speculative at best. It may have been due to the greater viewing angle or a difference in the nature or motivations of the drivers who chose to travel in lane 2.

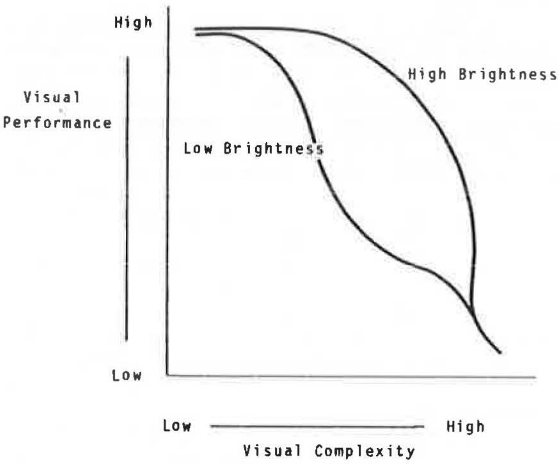
The findings for lane 1, however, suggest that there may have been an effect of visual complexity; that is, that the sign became less effective as scene complexity increased. This is indicated by the greater likelihood of speed reductions associated with the presence of the sign at the low-complexity site and the absence of such an effect at the high-complexity site.

The final analysis was to determine if sign brightness influenced the likelihood of speed reductions. Specifically, Did brighter signs elicit more frequent speed reductions? The results are given in Table 6, where the effect of sign brightness is examined within each complexity level.

As noted earlier, there were no significant effects for the lane 2 data. For lane 1, only the medium-complexity data reflected a statistically significant sign brightness effect. (The test statistics resulted from a comparison of the high- versus the low-brightness sign; while this procedure failed to use all the available information, it allowed the testing of one-sided hypotheses, thereby reflecting the ordinal nature of the levels of sign brightness.)

Once again, plausible, but speculative, explanations for the lane 1 findings are offered in the following. First, at the low-complexity site, the low-brightness sign may have been sufficiently detectable that increased brightness was superfluous; hence, increased brightness would not be expected to yield beneficial effects. The medium-complexity site may have fallen in a range of reduced sign conspicuity where sign detection was sensitive to sign brightness in the expected way. Finally, the high-complexity site differed from the other two in a unique way. On-site observations by the experimenters revealed that the sign was markedly darker than its surround. As a result, increasing sign brightness resulted in a reduction of sign-surround contrast. This, in turn, could well have led to the

Figure 2. Theoretical relations suggested by field study.



observed, though statistically insignificant, reduction in driver responsiveness to increases in sign brightness, as shown in Table 6.

In summary, lane 2 showed no sign, complexity, or brightness effects. For lane 1, a sign effect was found at the low- and medium-complexity sites but not at the high-complexity site; this, in turn, suggested the existence of a complexity effect. Sign brightness was found to influence driver responsiveness only at the medium-complexity site. This was consistent with reasonable expectations based on level of complexity and sign-surround contrast considerations.

SUMMARY

The two studies reported build a strong case for the usefulness of visual complexity as a predictor of sign detection and recognition. The results indicate that, at locations within complex visual scenes, measures of the scene and the sign's surround predict visual performance better than sign contrast and brightness. The laboratory study indicated that target brightness could offset the detrimental effects of visual complexity. Although the field study supported this finding, it also suggested that brightness might not have an effect at the extremes of visual complexity. The results of the field study suggested the theoretical relations shown in Figure 2. When visual complexity is low, performance is asymptotic and any reasonably reflective sign will be recognized. When visual complexity is extremely high, performance is likely to be equally poor with all retroreflective signs, and sign redundancy or internal illumination may be needed.

The measures of visual complexity, sign brightness, and contrast should provide a useful contribution to future research. Methods for measuring contrast, surround, and other variables have not previously been well defined for complex stimuli. The photometric methods for measuring internal contrast should also be of benefit. The fact that visual measures can have significant validity and be reliably coded suggests the potential for a practical method whereby field personnel can judge the visual complexity of a location and its effect on driver recognition of traffic signs.

The absence of cross validation and the inclusion of only three sites in the field study places obvious reservations on the reliability of the findings. Nevertheless, the consistency and pattern of results suggests that continuation of this research seems warranted. Given the abstract nature of most previous research, this report represents an important step toward developing practical and usable results. Additional work is now needed to refine the measures of visual complexity, formulate and test hypotheses about their interrelations, and develop a scale and procedure that makes visual complexity easier to measure.

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Assessing the Built Environment for Pedestrians Through Behavior Circuits

C.J. KHISTY

Planners are generally concerned with those social and physical attributes that are distributed in time and space. These attributes typically occur in independent clusters or behavioral settings that vary in scale from an apartment complex to a large urbanized region. Within these settings, attributes can normally be analyzed in terms of identifiable and recurrent elements, patterns, and sequences. If one divides human behaviors by their scale and generality, it will be noticed that the things people do at the widest compass can be called behavior streams or activities. These, in turn, can be further separated into behavior circuits, which are differentiated by specific purpose. A systems view of the behavioral science/transportation framework is first described. The paper then examines the use of behavior settings and behavior circuits in pedestrian planning, designing, and the development of performance standards. Relative to a set of needs and purposes, certain aspects of environmental form typically support or constrain desired human action and communicate meaning and value.

Assessing the built environment from the standpoint of safety is an interdisciplinary inquiry that embraces the applied social and behavioral sciences. This assessment depends on the development of fundamental knowledge of the interaction of man-made physical environment variables with other environmental variables in influencing behavior.

This paper examines the use of behavior settings and behavior circuits in assessing the built environment, particularly the infrastructure built for the transportation of people. The outcome of this investigation can be used productively in assessing either the existing built environment or for planning future facilities. Although this paper focuses on assessing pedestrian planning and safety vis-a-vis the built environment, the techniques described here can be extended to other modes of transportation.

BACKGROUND

With the introduction of the transportation system management (TSM) element (1) in urban transportation planning, there is widespread interest among engineers and planners to improve existing pedestrian facilities and to plan new ones. In this and former efforts there has been a persistent tendency to imitate the classic planning and designing procedures adopted by planners for highway facilities. This has been unfortunate. The current predicament is in part the consequence of a gross underestimation of the complexities of human perception and mental need. All of the planning tools and procedures may be impeccable, but if the physical consequences--the actual objects in space--do not add up to a satisfying, vigorous, and safe environment, the total effort is of little consequence.

In recent years, many designers and planners have formulated new microtheories of the environment in an attempt to plan cities. Lynch (2), in his Image of the City, takes a cognitive approach to the environment in his attempt to get to the visual quality of the American city. Alexander (3) argues that only by tracing the functional requirements of human needs and activities can one provide solutions to problems of the built environment. Carr (4) speculates about criteria for environmental forms in his City of the Mind. Perin (5) separates the activities of human beings into units called behavior circuits. Barker (6) attempts to determine the relations between what he calls the "extra-individual" patterns of behavior, i.e., the behavior that people reveal in a behavior setting. His definition of a behavior setting is an environment that is bounded in space and time and has a structure that interrelates physical, social, and cultural properties that elicite common or regularized forms of behavior. Barker's behavior settings in time, space, and place involve far more than just the physical setting.

These microtheories not only provide an additional research perspective in regard to city planning and design but also provide the techniques for analyzing and solving problems that occur in transportation planning, particularly for pedestrians. The use of behavior settings and behavior circuits will be examined still further in this paper.

BEHAVIORAL SCIENCE/TRANSPORTATION SYSTEM FRAMEWORK

The systems view is an attitude of mind in facing complexity; it reflects a search for the interrelatedness of things in any problematic situation. As a planning tool it means approaching the transportation system as a complex whole within which many elements act interdependently. Several researchers have made contributions in understanding the behavioral science/transportation connection. Parsons' efforts, in particular, have set the stage for a good deal of interaction between behavioral scientists and transportation engineers (7-11).

To start with, a systems view of the behavioral science/transportation framework is useful. Transportation engineering and planning is involved with a diversity of basic activities performed by transportation specialists, such as policymaking, operations and safety, and testing and evaluations. Figure 1 illustrates these activities in the context of transportation modes. Although the use and distribution of transportation by mode is a continuing source of controversy, one can recognize nine major categories, including several fringe and developing groups in transportation (12). Parsons has identified nine categories of human behavior that are affected by transportation. These categories are given below (11):

1. Locomotion (passengers, pedestrians);
2. Activities (e.g., vehicle control, maintenance, community life);
3. Feelings (e.g., comfort, convenience, enjoyment, stress, likes, dislikes);
4. Manipulation (e.g., modal choice, route selection, vehicle purchase);
5. Health and safety (e.g., accidents, disabilities, fatigue);
6. Social interaction (e.g., privacy, territoriality, conflict, imitation);
7. Motivation (positive or aversive consequences, potentiation);
8. Learning (e.g., operator training, driver education, merchandising); and
9. Perception (e.g., images, mapping, sensory thresholds).

Similarly, there are at least 11 properties of the physical environment that have a direct impact on human behavior. Details of these environments are provided below (11,13):

Figure 1. Transportation as a system.

	POLICY MAKING	ADMIN-MANAGEMENT	PLANNING	ANALYSIS, SYNTHESIS AND DESIGN	CONSTRUCTION	OPERATIONS	MAINTENANCE	TESTING AND EVALUATION
AIRWAYS								
CONVEYORS								
HIGHWAYS								
PIPELINES								
RAILWAYS								
WATERWAYS								
MULTI-MODAL								
EXOTIC								
QUASI-TRANSPORT								

1. Spatial organization: This dimension often includes the shape, scale, definition, bounding surfaces, internal organization of objects and people, and connections to other spaces and settings. Indeed, this is the dimension most people are referring to when they talk about the physical environment. The degree of dispersion, concentration, clustering, and proximity of facilities is also included.
2. Circulation and movement: This property includes people, goods, and objects used for their movement--cars, trains, highways, and rails--and also the forms of regulating them such as corridors, portals, turnstyles, and open spaces.
3. Communication: Both explicit and implicit signals, signs or symbols communication, required behavior, responses, and meanings are covered by this dimension; in essence, these are the properties of the environment that give users information and ideas.
4. Ambience: This dimension usually includes such items as microclimate, light, sound, and odor. Those features of the environment that are critical for maintaining the physiological and psychological functioning of the human organism are included.
5. Visual properties: The environment as it is perceived by its users is generally implied by this property and includes color, shape, and other visual modalities.
6. Resources: The physical components and amenities of a transportation system--paths, terminals, and vehicles--could be included. The measures of these resources could embrace such dimensions as the number of lanes or the square footage of the terminals.
7. Symbolic properties: The social values, attitudes, and cultural norms that are represented or expressed by the environment fall into this category.
8. Architectronic properties: This refers to the sensory or aesthetic properties of the environment.
9. Consequation: This is that characteristic of the environment that strengthens or weakens behavior. Measures of consequation may be items such as costs, risks, and congestion.
10. Protection: Safety factors in general are implied in this category.
11. Timing: All the items mentioned above are scheduled in time and some of them fluctuate with various cyclical rhythms such as daily, weekly, or hourly timings.

Figure 2 illustrates the impact of the environment

Figure 2. Relation between aspects of transportation and their effects on people.

Environmental Aspects \ Human Behavior	Activities	Locomotion	Social Interaction	Feelings	Perception	Motivation	Health and Safety	Learning	Manipulation
Spatial Organization		X	X					X	
Circulation and Movements	X	X	X				X	X	
Communication	X	X	X					X	
Ambience	X			X		X	X		X
Visual Properties					X				X
Resources	X			X			X	X	X
Symbolic Properties	X	X			X			X	X
Architectural Properties	X	X	X		X		X	X	X
Consequation		X				X		X	X
Protection							X	X	
Timing		X	X						

on aspects of human behavior relevant to transportation. Further, individual differences among the population that use and provide transportation should also be considered. These dimensions include age, ethnicity, income, car ownership, economic status, health, and skills. Also, some of the basic ingredients that are embraced by transportation design are as follows (14): safety, security, convenience, continuity, comfort, system coherence, and attractiveness. The variables contained in these lists and figures may appear overwhelming, but they do set forth a systems approach to the interconnection between transportation and human behavior.

BEHAVIOR SETTINGS

The behavior-setting survey technique of Barker (6) encompasses several advantages for design practice. The method assumes that the physical environment and behavior are inextricably bound together. Bechtel (15) has applied Barker's methods to practical problems of house design, community planning, and organizational design. He interprets Barker's definition of a behavior setting as follows:

1. A behavior setting is a standing pattern of behavior that occurs over and over again in a given place and at a given time. You can go to the place where it occurs at the time it occurs and see the behavior repeated each time the setting happens.
2. Yet behavior settings, even though they are defined as separate entities, are a part of the flow of behavior in a community. People move in and out of settings, but the settings do not disappear when different people arrive; they have a life of their own. Yet when the community changes, settings change also.

The behavior-setting survey, which is the basis of the analysis, can take as long as a year to do. Various scales are used to quantify behavior within and across settings. These scales provide no fewer than 63 separate bits of information about each setting (15):

> When completed, the behavior-setting survey data are the raw material around which the designer can give form to his structures. The survey can be tapped for information about a room, a building, streets, and sidewalks, or any other aspect of the community in part or in whole. The behavior-setting survey is a complete catalog of behavior indexed to locations, times, frequencies, populations, age groups, intensities, and a complex of other details. Its use is not easy to master but it provides the only known comprehensive way to master design elements of behavior.

The chief advantage of Barker's technique is its directness in collecting valid data. It measures what people do with design features, not what they say they do.

BEHAVIOR CIRCUITS AND TRACKING

The behavior circuit is a unit of analysis or a unit of behavior that can be observed, recorded, and compared. Some behavior circuits may be similar in the actions composing them but have different outcomes. The behavior circuit is thus a unit of analysis that permits combinations of concepts in different areas of social and behavioral sciences to be operational. Perin (5) describes this concept as follows:

> What behavior circuit implies is an anthropological ergonomics, tracking people's behavior through fulfillment of their everyday purposes at the scale of the room, the house, the block, the neighborhood, the city, in order to learn what resources--physical and human--are needed to support, facilitate, or enable them.

Much of the data obtained from behavior-circuit analysis will undoubtedly be untidy, overlapping, and conflicting, and in many cases it will be quite a challenge to create order out of this confusion. By the same token, these data will reveal realities previously unrecognized and, in the long haul, save a lot of time in collecting data regarding attitudes, opinions, values, and preferences on a piecemeal basis.

Intimately connected with behavior circuits is the gathering of data through tracking. Tracking is the systematic following of a pedestrian and the recording of his or her movements. Patterns of pedestrian activity are derived from tracking a large number of subjects. It is not necessary to question the subject or for the tracker to take time explaining his or her activities. Instead, he or she can work expeditiously, making a large number of observations that can be translated into patterns of movement (16). One can, of course, employ direct communication with a sample set of pedestrians or conduct experiments of role play to gain information on the environmental requirements necessary to support this behavior. It may be realized that the mere observation of behavior only provides information on what people are now doing, not on what they want to do, which is often the more important question.

DIMENSIONS OF BEHAVIOR CIRCUITS

All environments have both social and physical attributes. Planners and engineers are generally concerned with those attributes that are distributed in time and space and are at least controllable. These attributes typically occur in independent clusters, behavior settings, or regions that vary in scale from apartment sidewalks to large urbanized regions. Within these settings attributes can normally be analyzed in terms of identifiable and recurrent elements, patterns, and sequences. Psychological and behavioral significances can only be assessed relative to the general environmental context and the forms of man-environment interactions that occur within that context. Relative to a set of needs and purposes, certain aspects of environmental form typically support or constrain desired human action and communicate meaning and value. These aspects can usually be described in terms of the above-mentioned selected 11 properties of the physical environment that have an impact on human behavior.

DESIGN CRITERIA AND PERFORMANCE STANDARDS

The design of pedestrian facilities involves the application of traffic-engineering principles combined with consideration of human convenience and the design environment. Different environments logically require the application of different qualitative as well as quantitative design standards. Failure to take into consideration the sociocultural requirements of the users of the built environment accounts for many of the problems that people experience in dealing with the man-made world.

Design and performance criteria are the preferred means of control where they can be capably administered. Controls are based on standards, which are formal statements about the stable characteristics of environmental features that are presumed to make them universally desirable or acceptable. Standards are a necessity in order to simplify the large and shifting body of information about performance so that decisions are not lost in detail and uncertainty (17).

The design of pedestrian spaces involves not only the application of basic traffic-flow principles but also the consideration of human convenience and the design environment. For example, in the case of pedestrian movement, one could apply the level-of-service concept elaborated by Fruin (14), along with the concepts of task efficiency, reduction of stress, comfort, and safety. However, no quantitative tool by itself can substitute for good judgment.

It is important to find out what characteristics the environment ought to have to support pedestrians at one of the following levels:

1. The survival level,
2. The efficiency level,
3. The comfort level, or
4. The pleasure and enjoyment level.

These levels are, of course, arbitrary and are stated this way only for purposes of illustration. In a way, each type of behavior circuit suggests its own performance standard or criteria for design. Perin classifies behavior circuits as routines "when they recur so often as to have a regularized sequence that the person carries out relatively unconsciously and more or less independently of others" (5). A young person taking a casual stroll in the local park might fall into this category. "Behavior circuits are collaborations when the actions composing them recur frequently, but unlike routines, go beyond the compass of the self to require other persons or equipment for carrying them out" (5). Pushing a child in a stroller on the sidewalk of a busy street, or for an elderly person to operate the "walk" pushbutton at an intersection, could be considered a collaboration. "Behavior circuits are events when the maintenance of various kinds of group relations occur, at any level of frequency" (5). Weaving one's way through a large gathering of people in a shopping mall might be classified as an event, particularly if one is totally disoriented because of ambience. Perin also mentions a residual behavior circuit called "emergencies". One could interpret this category with those situations where the propensity for accidents and of insecurity is obviously high, for example, where the safety requirements influence the form of the structure. There is need for caution to be exercised in applying this classification. What is routine for the 20-year-old may be an event for an octogenarian. At the same time, a close look at all the behavior circuits and their categorization into routines, collaborations, events, and emergencies may result in the planner having to spend time only analyzing the events and emergencies. Perin (5) observes two important consequences as a result of using behavior circuits. First, the observer is alerted "to differences in age, residential location, education attainment, ethnicity, income, car-ownership and so on, casting the imperative for differing environmental resources in kind, interval, density, and so on" (5). Second, the relation between numbers of people and the amount of space needed to accommodate them is brought out strongly. Connected with this relation is the time dimension of behavior circuits. The introduction of time to control the level of use of a facility becomes possible only when the patterns and density of use are known. The time element naturally brings into focus the idea of movement. The time it takes to carry out any kind of behavior circuit is included implicitly as a performance standard.

SECURITY AND SAFETY

Even though the general public today is aware of the benefits of the walking mode, there is great concern regarding the increasing crime rate in large metropolitan areas, particularly for the pedestrian. People's beliefs about security help determine whether they will be pedestrians at all, and how, where, and when they will walk if they decide to do so.

The basics of "defensible space" developed by Newman (18) demonstrate that design features of the built environment can influence crime rates. Defensible spaces avoid features that convey stigma, vulnerability, or isolation. The concept of defensible space depends on a community's sense of territoriality, where the arrangements of pathways, sidewalks, and buildings foster a sense of control and cohesiveness among people living in the community. Second, defensible space requires surveillance through proper lighting and visibility. Security can be enhanced by providing facilities with a safe image. Third, nonstigmatizing design forms provide the basis for defensible space. Newman's work is most applicable to pedestrian safety and security. From the standpoint of behavior circuits, insecurity and accident-prone settings would fall under the category of emergencies. The use of behavior circuits in assessing the built environment for security and safety is particularly valid.

PLANNING PROCEDURES

In ex-post-facto studies of the built environment, one can always examine specific areas that reveal trouble spots through tracking, interviewing, or behavior settings and circuits. Pedestrian volume and density changes, shifts in user attitudes, or recent additions in pedestrian generators may have to be carefully reviewed. The results of demonstration projects of pedestrian facilities can provide excellent insights into problems likely to be faced in new or modified projects. It may also be possible to transfer knowledge gleaned from one situation to another. Some items of pedestrian design problems appear to repeat themselves more often than others, such as the problem of negotiating flights of steps, ramps, and slippery pavements; the hazards of cars turning right on red; street furniture obstructing pedestrians; lack of refuge islands on wide urban streets; and pedestrian facilities used predominantly by children and/or the elderly. It is these problems that ought to be looked into critically while new facilities are being designed, although they may appear to fall in the category of routines and collaborations. However, in general, one would look at events and emergencies very carefully. The use of behavior circuits will enable the planner to identify probable or actual crime problems. The ideal strategy, of course, is to assemble packages of countermeasures that complement each other and mutually permit the achievement of multiple goals. Some security design goals are providing adequate surveillance of pedestrian facilities, controlling access and egress, minimizing exposure time in crime-ridden areas, ensuring adequate communication, and enhancing perceived security.

Lynch (17) feels that, by describing behavior circuits in terms of behavior settings, the intended outcome is obvious. "It is linked to the restrictions and potentials of individual situations, the requirements of the users, the general objectives and the potentialities of future form. It is both a statement of detailed criteria and the essence of the design. It can also be the basis of collaboration between the behavioral scientist and designer because it is based on behavioral knowledge" (17). He further goes on to say that "the requirements of settings may be stated as required thresholds. Since different groups of people will be taking different actions for different purposes in any given situation, program requirements specify for whom the settings are intended and how predicted conflicts are to be handled. Programs of this kind not only link objectives to specifications but allow proposed designs to be evaluated by the way they fulfill explicit requirements. Once built, the environment can be monitored to see if it is performing as predicted" (17). The acid test, of course, is the way in which a plan for pedestrians supports purposeful behavior.

A unified set of procedures for incorporating security features for the pedestrian in the overall planning process is necessary. It is both easier and cheaper to include such security measures in the initial design phases. Richards and Hoel have suggested a plan for mass transit station security (19), and a modified form of their procedure is given below:

Step 1: Assess initial situation;
Step 2: Use behavior setting and behavior circuits through interviews, questionnaires, and observations;
Step 3: Anticipate and document safety problems;
Step 4: Establish safety and security goals;
Step 5: Select possible countermeasures;
Step 6: Evaluate possible countermeasures;
Step 7: Consider limits and constraints;
Step 8: Consider trade-offs;
Step 9: Establish countermeasure strategy; and
Step 10: Evaluate overall project after implementation.

Some of the basic characteristics that could contribute to safety problems can be identified through a variety of ways. Several ways of assessing the current situation are given below:

1. Existing facility: (a) design features; (b) neighborhood characteristics; (c) functional requirements; (d) crime statistics; (e) expert input by police, community leaders, behavior circuits, and behavior settings; (f) interviews with users; and (g) incident reports.

2. Planned facility: (a) design features, (b) neighborhood characteristics, (c) functional requirements, (d) crime statistics, and (e) input from potential users, community leaders, police, business people, behavior settings, and behavior circuits.

CONCLUSION

This paper examines the use of behavior settings and behavior circuits in assessing the built environment for one mode of transportation--the pedestrian mode. It provides a brief introduction to current microtheories of the environment, describes the use of behavior circuits and behavior settings, and comes up with possible strategies for their use in planning and safety programs.

Many disciplines have helped the engineer and planner to assess environmental quality. Difficulties arise in research of this nature because of its interdisciplinary content.

The use of behavior circuits in pedestrian planning and safety is comparatively new. It symbolizes the growing interaction of sociologists, psychologists, and planners to understand and resolve pedestrian problems. This is an instance where the use of behavior circuits gets the planner away from the usual circular reasoning to critically look at the whole complex of behaviors associated with a pedestrian. The use of behavior circuits in pedestrian planning enriches one's understanding of human behavior and enables the planner to get a holistic view of the problem.

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Pedestrian Accidents on Rural Highways

J. W. HALL

Pedestrian accident experience in two New Mexico counties is far in excess of statistically expected levels. These accidents, which occur principally on rural roads, result in a fatality more than 60 percent of the time. This research was undertaken to determine if engineering improvements could reduce the frequency of these accidents. Field studies were conducted at 95 rural pedestrian accident sites in these two counties. It was found that roadway geometrics at these locations were good and that sight distance exceeded standard requirements. Fixed objects, parked vehicles, and other features that might conceal a pedestrian along the roadside were noticeably absent. At the same time, pedestrian safety devices and amenities were employed infrequently at the accident sites. Most locations had adequate right-of-way width to accommodate separate sidewalks or paths for pedestrians. Although more than 80 percent of the accidents occurred at night, roadway lighting is a viable improvement at only a limited number of the rural locations. Two cluster areas of pedestrian accidents may warrant the most immediate attention. The inability of engineering features or deficiencies to explain a significant amount of the variation in the characteristics associated with these pedestrian accidents suggests that other, nonengineering factors may play a more important role in their occurrence.

One of the tasks of highway and traffic engineers is to provide for the safe movement of pedestrians. In several regards, the pedestrian is at a considerable disadvantage in the traffic system. However, engineers attempt to accommodate pedestrians through the provision of special facilities, such as sidewalks and crosswalks, and the use of other features, such as street lighting, which may be of value to both pedestrians and motorists. Because pedestrian activity occurs predominantly in urban areas, attention has traditionally focused on these areas.

PEDESTRIAN ACCIDENT STATISTICS

The problem of pedestrian-vehicle interaction is suggested by nationwide accident statistics. During the past decade, pedestrian fatalities in the United States have averaged 9400/year. Although pedestrians are involved in less than 1 percent of all accidents, they account for 18 percent of the highway fatalities. In 1980, New Mexico accounted for 1.29 percent of the nationwide pedestrian fatalities and 1.16 percent of the nationwide nonpedestrian highway fatalities. Table 1 compares nationwide characteristics of pedestrian accidents with those in New Mexico. As shown in the table, the fatal pedestrian rates based on population, registered vehicles, and vehicle miles of travel are all about twice the national rates. It is also noteworthy that nearly half of New Mexico's fatal pedestrian accidents occur in rural areas versus one-third nationwide.

Data from the 1980 Fatal Accident Record System (FARS) suggest that pedestrians account for a larger share of highway fatalities in the more urbanized states. Analysis of nationwide data shows a highly significant correlation between population density and the pedestrian proportion of highway fatalities. The 10 states with population densities greater than 200 persons/mile^2 report that 20 percent of their highway fatalities involve pedestrians, while in the 10 states with densities less than 20 persons/mile^2, the corresponding value is 12 percent. In New Mexico, where the density is 11 persons/mile^2, pedestrians constitute more than 19 percent of the highway fatalities.

Table 1. Pedestrian accident characteristics, 1980.

Item	United States	New Mexico
Pedestrian accidents	130 000[a]	630
Pedestrian fatalities	8180	106
Fatality index	0.06	0.17
Rural fatality (%)	34	49
Travel-based rates		
Accidents per 100 million vehicle miles	8.52	5.64
Fatalities per 100 million vehicle miles	0.54	0.95
Population-based rates		
Accidents per 100 000 persons	58.6	48.8
Fatalities per 100 000 persons	3.7	8.2
Mileage-based rates		
Fatalities per 1000 miles	2.1	1.5
Urban fatalities per 1000 miles	7.9	9.9
Rural fatalities per 1000 miles	0.9	0.8
Registration-based rate, fatalities per 100 000 vehicles	5.3	10.3

Note: Data in this table are compiled from the following sources: Federal Highway Administration; Fatal Accident Record System, National Highway Traffic Safety Administration, U.S. Department of Transportation; New Mexico accident record system; National Safety Council; and National Traffic Safety Bureau.
[a]Estimate. The accuracy of this number, and the rates based on it, are subject to debate.

These statistics suggest that the pedestrian accident experience in New Mexico differs somewhat from that of other rural states. To further examine this issue, the state's computerized accident records for 1978-1980 were evaluated. Of the 156 000 accidents during this period, 2090 (1.3 percent) involved a pedestrian, and in 96 percent of these, impact with a pedestrian was cited as the first harmful event. Of these accidents, 16 percent resulted in a fatality. An examination of accidents by county found that, with two exceptions, the reported percentage of pedestrian accidents was in good agreement with the proportionate share of all traffic accidents.

The two exceptions were McKinley and San Juan counties, which are located adjacent to each other

Table 2. Comparison of pedestrian accident characteristics.

Characteristic	San Juan/ McKinley (%)	Rest of New Mexico (%)
Nonintersection	80	70
Intersection	12	13
Intersection related	8	17
Fatal	26	14
Nonfatal injury	70	83
Daylight	37	56
Dark		
Lighted	25	20
Not lighted	38	19
Straight	93	95
Curve	7	5
Level	85	88
Grade, crest	15	12
Had been drinking	38	12
No drinking	29	66

Table 3. Selected characteristics of McKinley and San Juan counties.

Characteristic	McKinley	San Juan	New Mexico
Population	54 950[a]	80 833[b]	1 291 000
Area ($mile^2$)	5454[c]	5500[c]	121 412
Population density			
Total	10.1	14.7	10.6
Rural areas	6.3	5.9	3.6
Ethnic distribution (%)			
Anglo	23	52	52
Indian	61	35	7
Hispanic	14	12	38
Median age	19.2	20.7	23.9
Death rate per 1000 population	8.1	6.6	7.3
Median family income ($)	6780	8150	7850
Indian-owned land (%)	62	60	9

[a]4.3 percent of total population.
[b]6.3 percent of total population.
[c]4.5 percent of total area.

in the northwest corner of New Mexico. These predominantly rural counties account for 12.7 percent of all traffic accidents, while they report 21 percent of the pedestrian accidents. An even more disturbing statistic is that these counties report 33 percent of the fatal pedestrian accidents--a value almost three times as great as their share of all accidents. The pedestrian accident fatality index in these counties is 0.26, which is twice the value for the rest of New Mexico.

A comparison of selected pedestrian accident characteristics between these two counties and the remainder of New Mexico is summarized in Table 2. As the table shows, pedestrian accidents in these counties are more often characterized by nonintersection locations, the hours of darkness, and alcohol involvement. They occur slightly more frequently on curves and nonlevel roads.

Of the 393 pedestrian accidents in these two counties during a 30 month period, 144 (37 percent) occurred in rural areas. However, 82 (79 percent) of the 104 pedestrian fatalities were on rural roads. A major subset of the rural road system, which consists of those roads administered by the New Mexico State Highway Department, accounted for 122 pedestrian accidents and 74 fatalities.

Statistics clearly indicate that these two counties are responsible for a disproportionate share of the pedestrian accidents. The objective of this research was to determine, through an examination of rural pedestrian accident sites in these counties, if standard or specialized forms of remedial action could be implemented on the highways to reduce the potential for these types of accidents (1).

Selected characteristics of the two counties are summarized in Table 3. The counties account for 9 percent of the state's land area and nearly 11 percent of the population. The population density (persons per square mile) is 70 percent greater than the statewide average for rural areas. The counties both have a substantial Indian population, with McKinley and San Juan ranking first and third, respectively, in the percentage of Indians in the population. The median ages in these counties are the two lowest in the state. Because any of these distinguishing characteristics could influence the occurrence of pedestrian accidents, the results of this study may have limited applicability to other rural highways.

STUDY PROCEDURE

In view of the serious problem of pedestrian accidents in these counties, a plan was developed to examine the sites of 95 pedestrian accidents that occurred during a 30-month period on the 725 miles of rural, state-administered, non-Interstate roads in the counties. Of these accidents, 63 (66 percent) resulted in a fatality.

The initial procedures for data collection were predicted on the findings of previous studies of urban pedestrian accidents. However, there are obvious differences between the pedestrian problems in urban and rural areas. As a general rule, urban areas are characterized by lower speeds, greater traffic density, more children interacting with traffic, and more roadside distractions. Urban areas also make much more extensive use of special pedestrian facilities such as sidewalks, crosswalks, WALK/WAIT signals, and refuge islands. The principal emphasis for urban pedestrian facilities is at intersections, although two-thirds of the pedestrian accidents occur at nonintersection locations.

The factors that appear to be of primary importance in a site examination of rural pedestrian accidents include visibility, presence of special pedestrian facilities, and potential for countermeasure application. These factors are reflected by such items as alignment, illumination, presence of roadside objects, and application of traffic-control devices. A field form for obtaining these and related items was developed and tested in spring 1981. The actual data collection was accomplished by a two-person field crew who used simple engineering measurement devices.

DATA ANALYSIS

The mean, minimum, and maximum values of numeric characteristics evaluated for all pedestrian accidents are summarized in Table 4. The mean values of curvature are slightly, but not significantly, to the right. However, 85 percent of the pedestrian accidents occurred at sites where the degree of roadway curvature was less than 1°. The average gradient at the pedestrian sites is negative and is on the borderline of being statistically significantly less than zero. With respect to the general nature of the vertical alignment in the vicinity (0.25 mile) of the accident site, 40 percent were described as downgrade while 30 percent each were described as level or upgrade. As expected, superelevation averages were positive, where the sign convention employed the positive designation for normal crown sections on tangents and proper banking on curves.

The roadways averaged 35 ft in width, with a 9-ft shoulder on the side of the roadway on which the pedestrian was struck. The mean roadway width is somewhat misleading because the study sites were virtually all on two- or four-lane roads, with the former being slightly more common than the latter.

Table 4. Selected characteristics at accident sites.

Characteristic	Mean	Minimum	Maximum
Curvature[a]			
In advance of accident site			
150 ft	-0.04	-15.11	4.81
50 ft	-0.26	-30.26	2.75
Avg	-0.15	-22.68	3.69
Absolute[b]	0.55	0	22.68
Gradient			
In advance of accident site			
150 ft	-0.35	-5.80	4.80
50 ft	-0.19	-6.10	5.00
Avg	-0.27	-5.95	4.90
Superelevation			
In advance of accident site			
150 ft	1.70	-1.90	6.40
50 ft	1.67	-1.90	6.10
Avg	1.68	-1.25	6.25
Roadway width (ft)	35.5	14.3	74.5
Shoulder width (ft)	8.9	0	20.8
Right-of-way (ft)	200	40	720
No. of lanes	2.88	2	4
Avg daily traffic	6608	570	20 000
Speed limit (mph)	53	25	55
Pedestrian age	31.9	2	82
Driver age[c]	33.1	17	85

[a]Curves to the left were assigned positive algebraic signs.
[b]Absolute value of curvature.
[c]Not all driver ages were reported because of hit-and-run accidents.

The right-of-way width, which was estimated on the basis of fence locations or the position of other boundary locators, averaged 200 ft, which suggests that, in the typical case, there is sufficient roadside width to accommodate a pedestrian path or sidewalk.

The volume of vehicular traffic at the sites averaged 6600/day, although certain sites had daily volumes as high as 20 000. The median speed limit was 55 mph, which reflects the fact that the studies were conducted at pedestrian-accident sites on rural highways. As might be expected, pedestrians were slightly younger than the drivers of vehicles that struck them, but the difference is not statistically significant.

Because previous studies of pedestrian safety had indicated that objects along the roadside may tend to hide pedestrians from the driver's view, the field study included a count of spot objects and the measurement of continuous objects in a 200-ft-long rectangle immediately upstream of the crash site and that extended 20 ft from the edge of the roadway. A substantial number of the sites had no fixed objects. Although spot objects averaged about 1.7 objects/site for those sites with objects, the value averaged over all the study sites was only 0.6. With the exception of embankments (slopes steeper than 4:1), which were present at 24 percent of the study sites, continuous fixed objects were found infrequently. Sight distance, measured during daylight hours and by using a 3.5-ft height of a driver's eye and a 4-ft height of an object, was less than 1000 ft at only 15 percent of the study sites. All of the sites provided a sight distance in excess of the values recommended by the American Association of State Highway Officials (AASHO) (2). The preliminary conclusion from the data in Table 4 is that, as a general rule, neither roadway alignment nor roadside features are causing a serious sight-distance restriction that could contribute to pedestrian accidents.

In the field studies, an evaluation was made of those features that may influence the actions of pedestrians. These items have often been identified in studies of urban pedestrian accidents as either possible contributing factors or as potential forms of remedial action. The inherent differences between urban and rural areas caused these features to be found infrequently at these pedestrian accident sites. With the exception of refuge islands (that, as a practical matter, were installed at 23 percent of the sites to separate traffic and provide left-turn bays), few devices were present to promote pedestrian safety. Parked vehicles did not seem to pose a problem at any of the sites. None of the sites could properly be described as limited access roadways. In fact, 86 percent of the sites were within 1000 ft of an intersection, although only 15 percent were characterized as intersection related.

Because the accident sites studied in this research were principally on major state routes, it was expected that the majority would have traffic signs and markings in compliance with the Manual on Uniform Traffic Control Devices (3). Centerlines were found at 95 percent of the sites and edgelines at 79 percent. In view of the generally favorable roadway geometrics, it was unusual to find no-passing zones at 41 percent of the accident sites, although it is recognized that factors other than alignment may necessitate the use of such markings. Although a total of 60 official traffic signs were found among all of the study sites, the median number of signs at a site was zero. Pedestrian crossing signs were found at 14 percent of the accident sites.

The investigating officers' reports provided some interesting information with respect to these accidents. The reports indicate that 28 percent of the accidents were hit-and-run, which in turn reduced the amount of information on driver age and vehicle type. Among those accidents that involved a known vehicle type, in 47 percent of the accidents the impacting vehicle was a passenger car while in 40 percent of the accidents it was a pickup truck. Casual observation of the vehicle mix on these roads suggests that, despite their high percentage, pickup trucks may not be overrepresented in these accidents. The reports also indicate that the weather was clear in 88 percent of the accidents and that the pavement condition was dry 86 percent of the time. The hourly distribution of these accidents differs substantially from typical rural accidents, with 56 percent occurring between 6:00 p.m. and midnight, and an additional 26 percent between midnight and 6:00 a.m.

A review of the sketch drawn by the officer and his or her descriptive comments on the factors surrounding the accident gave some indication of the action taken by the pedestrian. The findings, summarized in the table below, must be viewed cautiously, considering that the majority of the pedestrians were fatally injured, and statements from drivers, who typically did not see the pedestrian until it was too late to take corrective action, must be considered suspect:

Action	Percent
Crossing roadway	47
Walking with traffic	19
Standing in roadway	8
Walking in roadway	6
Walk into vehicle path	5
Other	11
Unknown	4

The principal pedestrian action involved crossing the highway. Examination at the sites of these accidents showed that the principal attractions to cross the road were taverns (12 sites), houses (4), and to get to an intersecting street (4). In the remainder of the crossing cases, the reason for crossing could not be established. Although 18 accidents involved pedestrians walking with traffic, only 2 involved pedestrians walking against the direction of traffic.

The investigating officer is asked to report the sobriety of drivers and pedestrians involved in accidents. For the accidents evaluated in this study, 14 percent of the drivers and 40 percent of the pedestrians reportedly had been drinking. These values may be considerably in error, because the sobriety of 28 percent of the drivers (principally those involved in hit-and-run accidents) and 48 percent of the pedestrians was reported as unknown on the accident report. On the basis of supplementary information provided by the New Mexico Traffic Safety Bureau, it was determined that 61 percent of the pedestrians were intoxicated.

Following the measurements at the site and a review of the investigating officer's report, the researchers attempted to determine if one or more types of engineering countermeasures were applicable at the location. Several factors influenced the selection of suitable types of remedial action. For example, roadway illumination would not be recommended at an isolated point or at a location without available electric power. The table below summarizes the recommendations with respect to possible countermeasures. (Note, the percentage of sites totals to more than 100 percent because of multiple improvements at some sites. For improved signing, these primarily suggest the use of W11A-2, the pedestrian-crossing sign. Other actions primarily include improved pavement markings, although a few cases involved speed limit reductions.)

Countermeasure	Percentage of Sites
Shoulder improvements	36
Improved lighting	29
Improved signing	19
Visibility enhancements	12
Other actions	11
Installation of refuge island	5
Installation of sidewalks	2
Pedestrian crosswalk	1

There is no guarantee that implementation of these recommendations will reduce the frequency or severity of the pedestrian accidents. The suggestions are based on site characteristics, the nature of the accident, and recommendations from technical literature (4).

The accident site data were examined with correlation analysis techniques. Although a number of variables were significantly correlated, their practical interpretations were intuitive and added little to the study's findings. Perhaps the most meaningful result of this analysis was the lack of significant correlation in two areas. It was found that the alignment parameters (curvature, gradient, and superelevation) were not correlated with one another or with any other variable in the analysis. In addition, roadway width--which is clearly a measure of the importance of the road as indicated by its positive correlations with average daily traffic (ADT), speed, and functional class--is not correlated with shoulder width. In practical terms, the conclusions from this analysis are that, at the sites of pedestrian accidents, higher-level rural roads do not have significantly better alignment features or shoulder width than the lower-class (functional) roads.

Previous analysis had found that only a few sites had what might be described as adverse geometrics. The degree of curvature was in excess of 2° at 5 of the sites, while 12 of the sites had gradients with an absolute value in excess of 3 percent. An examination of the combined alignment features showed that, for the most part, sites with sharper curvature tended to be close to level and sites on grades tended to be nearly tangent. Only one site had the combined features of high curvature and grade.

In a final attempt to determine underlying relations among the characteristics of the variables measured at the pedestrian accident sites, factor analysis was applied to 11 variables: ADT, light condition, pedestrian age, speed, road width, shoulder width, right-of-way width, number of lanes, curvature, gradient, and superelevation. This methodology sought to identify groups of variables (referred to as factors) that could explain substantial portions of the observed variations in the variables and had been used previously with some success in a study of suburban pedestrian accidents (5). Each factor consists of two or more variables that, when grouped together, provide a general characterization of the accident occurrence. The factors themselves do not have names, but rather must be interpreted by the analyst.

By using accepted statistical techniques, it was possible to reduce the 11 variables to 4 factors that accounted for a total of 61 percent of the variation. These factors are identified in the table below [note, all variables have positive loadings (i.e., correlations) with the factors except speed, which has a negative loading]:

Factor	Included Variables	Variation (%)	Interpretation
1	No. of lanes, road width, and ADT	20.4	Major road
2	Right-of-way and shoulder width	14.9	Room for safe pedestrian movement
3	Curvature, gradient, and speed	13.5	Adverse geometrics
4	Superelevation and pedestrian age	12.4	Uncertain

Factor 1 is made up of three variables, which as a group suggests a major roadway. This factor is not surprising, since the site-selection process emphasized these types of roadways. Factor 2 consists of two factors, which suggests the availability of room alongside the roadway for safe (longitudinal) pedestrian movement. Factor 3 consists of two alignment variables together with the speed limit, which has a negative correlation with the factor. The factor indicates poorer geometrics, principally at sites with lower speed limits. Factor 4, while statistically meaningful, does not seem to have a practical interpretation.

CONCLUSIONS

Rural pedestrian accidents have not been studied in detail because pedestrian activity in these areas is low, pedestrian accidents are (generally) few in number, and those that do occur are so spacially separated as to preclude effective remedial action. However, the limited geographic area examined in this research differs from the typical pattern of rural pedestrian accident frequency and, therefore, offered the potential of discerning what, if any, characteristics of a type susceptible to engineering forms of corrective action were common to these accidents.

Data from this study show that the pedestrian accident sites have several common characteristics.

On the positive side, the sites were generally characterized by good alignment. Daytime sight distance was more than adequate, with 85 percent of the sites having sight distances in excess of 1000 ft and all providing sight distances in excess of AASHO standards. The data also reflect the typically uncluttered nature of New Mexico's roadsides, with the absence of fixed objects, which precludes the opportunity for a pedestrian to dart-out from behind a tree or post into the path of a vehicle. Lane width, with an average of just greater than 12 ft, is certainly adequate for the types of roadways studied in this project. The presence of pedestrian warning signs at 13 of the accident sites shows a recognition by highway officials of the pedestrian activity at some of the study locations.

The principal negative finding from the field study involves the general lack of pedestrian facilities on these roadways. The absence of such facilities may be attributed in part to the failure of AASHO, the prime source of standards for rural highway design, to provide warrants for their use. Although the "Blue Book" (2) suggests that sidewalks along rural highways will reduce pedestrian accident experience, the general absence of sidewalks in rural areas of New Mexico virtually precludes the opportunity to evaluate either their use or effectiveness. AASHO supports the use of adequate shoulders in lieu of sidewalks.

A factor that is troublesome for the engineer is the 80 percent of the pedestrian accidents that occur at night. The good alignment, adequate sight distance, and clear roadside provided by the engineer all assume secondary importance when the actual sight distance is limited by the illumination from vehicle headlights. Although some of the sites, principally those near major intersections or commercial development, could profit from the installation or upgrading of roadway lighting, the application of this type of improvement was judged to be useful or practical at less than 30 percent of the accident sites.

One finding of the study that could be of value in planning and implementing corrective action is that the accidents are not uniformly distributed on the road system. The 95 sites studied actually involved only 82 locations; the difference is due to sites with multiple accidents during the study period. Two clusters of pedestrian accidents along 14 miles of rural road accounted for 39 percent of the accidents, and they obviously deserve the most immediate attention.

A final conclusion is worth mentioning. This study is one of the more thorough that has been conducted on this topic. The measurements made were comprehensive from an engineering point of view and suitably precise to meet the needs of the study. Statistical techniques, however, which experience has often shown are able to help in explaining the interaction of parameters associated with accident occurrence, yielded somewhat inconclusive results. The most logical explanation is that the human and vehicular factors not examined in this study may be of overriding importance in these accidents.

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The Obstacle Course: Pedestrians in Highway Work Zones

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There has been an increased awareness of the safety problems associated with highway work zones among traffic professionals and government agencies during the past several years. For the most part, attention has been focused on motorist safety, while pedestrian safety has been virtually ignored. Federal, state, and local manuals do not adequately address the issue of pedestrian accommodation and safety in highway work zones. There are currently no well-defined techniques, standards, or practices that pertain to pedestrian control and safety in work zones. Traffic-control plans for maintenance of traffic through work zones are prepared primarily for vehicular traffic and make rare reference to pedestrians. Inadequate pedestrian accommodation in work zones forces pedestrians to choose their own paths and fight through construction areas full of debris and other obstructions. Pedestrian accommodation and safety in work zones deserve careful attention. Guidelines for accommodating pedestrians should be developed at the federal level and preferably included in the Manual on Uniform Traffic Control Devices for proper implementation and uniform compliance. A concerted effort to improve pedestrian safety in work zones is highly desirable. Pedestrians should be afforded the same rights and privileges enjoyed by vehicles that pass through construction zones.

Traffic control in highway work zones has become a major safety concern for traffic professionals and government agencies during the past several years. Traffic-control efforts in highway work zones have primarily been directed toward vehicular traffic. The Manual on Uniform Traffic Control Devices (MUTCD) (1) and the associated supplements (2,3) provide a comprehensive coverage of guidelines, principles, and devices for vehicular traffic control in highway work zones. These guides, however, do not adequately address the issue of pedestrian safety and accommodations in highway work zones.

The requirements for preparation of traffic-control plans (TCPs) for all major highway construction projects are essentially related to vehicular traffic control in work zones. Maintenance of traffic is generally included as a separate pay item on all major highway projects. TCPs for highway projects rarely make reference to pedestrian accommodations and safety in work zones.

Given this lack of attention to pedestrian needs, one could expect to find, in general, a lack of pedestrian accommodations in terms of separators, protection, and guidance devices at highway work zones. This situation is exactly what was observed at several sites surveyed during the course of a study on pedestrian accommodation in highway work zones conducted for the Federal Highway Administration (FHWA) (4).

Figures 1 through 8 provide evidence of some typical hazards and pedestrian safety problems that result from the lack of (or inadequate) accommodation for pedestrians in work zones. Blocking or closing of existing sidewalk pathways without providing alternate pathways and adequate information endangers the safety of pedestrians in and around work zones. The lack of pedestrian accommodations and guidance forces pedestrians to choose their own paths, walk in traffic lanes, and place themselves in hazardous conditions. Improperly placed traffic-control devices, signs, barriers, and separators create their own hazards and add to the confusion of pedestrians and motorists alike.

The important reasons for accommodating pedestrians in work zones include the following:

1. To provide pedestrian access to abutting properties or the right-of-way in a safe manner,
2. To minimize adverse economic consequences for commercial establishments in the vicinity of the highway work zone, and
3. To minimize the possibility of tort liability claims that result from inadequate provisions and protection.

In addition, the needs of special groups of pedestrians, such as children, the elderly, the handicapped, and the blind, where they are expected to pass through work zones, need to be considered, evaluated, and adequately accommodated.

This paper briefly summarizes the state of the art and current practice for pedestrian accommodations in work zones, findings based on field investigations and discussions with professionals, and recommendations for future efforts in this area. This paper deals primarily with work zones that involve highway construction and maintenance activities, but other work-zone types, which involve building construction, public works projects, and utility operations, were also included in the study to provide a good mix of field sites.

Figure 1. Sidewalk completely blocked by construction material; no pedestrian information or accommodations provided.

Figure 2. Sidewalk completely blocked by construction equipment; pedestrians forced into traffic lanes.

Figure 3. Inadequate protection around utility excavation creates pedestrian safety problem.

Figure 4. Temporary ramps with steep slopes create unsafe conditions for pedestrians with mobility problems.

Figure 5. Inadequate delineation of pedestrian pathway and protection results in confusion and discourages use.

Figure 6. Crosswalk blocked by several devices; no pedestrian information provided.

CURRENT POLICIES AND PROCEDURES

Current policies and procedures for accommodating pedestrians in work zones were determined from various sources, including a literature review, field visits, and discussions with professionals associated with various levels of government.

Literature Review

A literature review of the national, state, and local manuals and other publications revealed only sparse reference to pedestrian accommodations in work zones. At the federal level, guidelines and standards for work-zone traffic controls are contained in MUTCD (1) and its supplemental handbooks. Part VI of MUTCD provides specifications and guidelines for traffic control in highway work zones. Although some of the principles and information provided in MUTCD would apply to pedestrian controls, the entire section is primarily devoted to vehicular traffic control with no specific guidance for pedestrian control.

The appendix to Part VI of MUTCD, Traffic Control

Figure 7. Absence of pedestrian access to abutting properties on highway widening project results in hazardous conditions.

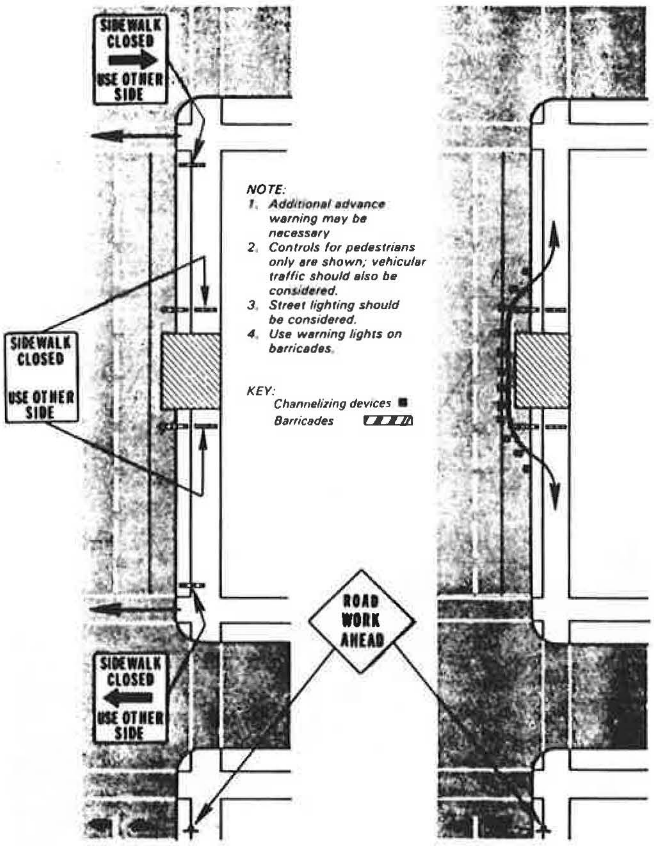
Figure 8. Inadequate separation and protection from vehicles endanger pedestrians' safety.

Devices Handbook--An Operating Guide, contains only very broad references to pedestrian needs throughout the handbook and contains a brief section specifically relating to pedestrian considerations in work zones (2). It also provides a schematic illustration of methods of controlling pedestrian traffic, as shown in Figure 9 (2, p. 61). This information is simply not specific enough for planners to determine the type and level of accommodation for pedestrians through work zones.

Some additional guidance for pedestrian control is contained in Traffic Controls in Construction and Maintenance Work Zones (3). The report suggests the use of protective barricades, fencing, handrails, and bridges together with warning and guidance devices for the safety and delineation of passageways for pedestrians and bicycles, bridle paths, and other nonmotorists in work zones. Provision of alternate walkways is also recommended where existing walkways are closed by construction or maintenance. Installation of a fixed pedestrian walkway of the fence-and-canopy type to protect and control pedestrians is also recommended where hazardous work conditions exist overhead.

A search of the state manuals again revealed very little reference to pedestrian planning considerations in work zones. Some states simply included the pedestrian protection requirements in their own manuals on uniform traffic-control devices.

Figure 9. Typical application of two methods for accommodating pedestrians in work zones.



The description of pedestrian protection included in city and county construction manuals is also very brief and of a very general nature. Most of these manuals contain either the same or similar general and brief paragraphs and descriptions that relate to pedestrian considerations in work zones. The work area traffic-control manuals for the cities of High Point, North Carolina; Wichita, Kansas; Los Angeles; Phoenix; Flint, Michigan; and Seattle included somewhat detailed coverage of pedestrian protection requirements and also some typical illustrations for pedestrian control in work zones (5-10). The information contained in these city manuals is neither detailed nor site specific to help the designer evaluate pedestrian accommodation needs in work zones (11).

In summary, the federal, state, and local manuals reviewed do not specify any conditions under which pedestrian accommodations in highway work zones are essential. There are currently no well-defined techniques, standards, or practices that pertain to pedestrian accommodation, control, or safety in work zones. Also, no documentation or criteria for determining the type of pedestrian accommodation needs and devices to be used at a given site for a particular pedestrian or traffic volume is currently available.

Discussions with Federal, State, and Local Officials

Discussions held with officials of various government agencies confirmed the contention that little, if any, attention has been given to the pedestrian needs through and around work-zone sites, particularly on highway construction projects. The usual reasons given for this lack of attention range from "There are no pedestrians on our highway jobs," to "MUTCD does not specifically require a plan for pedestrians in construction areas."

The following additional observations are made based on the general findings gleaned from the discussions with the various officials:

1. Some officials mentioned that because, in many situations, pedestrian facilities such as sidewalks are either nonexistent or poorly maintained during nonconstruction times, how can one expect to provide a neat, clean, reasonably safe, and obstacle-free pedestrian path through a work zone just for the duration of the construction period. The basic tenet is that, "If you don't do it, why do I have to do it."
2. Some officials indicated that no problem of pedestrian accommodations and safety in work zones existed because they had received no complaints. On the other hand, people affected to not seem to be disposed to complain because they either do not know to whom to complain or feel that complaining does not do any good.
3. Currently, no statistics that relate to pedestrian accidents in work zones are available to truly gauge the type and magnitude of the pedestrian safety problem. However, realizing the potential benefits, some states have now started to keep proper records of vehicular and pedestrian accidents in work zones.
4. All agencies generally referred to Chapter VI, Traffic Construction and Maintenance, of MUTCD for preparation of TCPs for work zones. However, MUTCD does not adequately address the problem of pedestrian accommodation and safety in a work-zone environment.
5. Urban areas included in the study generally seem to have a greater awareness of the pedestrian problems in work zones and have developed some guidelines and policies for these requirements. However, the guidelines and policies were not available in published form in all cases.
6. Regarding the question of establishing a minimum threshold level of pedestrian activity for considering pedestrian accommodation in work zones, the response was generally guarded. The general feeling was that pedestrian needs should be considered in relation to pedestrian activity, type and duration of the project, and level of hazard in a work-zone environment. Some work sites would not require any accommodation, regardless of pedestrian volume, while some other types of work zones (building construction, deep excavation, etc.) would require pedestrian accommodation and protection in most cases. Minor sidewalk repair or interruption seemed to be of least concern for pedestrian considerations, while work sites that involve deep excavations (for example, subway construction) are the greatest concern.
7. Some agency officials cited lack of any information, guidance, and standards for pedestrian accommodation in work zones as the primary reason for not considering the pedestrian access needs in a highway construction environment.
8. There is a general recognition and realization among professionals that pedestrian access and safety needs should be considered during the planning and design stages of highway construction or reconstruction projects.
9. The general feeling among professionals was that the guidelines for pedestrian accommodation and safety in work zones should be incorporated in MUTCD for proper compliance and implementation. MUTCD is the single primary resource used by professionals and field staff for maintenance of traffic in work zones.

Table 1. Distribution of study sites.

Study Area	Area Type: Commercial	Residential	Educational	Other	Construction Type: Highway	Utility	Building	Other	Duration: 1 Day or Less	1 Day to 1 Week	1 Week to 1 Month	More Than 1 Month	Pedestrian Volumes[a]: Low	Medium	High	Total
Tampa area	3	2	2	4	6	3	2					9	4	4	1	9
St. Petersburg area	5	3		1	7	1				1		6	5	2		7
Fort Meyers area	4	2			4	1					1	4	5			5
Atlanta	9	8	3	4	7	2	1	2	2	1		9	4	8		12
Denver	6	2	2	2	7	4					1	7	2	1	5	8
Salt Lake City	7	5	1		5	4	1			2		7	4	5		9
Washington, D.C.	5	2			5	1	1	3			1	6		3	4	7
Washington metropolitan area	8	9	3	1	9	3		6		1	3	8	8	1	3	12
Baltimore metropolitan area	12	9	1	2	12	3		4		4		11	6	4	5	15
Northern Virginia area	3	4	1		3	1				1		3	2	2		4
Miami and Ft. Lauderdale area	8	5	4		7	2		3			2	9	7	4		11
Total	70	51	17	14	72	25	5	18	2	10	8	79	47	34	18	99

[a]Pedestrian volumes are as follows: low = less than 50 pedestrians/h, medium = 50-150 pedestrians/h, and high = more than 150 pedestrians/h.

Figure 10. Example of bad pedestrian accommodation; pedestrian needs totally ignored on this side.

Figure 11. Example of good pedestrian accommodation; adequate pedestrian bypass provided on this side.

10. Traffic engineers and construction engineers suggested that the type, size, and color of the signs to be used for pedestrian information and accommodation in a work-zone environment should be standardized. The same suggestion applies to other traffic-control devices needed for pedestrian accommodation in work zones. The current practice of using nonstandard signs of various color, size, and materials provided misleading and confusing information in some work-zone situations.

11. Pedestrian accommodations are generally provided only if the design plans indicate their need or are based on the personal judgment or knowledge of the field engineer responsible for construction.

12. Currently, enforcement of traffic-control plans is done primarily on major construction projects. In regard to enforcement, some agencies stated that they can shut down the job if a particular contractor is in constant violation of the approved TCP. Others mentioned that they simply do not have any legal basis for such an action.

Field Investigations

Field investigations were conducted at nearly 100 work-zone sites located in different geographic sections of the country. Table 1 gives the distribution and other attributes of the study sites. The purpose of the field observations was to conduct investigations and critiques of as many different types of pedestrian work zones as possible. Field investigations also provided a basis for identification of good and bad practices related to provisions for pedestrian control and safety in work zones. An example of good and bad pedestrian accommodation at the same work zone site is shown in Figures 10 and 11. Adequate guidance, channelization, and bypass away from the work site were provided for pedestrian traffic on one approach (Figure 11), while pedestrian needs were totally ignored on the other approach of the same site (Figure 10).

Field data were based mainly on observations (except for some physical site data and traffic count data). No specific pedestrian or driver performance data were collected. Site investigations included an evaluation and critique of the pedestrian information (and motorist information if applicable to the pedestrians), guidance, separation, and protection needs. Site-specific pedestrian needs were identified, and adequacy of traffic-control plans to meet desired needs were evaluated based on observed pedestrian behavior and engineering judgments. Good and bad practices were identified to extrapolate planning principles and guidelines based on observations. The data-reduction procedure consisted of reviewing slides, photographs, field sketches, comments, and notes made on the data-collection form for each study site.

The following observations are made based on the results of the field investigations:

Figure 12. Informational signs currently used at work zones.

SIGN MESSAGE	COLOR BACKGROUND	COLOR LETTERING	MATERIAL
CROSSWALK CLOSED USE OTHER SIDE	WHITE	BLACK	WOOD
SIDEWALK CLOSED PEDESTRIANS USE OTHER SIDE	WHITE	BLACK	WOOD
PEDESTRIANS USE OTHER BRIDGE	WHITE	BLACK	METAL
DANGER NO PEDESTRIANS	RED	BLACK	METAL
SIDEWALK CLOSED PEDESTRIANS CROSS OVER ← - →	WHITE	BLACK/ RED	WOOD
PEDESTRIANS USE WALKWAY ↘	WHITE	BLACK	METAL
PEDESTRIANS KEEP RIGHT	CONSTRUCTION ORANGE	BLACK	METAL
PEDESTRIANS ←	CONSTRUCTION ORANGE	BLACK	METAL
PEDESTRIANS ← SIDEWALK	CONSTRUCTION ORANGE	BLACK	METAL
SIDEWALK CLOSED USE EAST SIDEWALK	WHITE	BLACK	METAL
PEDESTRIAN WALKWAY ↘	WHITE	BLACK	CARD- BOARD
SIDEWALK CLOSED	WHITE	BLACK	WOOD/ METAL
SIDEWALK CLOSED / PEDESTRIANS ←	WHITE / CONSTRUCTION ORANGE	BLACK	METAL
PEDESTRIAN CROSSING	WHITE	BLACK	METAL
PEDESTRIANS PROHIBITED ON THIS SIDE	WHITE	BLACK	METAL
DANGER DO NOT WALK ALONG FENCE →	YELLOW	BLACK	METAL

NOTE: SIZES OF SIGNS VARY AND ARE IRREGULAR.

1. Pedestrian activity can be found at nearly every work-zone site.
2. Pedestrian accommodations of some type were generally provided for the following type of construction zones: (a) major building construction work where required as a part of local building codes; (b) construction zones located in urban areas, especially in downtown areas that generate heavy pedestrian and vehicular traffic volumes; and (c) work zones that are associated with subway construction and that impact highway and pedestrian traffic.
3. The agencies that provided pedestrian accommodation in work zones have used traffic-control devices, including cones, signs, rope, flagging tape, barricades (types I and II), and other types of barriers (fencing, plywood walls, etc.) mostly based on the judgment of the construction supervisor. Canopy-type protection was used where there is an overhead construction hazard.
4. Currently, informational signs used at work-zone sites vary by size, color, and material. Both regulatory and construction orange signs are common. Sign material included cardboard, wood, and metal. Some of the signs used appeared to have been fabricated at the site. Figure 12 shows an assortment of signs and messages used for pedestrian information at some of the study sites.
5. Many jurisdictions require submission of TCPs that conform to MUTCD on major highway and utility construction projects. Currently, this requirement is applicable to control of vehicular traffic in work zones. Pedestrians are considered only in situations where the volume and safety problems justify their accommodation (i.e., urban areas or, specifically, downtown areas). In some cases, a general and brief note that relates to pedestrian considerations in work zones appeared on some TCPs. The following are typical examples of such notes found on the TCPs reviewed for some of the study sites: "If pedestrian traffic cannot be maintained on existing crosswalks, pedestrians must be routed to desired paths by means of barricades and/or signs." "A 5-foot walkway suitable for pedestrian use must be provided at Cherry Street, Dahlia Street, etc., ... on weekends and holidays, to allow pedestrian access to churches in that area." "Special considerations shall be given to pedestrian safety when construction work encroaches upon a sidewalk, walkway, or crosswalk."

Finally, traffic regulation specifications for Metropolitan Atlanta Rapid Transit Authority (MARTA) construction projects in Atlanta included the following special provision in the contract documents:

> Regulating vehicular and pedestrian flow adjacent to the work site shall consist of ensuring that construction operations do not impede vehicular and pedestrian traffic to the extent that public safety will be threatened and the passage of emergency vehicles will be restricted. Public ways, including streets, sidewalks, and accesses to public and private properties, shall not be obstructed, and the carrying capacity shall not be reduced, except as indicated on the reviewed and accepted traffic regulating plan. Pavement surfaces shall be maintained in a smooth riding plane where vehicular and pedestrian traffic is routed. Excavations in those areas shall be backfilled, and temporary pavement shall be installed immediately after the backfill has been placed. If temporary pavement becomes rutted, it shall be supplemented with steel plates over the excavation or by other accepted means. Each section of permanent pavement and sidewalk shall be restored as soon as is practicable after completion of the work for which that section of pavement and sidewalk was removed. Obstructed public ways, including streets, sidewalks, and accesses to public and private properties, shall be returned to public and private uses when the obstruction thereto is no longer necessary for the prosecution of the project.

Vehicular and pedestrian access to buildings adjacent to the work site shall be unimpeded by construction operations to the extent that public safety will not be threatened and that public convenience will not be unduly impaired in the opinion of the engineer.

Closing streets and sidewalks shall be in accordance with the reviewed and accepted traffic regulating plan.

CURRENT TECHNIQUES AND DEVICES

Techniques and devices currently used for pedestrian accommodation in work zones have been identified from two separate sources--the literature review and the field investigation. The literature review indicated a very general and brief discussion of when and how to accommodate pedestrians in work zones. Diversion of pedestrians into curb or parking lanes or to the other side of the street is suggested where existing walkways necessitate blocking or closing for construction and maintenance activities. Likewise, installation of a fixed pedestrian walkway of the fence-and-canopy type for pedestrian accommodation is recommended where overhead protection is required.

The pedestrian accommodation techniques found in the literature do not specify their use in relation to site conditions, i.e., type of work zone, duration of construction activity (long- versus short-term projects), pedestrian activity levels, vehicular traffic, and degree of hazard. Further, no detailed planning and implementation guidelines are currently available that can be used by planners or designers for accommodating pedestrians in work zones.

Based on field investigations, the following techniques for pedestrian accommodation have been used in varying degrees at some of the sites:

1. None; the majority of sites reviewed did not employ pedestrian accommodations of any type, even though pedestrian use and the potential for accommodation was observed;
2. Use of an existing pathway mainly where construction was either overhead (bridge-widening projects or building construction) or the physical separation between the pedestrian path and construction site was adequate;
3. Directing pedestrians to either an alternate walkway or another route by use of a bypass or a detour; and/or
4. Closing the work-zone area to pedestrians by erecting NO PEDESTRIANS signs without providing an alternate pathway for the pedestrians.

Field investigations also revealed that pedestrian accommodation techniques were not uniformly used at different segments of the same work zone or at different sites in the same area. This could be due to lack of adequate local planning and enforcement policies and procedures. Further, techniques were rarely modified to meet changing work-zone environments in relation to construction schedules and activities. The literature review and field investigations revealed that a wide assortment of devices is currently used for accommodating pedestrians in and around a work-zone environment. The outline below lists these devices by groups:

- I. Channelization
  - A. Cones
  - B. Barricades (types I and II)
  - C. Markings
  - D. Flagging tape
  - E. Rope
  - F. Construction delineators
- II. Advance warning
  - A. Signs
  - B. Barricades (for sidewalk closure or blockage)
  - C. Cones
- III. Separators
  - A. Pedestrian and vehicle
    - 1. Traffic cones
    - 2. Barricades (type I and II)
    - 3. Wooden handrails
    - 4. Portable concrete barriers
  - B. Pedestrian and construction
    - 1. Wooden handrails
    - 2. Wooden post and plywood fences (walls)
    - 3. Chain-link fences
    - 4. Portable concrete barriers
- IV. Pathway material
  - A. Concrete
  - B. Wood
  - C. Plywood overlaid with roofing felt material
  - D. Steel plates
  - E. Asphalt

Available literature and field investigations do not provide any information relating to the type of device to be used for a given situation based on type and duration of construction work or pedestrian and vehicular traffic levels. Further, use of various devices to perform functions such as pedestrian control, separation, protection, informational needs, channelization, and guidance in work zones is not adequately documented in the existing literature.

CONCLUSIONS AND SUGGESTIONS

The results of the literature review revealed only brief and sparse reference to pedestrian planning and safety considerations in work zones, except where pedestrian accommodations are required by local building codes for building construction work. Traffic-control plans required for highway construction projects make rare reference to pedestrian safety needs and are primarily directed toward vehicular traffic control. Very little formal guidance for use of any techniques or devices for pedestrian accommodation for a given work-zone situation is currently available.

It seems as though there is no real concerted effort being made by any organization, group, or agency to afford the pedestrian the same rights and privileges that a vehicle has as it passes through a construction zone. The vehicle is provided with an open, nearly unrestricted, relatively smooth, fairly well-marked way to proceed either around and through or detoured completely away from construction areas and problems. The pedestrian, however, is simply allowed to fight through construction areas full of debris, mud, and other obstructions.

Because nationally accepted guidelines for pedestrian accommodation in work zones are currently not available, some local jurisdictions have taken a lead and developed their own policies and standards. Other local jurisdictions that recognize the problem are simply following the information and guidance developed by these agencies.

Inadequate accommodation of pedestrians in work zones is the result of (a) a lack of awareness of the pedestrian safety problems; (b) a lack of the necessary information, guidance, applicable standards, adequate legal precedence, and work-zone pedestrian accident statistics; and (c) the absence of adequate enforcement policies and procedures at local levels.

The following observations and professional judgments are based on the experience gained through the conduct of this study:

1. Pedestrian activity can be found near every work zone.
2. Pedestrian routes should be well marked, safe, efficient, and easy to follow.
3. Pedestrians should have the same right to traverse a work zone as does a vehicle, even if detouring or rerouting is required.
4. In residential and commercial areas, adequate pedestrian access should be provided to properties abutting work zones.
5. Logical, visible, and direct paths will be followed by pedestrians regardless of the level of hazard. Pathways that route pedestrians out of their way will only be used when absolutely necessary.
6. A minimum amount of special devices need to be provided for special groups, i.e., the elderly and the handicapped. Designs should include ramps and curb-free walkways.

A general apathy toward pedestrians also exists. The persons responsible for developing maintenance of traffic plans still seem to be resistant to considering the pedestrians within the scope of maintenance of traffic. The problem seems to be a spill-over from the general attitude toward pedestrians.

The bottom line is that efforts must be made to accommodate and provide for pedestrian safety through work zones. This, however, is dependent on the fact that the pedestrians or nonmotorized vehicles have the right to traverse the work zone or gain access to abutting properties in a safe manner.

Based on the experience gained through field work, the project study team recommends consideration of pedestrian accommodation within work zones in the following situations:

1. Where sidewalks existed prior to construction,
2. Where the work-zone site is located along a designated route to a school,
3. Where there is evidence of pedestrian use (i.e., well-worn paths exist), and
4. Where existing land use (shopping centers, parks, recreation areas, educational institutions, community centers, residential developments, etc.) generates pedestrian traffic.

The results of the research indicated the need for pedestrian considerations in urban work zones. TCPs for highway construction projects should include considerations for pedestrian accommodation and safety where pedestrian use either exists or is expected to be generated based on land use. Further, the guidelines for pedestrian accommodation in highway work zones should be prepared at the federal level and preferably included in MUTCD for proper compliance, implementation, and enforcement. It is desirable that guidelines should specify conditions under which pedestrians should be accommodated in work zones and also suggest techniques and devices to be used for site-specific needs. The guidelines should be rational, simple, easy to understand and implement, and usable by planners, designers, engineers, and construction and maintenance personnel.

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The contents of this article reflect our views, and we are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policy of the U.S. Department of Transportation. The U.S. government assumes no liability for its contents or use thereof.

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Development of Improved Pedestrian Warrant for Traffic Signals

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The 1978 version of the Manual of Uniform Traffic Control Devices (MUTCD) specifies a total of eight traffic signal warrants, and one of these is the minimum pedestrian volume warrant. Many traffic engineers and researchers have argued that the minimum pedestrian volume is inappropriate, in that an inordinately large pedestrian volume is required over an extended period (150 pedestrians/h for 8 h during a 24-h day on the highest volume crosswalk). As a result, most of the pedestrian signals installed in the United States today are based largely on the intuitive judgment of traffic engineers. The purpose of this paper is to (a) conduct an in-depth review of the current MUTCD pedestrian volume warrant and other recommended warrants reported in the literature and from other countries, and (b) develop and recommend a revised pedestrian signal warrant that might lend itself to better practical application. Five criteria were used in evaluating the MUTCD pedestrian warrant, and the review generally indicated that the current warrant is inappropriate for most real-world situations. A revised warrant was developed based on an analysis of pedestrian volume distributions in a number of U.S. cities and a branching analysis of pedestrian accident and volume data. The warrant requires minimum hourly pedestrian volumes on an average day of 60 or more (for each of any 4 h), 90 or more (for each of any 2 h), or 110 or more (during the peak hour). The number of adequate gaps in the traffic stream should also be less than 60/h during the same period when the pedestrian volume criterion is satisfied.

The 1978 version of the Manual of Uniform Traffic Control Devices (MUTCD) specifies eight warrants for the installation of a traffic signal. Of the eight warrants, warrant 3 (minimum pedestrian volume) and warrant 4 (school crossing) are most related to pedestrians. According to MUTCD, pedestrian signal indications (i.e., WALK/DON'T WALK signals) shall be provided when a traffic signal is installed under the pedestrian volume or school crossing warrant (1). Warrants 6 (accident experience) and 8 (combination of warrants) also allow for some consideration to pedestrians. In addition, MUTCD requires pedestrian signal indications (a) when an exclusive pedestrian phase is provided, (b) when vehicular signal indications are not visible to pedestrians, and (c) at signalized school crossing intersections.

Also, MUTCD suggests that pedestrian signal indications may be installed when (a) a pedestrian signal is needed to minimize vehicle-pedestrian conflicts or to assist pedestrians in making a safe crossing, (b) multiphase indications may confuse pedestrians, and (c) a divided roadway exists and the signal timing only allows pedestrians to cross to the island during one interval.

Note that two separate issues must be addressed, including (a) warrants for installing new traffic signals (with pedestrian signal indications) based on pedestrian considerations, as provided in warrant 3 (minimum pedestrian volume) and warrant 4 (school crossing), and (b) warrants for installing new pedestrian signal indications (i.e., WALK/DON'T WALK signals) where traffic signals already exist. This study focuses primarily on the former issue, that is, warrants for installing new traffic signals, particularly as they relate to the minimum pedestrian volume warrant (warrant 3).

The basic minimum pedestrian volume warrant requires 600 vehicles/h entering the intersection (both approaches of the major street) for each of any 8 h of an average day and also 150 or more pedestrians/h during the same period on the highest-volume crosswalk. Many traffic engineers and researchers have argued that the current MUTCD pedestrian volume warrant is inappropriate. Pedestrian volume requirements are considered too high by most traffic experts to have any practical applications. In order to provide pedestrian signalization, many traffic engineers must rely on their own engineering judgment when selecting locations for pedestrian signal installations. This has created inconsistencies between regions of the country and often between state and local agencies concerning the conditions under which pedestrian signals are installed.

The purpose of this paper is to review and critique the existing MUTCD warrant and other relevant guidelines reported in the literature and to recommend a revised warrant more suitable for practical application in the United States. The work reported in this paper was conducted as a part of a Federal Highway Administration (FHWA) sponsored study on pedestrian signalization alternatives, which is currently in the final stages of completion.

REVIEW OF EXISTING AND PROPOSED WARRANTS

A review of the pedestrian volume warrant included conducting a comprehensive literature review to find other studies that have been conducted relative to this warrant. In particular, studies that provided other recommended warrants to replace the current MUTCD one were analyzed to determine their validity. A critical analysis of the MUTCD warrant and the other proposed pedestrian volume warrants was helpful in the development of recommended warrants. This review is presented below.

The 1978 MUTCD warrant for minimum pedestrian volume (warrant 3) is satisfied when 600 or more vehicles/h enter an intersection (both approaches of the major street) for each of any 8 h of an average day along with 150 or more pedestrians/h during the same period crossing the highest-volume crosswalk crossing the major street. For a divided highway, 1000 or more vehicles/h are required. Where the traffic speed exceeds 40 mph or in isolated communities (less than 10 000 population), the requirements are only 70 percent of those stated above. At midblock locations, the warrants are the same, provided that the crosswalk is not closer than 150 ft to another established crosswalk (1).

In 1967, a study was conducted by Box for the signal committee of the National Joint Committee on Uniform Traffic Control Devices (2). The purpose of the study was to review warrants for traffic signals and suggest considerations and numerical values for warrants. The warrant recommended in this study requires a minimum of 60 pedestrians/h for 1 h (or for two 30-min periods) and also an average of 60 s of mean delay per pedestrian for one of the two 30-min periods. This warrant is based on the premise that pedestrians are subjected to greater exposure to injury compared with motor vehicles, and motorists have the added protection from inclement weather.

A study was conducted in 1976 for the National Cooperative Highway Research Program (NCHRP) entitled Traffic Signal Warrants (3). The warrant is based primarily on pedestrian delay considerations and is presented in graphical form for undivided and divided streets. A minimum of 100 pedestrians/h is required to meet this warrant. Minimum required traffic volumes for undivided and divided streets are 500 and 1000 vehicles/h, respectively (3).

A delay-based warrant was presented by King in 1977 (4) that uses an exponential arrival distribution model originally developed by Tanner in 1951 (5). Based on a 30-s assumed acceptable level of pedestrian delay and a 60-s level of maximum tolerable pedestrian delay, pedestrian signal warrants were prepared graphically for undivided and divided highways. It should be noted that Tanner's delay model is based on the assumption of random arrival of vehicles, whereas vehicular arrivals in most urban intersections are not likely to be random in nature. Thus, the validity of using the Tanner delay model for developing warrants at urban intersections may be questioned.

The Canadian traffic signal installation warrant developed in 1966 is based on pedestrian volumes and delays. The specific warrant is as follows (6):

> a. Pedestrians on an average must wait in excess of 60 seconds before being able to cross the main street in safety;
> b. The number of pedestrians wishing to cross is at least 60 per hour;
> c. The conditions specified in (a) and (b) exist for any four not necessarily continuous hours of a normal day;
> d. The intersection or other location is suitable for signalization; and
> e. The nearest existing or proposed signal installation is more than 1000 feet away.
>
> The existing delay occasioned to pedestrians should be determined by a study at the location in question.

The Canadian warrants are similar to the warrants recommended by Box (2) in terms of the minimum required pedestrian volumes (60/h) and mean delay per pedestrian (60 s). However, the Canadian warrant requires those conditions for 4 h, compared with two 30-min periods in the Box-recommended warrants.

As a part of the FHWA-sponsored study from which this paper originates, we also reviewed pedestrian signal warrants in a number of other countries, including those in Great Britain, Ireland, Australia, and New Zealand. This review revealed a considerable amount of variation in the pedestrian volume (ranging between 90 and 600 pedestrians/h) for warranting a signal.

CRITIQUE OF MUTCD WARRANT

In order to evaluate the existing pedestrian signal warrant, five specific criteria were selected, as follows:

Criterion 1: appropriateness and reasonableness of the warrant,
Criterion 2: complexity of the warrant,
Criterion 3: data requirements,
Criterion 4: flexibility of the warrant, and
Criterion 5: acceptability of the warrant by practicing traffic engineers in the United States.

The intent of criteria 1 is to see if this warrant is realistic in terms of how many locations are likely to meet the warrant under real-world conditions. Criteria 2 is designed to test the amount of time and expertise needed to apply the warrant. Criteria 3 is on the data burden associated with the warrant, and criteria 4 is designed to answer the question if it can account for most of the real-world situations or if it offers ways to reduce required data-collection efforts or simplify the analysis procedure. Last, criteria 5 is somewhat a combination of the preceeding four but is important in its own right, since the traffic engineering community is ultimately responsible for using the warrants to install signals.

As a pedestrian signal warrant is tested by using each of the criteria, a rating of excellent, good, fair, or poor was assigned, depending on how well the criteria are satisfied. The assignment of these ratings is largely subjective, but much objective information was used to apply them. Also, it is important to state that not all criteria are of equal importance. For example, the appropriateness of a warrant is certainly the most important criteria, since if the warrant is totally inappropriate (criteria 1), then the other criteria do not really matter.

Criteria 1: Appropriateness and Reasonableness

The 1978 MUTCD pedestrian volume warrant (warrant 3) was evaluated by using the five criteria discussed above. In terms of appropriateness and reasonableness (criterion 1), several sources were used to judge the warrant. Discussions were held with more than 50 traffic engineers throughout the country who overwhelmingly indicated that the current MUTCD pedestrian volume warrant is unrealistically high. In most cities, few or no traffic signals can be justified based on the pedestrian volume warrant. This is confirmed by a survey of current practices in the NCHRP report, which showed that only 171 out of 12 780 traffic signal installations (1.3 percent) were installed based on the pedestrian volume warrant (3). Also, a majority of the available studies that reviewed the pedestrian volume warrant recommended or suggested warrants that were much lower (easier for a signal to be justified) than the MUTCD warrant in terms of required numbers and duration of pedestrian volumes.

To gain further insight into the reasonableness of the MUTCD pedestrian volume warrant, an analysis was conducted of the daily pedestrian volumes that would be required to warrant a pedestrian signal. Pedestrian and traffic volume data were collected from 388 locations from Chicago, Richmond, and Detroit. At each of the sites, 12-h pedestrian counts were obtained from the local agencies. Next, a computer program was used to develop the distribution of the pedestrian volumes from the 1st highest hour to the 12th highest hour.

The highest hourly pedestrian volume (in percent) was found for each location (regardless of when that hour occurred), and the average of the 388 highest hourly pedestrian volumes was 16.5 percent (of the 12-h total volume). The average of the second highest hourly volume was 13.3 percent (of the 12-h volume), and so on, as shown in Table 1. By using data from 24-h pedestrian counts from Seattle (7), it was found that the peak 12-h pedestrian volume (7:00 a.m. to 7:00 p.m.) represented 86 percent of the 24-h pedestrian volume. The percentage volumes were then adjusted to the percentage of the 24-h volumes for the central business district (CBD), the outlying business district (OBD) and fringe areas, residential areas, and all locations combined (Table 1).

It can be seen that, for an average intersection, the eighth highest hourly pedestrian volume would represent about 5.5 percent of the 24-h pedestrian volume. Also, the cumulative total of the highest 8 h of pedestrian volume represents about 70.5 percent (14.2 + 11.4 + ... + 5.5) of the 24-h total (Table 2). A plot of the distribution of pedestrian volume from the 1st to the 12th highest volume hours is shown in Figure 1. One must consider the following information in order to assess the implication of the MUTCD warrant:

Table 1. Distribution of pedestrian volume by the 12 highest hourly volumes.

Hour	CBD Locations (%) (n = 43)		OBD and Fringe Locations (%) (n = 77)		Residential Locations (%) (n = 268)		All Locations (%) (n = 388)	
	12 h	24 h	12 h	24 h	12 h	24 h	12 h	24 h
Highest	18.6	16.0	16.0	13.8	16.4	14.1	16.5	14.2
2nd	14.7	12.6	13.1	11.2	13.2	11.4	13.3	11.4
3rd	11.9	10.2	11.2	9.6	11.2	9.6	11.3	9.7
4th	9.7	8.3	9.8	8.4	9.9	8.5	9.8	8.4
5th	8.8	7.6	8.9	7.7	8.9	7.7	8.9	7.7
6th	7.9	6.8	8.2	7.1	7.9	6.8	8.6	7.4
7th	6.8	5.8	7.3	6.3	7.2	6.2	7.2	6.2
8th	6.0	5.2	6.6	5.7	6.4	5.5	6.4	5.5
9th	5.2	4.5	5.9	5.1	5.8	5.0	5.7	4.9
10th	4.5	3.9	5.3	4.5	5.1	4.4	5.0	4.3
11th	3.6	3.1	4.3	3.7	4.4	3.8	4.3	3.7
12th	2.3	2.0	3.4	2.9	3.6	3.0	3.0	2.6
Total	100.0	86[a]	100.0	86[a]	100.0	86[a]	100.0	86[a]

Note: CBD = central business district, and OBD = outlying business district.

[a]The remaining 14 percent of the daily pedestrian volume occurs during nighttime hours (between 7:00 p.m. and 7:00 a.m.).

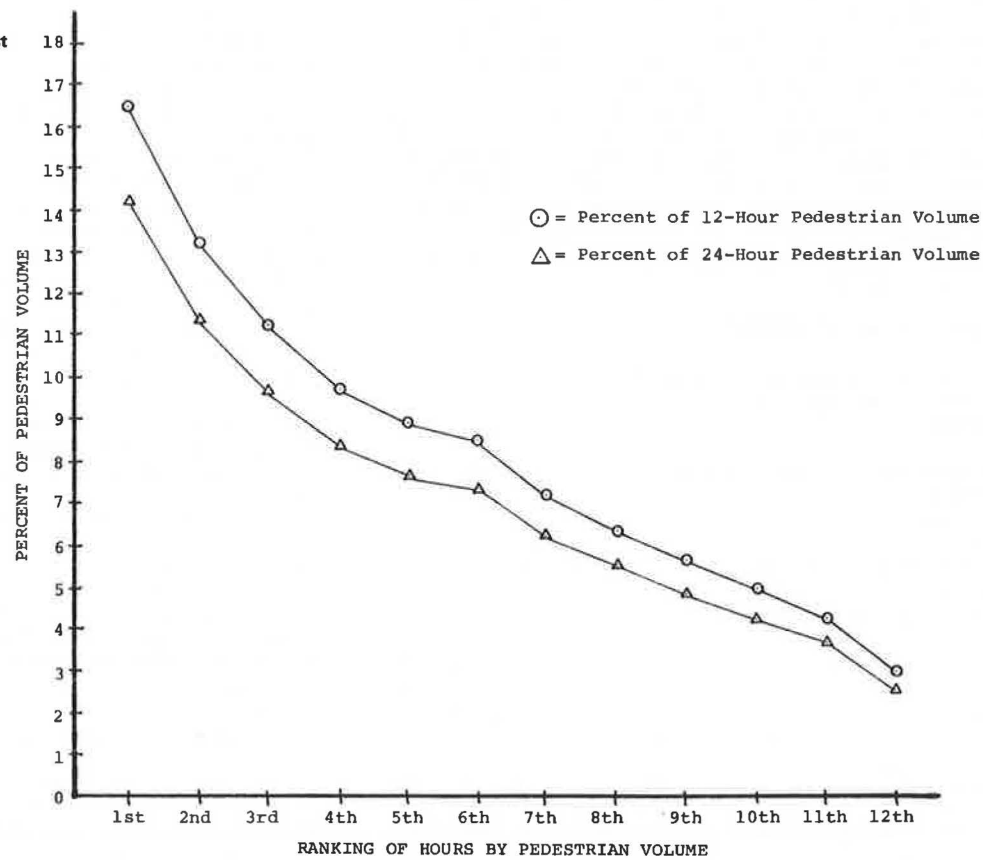
Table 2. Summary of minimum hourly volume required.

Hour[a]	Daily 24-h Pedestrian Volume (%)	Minimum Hourly Volume Required to Meet Pedestrian Volume Warrant	Hour[a]	Daily 24-h Pedestrian Volume (%)	Minimum Hourly Volume Required to Meet Pedestrian Volume Warrant
Highest	14.2	387	8th	5.5	150
2nd	11.4	311	9th	4.9	134
3rd	9.7	264	10th	4.3	117
4th	8.4	229	11th	3.7	101
5th	7.7	210	12th	2.6	71
6th	7.4	202	13th-24th	14.0	382
7th	6.2	169	Total[b]		2727

Note: The minimum hourly volume for each row was based on the control total of 150 hourly pedestrians comprising 5.5 percent of the total during the 8th highest hour.

[a]Column gives the Xth highest hourly volume for the hours of the day.
[b]Total of the 24-h volume.

Figure 1. Distribution of pedestrian volume by hour for the first 12 highest hourly volumes.



1. A minimum of 150 pedestrians/h are required on the highest volume approach (which is only one leg of an intersection) for any 8 h of an average day. Thus, the first eight highest hours of an average day must each have at least 150 pedestrians/h, even though that eighth highest hour only represents 5.5 percent of the daily (24-h) pedestrian volume.

2. In order to meet the MUTCD minimum pedestrian volume warrant (150 pedestrians/h for the eighth hour), the pedestrian volumes that correspond to the first seven highest hours can also be computed based on the pedestrian volume distribution given above. For example, if the eighth highest hour requires 150 pedestrians and corresponds to 5.5 percent of the daily traffic, the pedestrian volume for the 1st highest hour (for an urban intersection with an average volume distribution) can be computed as follows:

$$(14.2 \text{ percent}/5.5 \text{ percent}) \times 150 = 387 \text{ pedestrians in the 1st highest hour} \quad (1)$$

3. Likewise, for the second highest hour (11.4 percent of the daily volume) of the day, the volume is calculated as 311 pedestrians/h. For each of the other time periods, the hourly pedestrian volumes can be computed in a similar fashion. As Table 2 shows, a minimum of about 2727 pedestrians/day is required on the highest volume approach in order to satisfy the minimum pedestrian volume warrant (assuming an average hourly distribution of pedestrian volumes).

4. The next step is to equate the 2727 pedestrians on the highest volume approach to the equivalent total pedestrian volume on all four approaches for an average four-legged intersection. If pedestrian volumes crossing all four approaches were equal, then 25 percent of the pedestrian volume would cross each leg. However, such uniform crossing volumes do not usually exist in the real world. An analysis was conducted of 101 intersections (selected at random from Chicago and Washington, D.C.) to determine what percentage of the volume actually corresponds to the highest-volume leg for a typical four-legged intersection. By computing the percentage of the first, second, third, and fourth highest legs (based on pedestrian volume), the following percentages were found:

Leg Volumes	Mean	Standard Deviation	Percent
Highest	0.360	0.079	36.0
2nd	0.265	0.037	26.5
3rd	0.214	0.0396	21.4
4th	0.161	0.0425	16.1
Total	1.00		

Based on these percentages, the highest-volume crossing represents about 36 percent of the total intersection volume. Note that the standard deviation of each average value is quite low, which indicates low deviation from the mean. Therefore, it is possible to convert the minimum required daily volume of 2727 for the highest-volume leg to an equivalent total intersection volume for an average four-legged intersection, as follows:

$$2727/0.36 \text{ percent} = 7575 \text{ pedestrians/day (all four approaches of a four-legged intersection)} \quad (2)$$

The equivalent pedestrian volume for a three-legged intersection would be less than 7575. For a midblock crossing, the previously calculated value of 2727 would be the expected minimum daily pedestrian volume that corresponds to the MUTCD minimum pedestrian volume warrant.

The above analysis was conducted to illustrate the high daily volume of pedestrians (about 7600 at

Figure 2. Distribution of pedestrian volume by time of day.

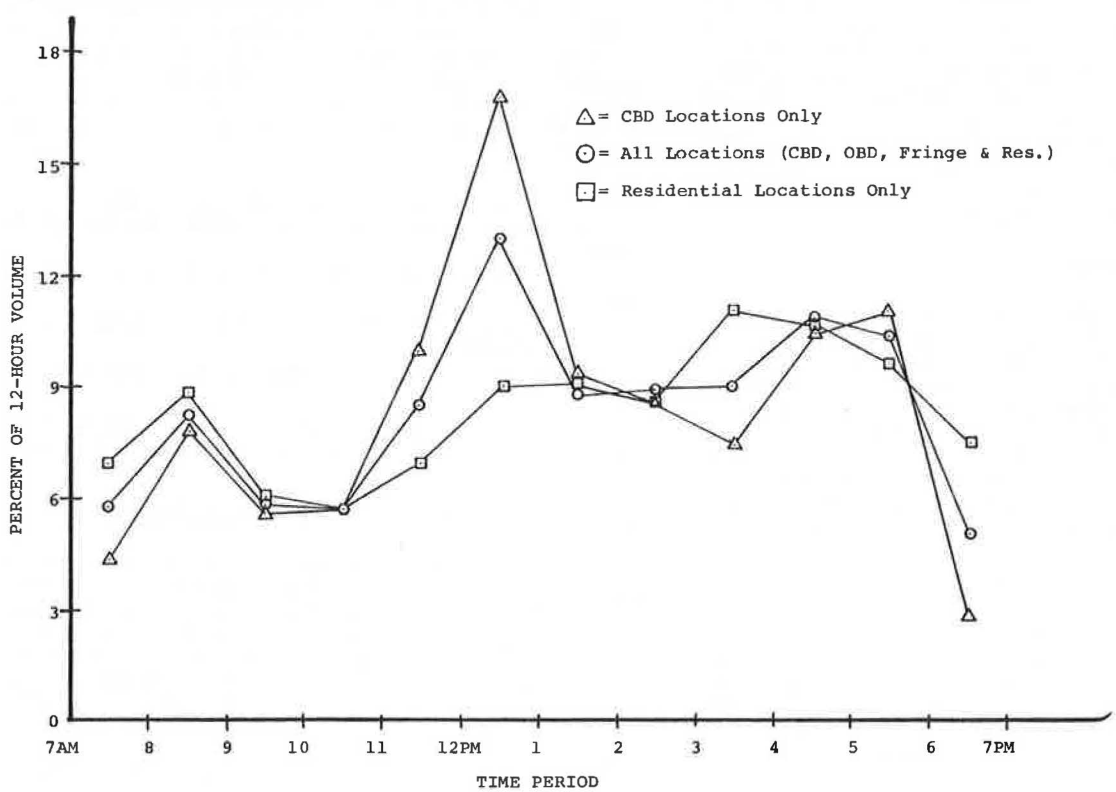


Table 3. Number of intersections meeting various vehicle and pedestrian volume criteria for different data-collection periods.

		Locations Meeting Vehicle Warrant		Locations Meeting Pedestrian Volume Warrant											
				60 per Hour								100 per Hour			
				1 h		2 h		4 h		8 h		1 h		2 h	
City	No. of Intersections	No.	Percent	No.	Percent	No.	Percent	No.	Percent	No.	Percent	No.	Percent	No.	Percent
Chicago	236	212	90	217	92	202	86	174	74	136	58	176	75	161	68
Washington	186	143	77	126	68	112	60	80	43	57	31	97	52	81	44
Total	422	355	84	343	81	314	74	254	60	193	46	273	65	242	57

a typical four-legged intersection) that is necessary in order for the minimum pedestrian volume to be met. Such high volumes are quite unrealistic, except in a very small number of locations (such as in large urban areas). A plot of the average pedestrian volumes (in percent) by time of day is shown in Figure 2, as determined from the data base.

To further test the MUTCD warrant for appropriateness and reasonableness, an analysis was conducted to determine the percentage of the traffic-signalized locations that would meet the 8-h MUTCD pedestrian volume warrant in two large urban areas. Chicago and Washington were chosen, since 10-12 h of pedestrian volume data were readily available at intersections in those cities. Of 422 intersections chosen in the two cities, 355 (84 percent) had sufficient vehicle volumes (600/h for 8 h), but only 34 (8 percent) had sufficient pedestrian volumes (150 or more on the highest-volume crosswalk for any 8 h) to meet the warrant. An additional 78 intersections (19 percent) could have met a 4-h pedestrian volume warrant (150/h on highest-volume approach for at least 4 h). A total of 156 of the signalized intersections (37 percent) had sufficient pedestrian volumes for at least 1 h/day. A summary of the data is given in Table 3. It appears that virtually all of the signals in the sample were probably installed based on other signal warrants.

Based on all the available information discussed above, it was determined that the pedestrian volume requirements (150/h on the highest-volume approach for each of 8 h) of the MUTCD minimum pedestrian volume warrant is unrealistically high. It is not appropriate for most cities and should be revised to allow for signal installations at locations with daily pedestrian volumes considerably below the current high requirements. Thus, the MUTCD warrant was rated as poor based on criterion 1.

Criteria 2: Complexity

The pedestrian volume warrant was next evaluated based on criterion 2, which involves the complexity of using a warrant. The warrant is applied by simply reviewing the hourly volumes of pedestrians and vehicles and determining whether eight of those hours meet the criteria. An adjustment of 70 percent is made in these minimums for average traffic speeds more than 40 mph. This is a relatively uncomplicated procedure to use, so the MUTCD minimum pedestrian volume warrant was rated as good based on criterion 2.

Criteria 3: Data Requirements

The data requirements (criterion 3) are somewhat difficult to meet for most cities. In order to find 8 h of volumes that meet the warrants, a city traffic engineer may need to collect volume counts for 8-12 h in a single day, since the peak 8 h may not always be known until the data have been collected. Of the more than 70 major U.S. cities contacted, only Detroit and Chicago were each found to routinely conduct 10- to 12-h pedestrian volume counts. Also, in Washington, D.C., 10-h pedestrian volume counts were available at most signalized intersections. Richmond had more than one hundred 12-h counts, and Seattle had collected some 24-h sidewalk counts with a mechanical counter a few years ago, although peak-hour pedestrian counts are more common. A few cities collected occasional manual counts of 1-3 h. Except for those cities, most of the other cities contacted collected little or no pedestrian volume data.

Based on these findings, it is not realistic to expect cities to use their limited personnel to collect large amounts of additional data in order to use a signal warrant (particularly in the current financial situation, when many city and state agencies are forced to reduce their existing staffs). Therefore, a poor rating was assigned to the MUTCD warrant for criterion 3.

Criteria 4: Flexibility

Criterion 4 involves the flexibility of the warrant in accounting for a range of highway and traffic conditions. The current warrant allows a 70 percent adjustment in the minimum criteria for high-speed locations (greater than 40 mph) or small towns (less than 10 000 population). Also, the minimum traffic volume is 1000 vehicles/h instead of 600 vehicles/h if a raised median exists. However, except for this possible one-time adjustment of 70 percent, the warrant is not adequately sensitive to gaps in traffic or to the following related traffic and highway variables:

1. Traffic speed (i.e., 25 versus 35 mph),
2. Street width (i.e., undivided streets of 20 versus 50 ft),
3. Vehicle volumes (i.e., volumes of 700 versus 2000/h),
4. Vehicle arrival rates (i.e., random versus traffic queues), and
5. Pedestrian walking speeds (2.5 versus 4 ft/s).

Therefore, the MUTCD minimum pedestrian volume warrant was rated as fair/poor according to criterion 4 (flexibility).

Criteria 5: Acceptability by Traffic Engineers

Criterion 5 is the acceptability of the warrant by practicing traffic engineers in the United States. As discussed previously, the current MUTCD minimum pedestrian volume warrant fares poorly in the opinion of many traffic engineers in the United States (based on discussions with traffic engineers in numerous large cities) due to its unrealistically high required pedestrian volume and large amount of required data.

In summary, the following represent the ratings

				150 per Hour							
4 h		8 h		1 h		2 h		4 h		8 h	
No.	Percent	No.	Percent	No.	Percent	No.	Percent	No.	Percent	No.	Percent
126	53	83	35	89	38	63	27	36	15	12	5
61	33	36	19	67	36	56	30	42	23	22	12
187	44	119	28	156	37	119	28	78	19	34	8

of the MUTCD minimum pedestrian volume warrant according to the five criteria:

Number	Description	Rating
1	Appropriateness and reasonableness	Poor
2	Complexity	Good
3	Data requirements	Poor
4	Flexibility	Fair/poor
5	Acceptability by traffic engineers	Poor

DEVELOPMENT OF REVISED WARRANT

Based on the review of various warrants in North America and abroad, a number of different concepts were identified, including those that are based on minimum pedestrian volume, delay, weighting of pedestrians with vehicular traffic, and others. It was felt that a warrant based on a minimum volume of pedestrians for a specified period and that conforms to either a minimum delay per pedestrian or a maximum number of adequate gaps per time (1-h, 4-h, etc.) provides the best approach for a revised warrant. With this in mind, development of a revised warrant must take into account the following considerations:

1. Duration of required time,
2. Number of legs for warrant,
3. Minimum pedestrian requirement, and
4. Criteria for gaps or pedestrian delay.

Duration of Required Time

The duration of required time should be somewhere between 1 and 4 h, since less than 1 h is likely to give erroneous results, and collection of pedestrian volume data is simply not practical for most cities for more than 4 h/site. The use of several warrants for several time periods may also allow for more widespread application of the warrant. For example, a signal could be warranted based on either a 1-h warrant, a 2-h warrant, or a 4-h warrant. Locations could warrant a signal based on one high peak hour per day or on lower pedestrian volumes that occur during 4 h (i.e., one morning peak hour, one noon peak hour, and two afternoon peak hours). Based on known distributions of pedestrian volumes by hour, it would be quite simple to develop equivalent pedestrian volume levels for any time duration, as discussed earlier. The requirement of pedestrians per hour would be higher for the 1-h warrant than the 2-h warrant.

Number of Legs for Warrant

The next issue involves the number of intersection legs that should be specified as part of the warrant. Of all the studies reviewed, the MUTCD warrant is the only one that requires that the pedestrian volume be on the highest volume approach, which can cause problems. For example, assume that

Table 4. Summary of existing and recommended minimum pedestrian volumes and data-collection periods.

Source	Minimum Pedestrians per Hour	Time Period	Equivalent Pedestrian Volumes per Day[a]
Box (2)	60	Two at 30 min	420
Canada (6)	60	4 h	710
NCHRP (3)			
King (4)	100	4 h	1190
MUTCD (1)	150	8 h	2272[b]

[a]Based on pedestrian volume distributions from the 1st to the 12th highest hours of pedestrian volume; volumes are rounded to the nearest 10.
[b]At midblock or on one intersection leg.

intersection A has 140 pedestrians/h on each of four approaches during the eighth highest hour of an average day. Location B has 155 pedestrians/h on one approach and 20/h on each of the other two approaches. Location A has a higher traffic volume, but both intersections have traffic volumes greater than 600/h for 8 h. In all, intersection A has 560 pedestrians/h compared with 215/h on intersection B. However, intersection B meets the MUTCD warrant for a traffic signal (with 215 pedestrians/h), but intersection A does not (with 560/h).

This example may be exaggerated to illustrate a point, but this high requirement for volumes on one intersection approach is one of the problems of the current MUTCD minimum pedestrian volume warrant. It is therefore recommended that the warrant should be in terms of pedestrians crossing the highest-volume street (or crossing a midblock location).

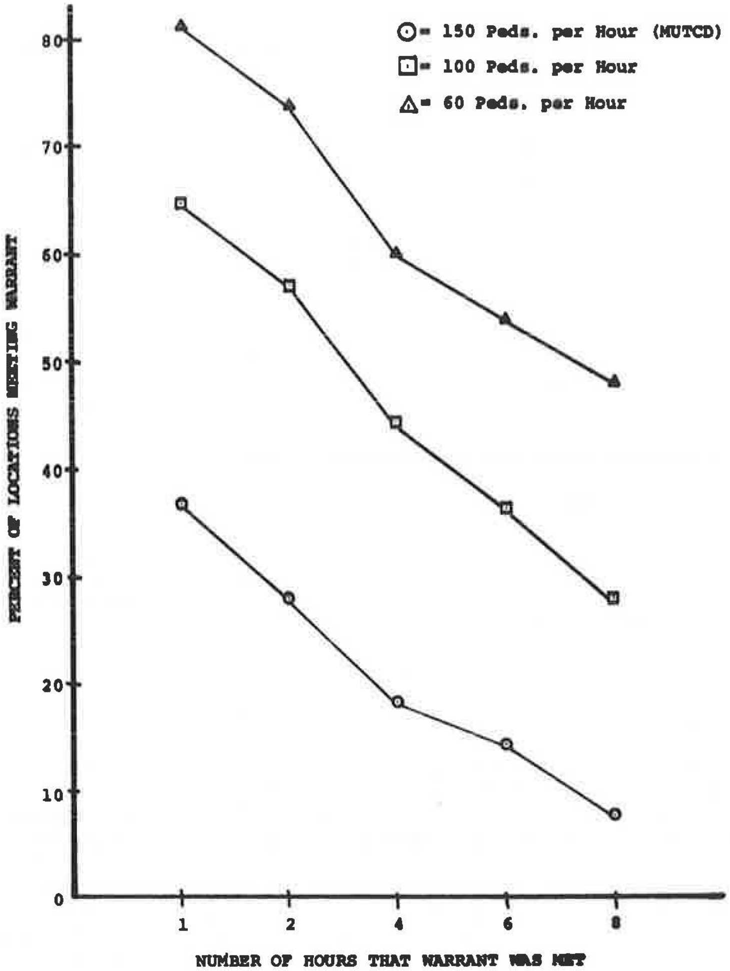
Minimum Pedestrian Requirement

The minimum required pedestrian volume was the next issue that was addressed. Some of the existing or commonly recommended minimum pedestrian volumes and time periods are given in Table 4. In order to further review the consequences of these various pedestrian warrants, they were applied to a sample of 388 traffic-signalized intersections in Chicago and Washington (where 10 h or more of pedestrian volume data were available). Each location was tested to see how many hours that it would meet each of the pedestrian volume criterion:

1. 60 pedestrians/h (major street),
2. 100 pedestrians/h (major street), and
3. 150 pedestrians/h (highest-volume leg), as per the MUTCD warrant

The results are illustrated in Figure 3, which shows the percentage of the intersections that meet various pedestrian volume criteria for various hours in a day. For example, the MUTCD warrant was met for 1 h or more by about 38 percent of the locations, but was met for 8 h by only about 5 percent of the locations. The warrant of 60 pedestrians/h was met by at least 1 h/day by more than 80 percent

Figure 3. Percentage of locations meeting various pedestrian volume warrants (422 signalized intersections in Chicago and Washington).



of the locations and by at least 4 h for more than 60 percent of the locations. These percentages, of course, are for locations with mostly moderate to high volumes of traffic and pedestrians with existing traffic signals. Therefore, the percentage meeting the warrants would be much lower for a random sample of unsignalized locations.

The purpose of this illustration is to show the relative effect of the length of time and hourly pedestrian volume criteria on the number of traffic signals that would meet various warrants. Note that the percentage of locations meeting any pedestrian volume level decreases drastically as the required time period is increased (high negative slope of the curves). The vertical difference between curves illustrates the effect of different pedestrian hourly volume criteria on the percentage of locations that may satisfy a particular warrant.

To add further insight into an appropriate pedestrian volume criterion, a branching analysis was conducted on 1289 signalized intersections to determine what traffic and roadway variables explain the most variation in pedestrian accident experience. The reader is referred to the work of Zegeer and others (8) for an in-depth analysis of pedestrian accident data. Also, it was hoped that the analysis would provide insights on the traffic and geometric factors that are important in pedestrian accident experience.

The branching program was run by using the Statistical Analysis System (SAS) program package. The program looks for the dichotomous split on the predictor variable (i.e., pedestrian volume, traffic volume, street width, etc.) that best predicts the dependent variable (i.e., pedestrian accidents). The program operates under the principle of least squares and subdivides the data set into mutually exclusive subgroups (9).

The results of the branching analysis showed the following conclusions (8):

1. Pedestrian volume is the variable that by far explains the greatest amount of variation in pedestrian accidents than any other single variable (14.9 percent of variance explained).
2. After trying several groupings of pedestrian volume, the breakpoint occurs for a pedestrian average daily traffic (ADT) level of 1200. In fact, for the 609 locations with pedestrian ADT less than 1200, the mean pedestrian accidents (per location per year) was 0.178 compared with 0.376 for locations with more than 1200 pedestrian ADT.
3. The three variables that were most important in explaining the variation in pedestrian accidents (in order of importance) were pedestrian volume, intersection volume, and intersection operation (one-way and two-way streets).
4. Other variables that were found to be of some importance in explaining pedestrian accidents included bus operation, percentage of vehicle turns, intersection design, area type (CBD, OBD, fringe, or residential), and street approach width.
5. In all, 36.6 percent of the variance in pedestrian accidents was explained by the variables that were included in the analysis.
6. Although all intersections in the analysis

had a traffic signal, the presence or absence of a pedestrian signal indication had no significant effect on pedestrian accident experience.

It should be mentioned that the 1289 intersections in the analysis had traffic signals, so the breakpoint of 1200 pedestrians/day from this analysis may not necessarily be the exact same breakpoint for pedestrian accidents at nonsignalized intersections. If one assumes that the addition of a traffic signal improves pedestrian safety (due to creating artificial gaps in traffic for pedestrians to cross), then the critical breakpoint for unsignalized intersections would logically be something less than 1200 pedestrians/day. Thus, a value of 1200 would be a conservatively high value. Many might argue, however, that in areas of poor signal compliance, the addition of a traffic signal could actually reduce pedestrian safety due to the high incidence of pedestrian and motorist signal violations. Obviously, it would be very difficult to define the optimal pedestrian breakpoint value for all roadway situations, but 1200 pedestrians/day may be a reasonable approximation based on available data.

An intersection pedestrian ADT of 1200 at four-legged intersections corresponds to a pedestrian volume of 750 crossing the major street (two highest volume legs) based on 62.5 percent (36 + 26.5 percent) of pedestrians crossing the highest volume legs. Based on hourly pedestrian distributions, this would convert to the following volumes for the first, second, and fourth highest hourly volume:

Volume Period	Equivalent Pedestrian ADT (nearest 10 pedestrians)
ADT	750
1st highest hour	110
2nd highest hour	90
4th highest hour	60

The corresponding minimum pedestrian volume for the fourth highest hour corresponds to the Canadian pedestrian volume criterion of 60 pedestrians/h for 4 h. It would be stricter than the Box warrant, which requires 60 pedestrians/h for each of two 30-min periods (2). The pedestrian volume criterion would be less strict than the 100 pedestrians/h for 4 h as required by King (4) and NCHRP (3). For shorter time periods of 2 or 1 h, pedestrian volumes of 90 and 110/h would be required, respectively.

Criteria for Gaps or Pedestrian Delay

A pedestrian signal warrant must consider not only pedestrian volumes but also the time available for pedestrians to cross the street (i.e., available gaps in traffic). The number of adequate gaps in traffic is directly related to various combinations of traffic speed, traffic volume, and traffic arrival distribution. Further, the number and duration of gaps needed for safe pedestrian crossings is a function of street width, pedestrian walking speed, and pedestrian volume (and/or pedestrian group size). The number of adequate gaps in traffic can be quickly and easily determined based on field surveys (or other methods), as described by the Institute of Transportation Engineers (ITE) (10).

A gap-based warrant of less than 60 acceptable gaps/h is currently the school crossing warrant prescribed in MUTCD. This gap-based criterion actually accounts for site-specific combinations of street width, pedestrian walking speed, vehicle speed, traffic volume, and traffic arrival distribution. It is therefore conceptually appealing as well as practical to use along with a pedestrian volume criterion for a limited time period per site.

In the absence of additional objective information, the minimum pedestrian volume criterion was selected as follows. The minimum required pedestrian volume crossing the major street per hour for an average day must be (a) 60 or more for each of any 4 h, or (b) 90 or more for each of any 2 h, or (c) 110 or more during the peak hour.

In addition to a minimum pedestrian volume of those stated above, the number of adequate gaps in the traffic stream (during the same periods as above) should be less than 60/h during the same period when the pedestrian volume criterion is satisfied.

CONCLUSIONS AND RECOMMENDATIONS

The purpose of this paper was to evaluate existing MUTCD warrants related to pedestrian signals and actuation devices by examining the existing literature and operational practice. If the existing MUTCD warrants were found to be inadequate, new warrants were to be developed.

The MUTCD minimum pedestrian volume warrant was found to be unacceptable in terms of its (a) appropriateness to real-world conditions, (b) data requirements, (c) flexibility, and (d) acceptability to practicing traffic engineers.

Based on all available literature and on existing pedestrian signal warrants, a number of different warrant concepts were examined. The preferable concept was found to be one that incorporates a minimum pedestrian volume per hour and a number of adequate gaps per hour. Based on an in-depth study of hourly pedestrian volume distributions and an analysis of data at 1297 intersections, the following minimum pedestrian volume warrant was recommended.

A traffic signal is warranted if

1. The minimum pedestrian volume crossing the major street equals or exceeds (a) 60/h for each of any 4 h, or (b) 90/h for each of any 2 h, or (c) 110/h during the peak hour;
2. The number of adequate gaps in the traffic stream on the major street is less than 60/h during the same period when the pedestrian volume criterion is satisfied; and
3. When a traffic signal is warranted based on criteria 1 and 2 above, pedestrian indications should be used; the warrant is for either midblock locations or for intersections.

The recommended minimum pedestrian volume warrant has some similarities to warrants recommended by Box (2) and the Canadian warrant (6).

ACKNOWLEDGMENT

The opinions and viewpoints expressed in this paper are ours and do not necessarily reflect the viewpoints, programs, or policies of the U.S. Department of Transportation or any state or local agency.

Discussion

K. Todd

When signal warrants first came into being half a century ago, their purpose was twofold: (a) to give traffic engineers satisfactory guidelines that justified the decision to install or not to install a signal on the basis of operational and safety considerations, and (b) to convince the public and elected officials of the reasonableness of such a decision.

Installation of pedestrian and other signals is often preceded by requests from the public and by hearings and discussions at public meetings. Statistical analyses and mathematical computations do not easily convince pedestrians who are asking for the protection of a signal, nor do they convince motorists who fear additional delays. In consequence, a further criterion--acceptability by the public--might be added to the five criteria listed by Zegeer, Khasnabis, and Fegan.

From the public's point of view, the proposed warrant may be queried on the following issues:

1. A pedestrian signal is warranted where the minimum pedestrian volume crossing the major street equals or exceeds 110 during the peak hour. The stipulation gives rise to the following question. If fewer than 110 pedestrians (100 of them, for example) are expected to cross the major street safely and without undue delay during the peak hour without the help of a signal, why would a signal ever be warranted in circumstances when pedestrian volumes are even lower?
2. If pedestrians are refused a signal unless the number of adequate gaps in traffic on the major street is less than 60/h, why do they have to wait at a signal during periods of the day when the number of such gaps is larger?
3. The pedestrian signal is warranted on the assumption that there are fewer than 60 adequate gaps/h in traffic on the major road and that the vehicle volume on the minor road is too low to warrant a signal. If 60 pedestrians/h, which represent 36 percent of the total pedestrian volume, justify the installation of a signal to help them across the highest-volume leg of the major road, the two low-volume legs on the minor road would likely be crossed by 36 (21.4 percent) and 27 (16.1 percent) pedestrians, respectively, during the same hour. The question may be asked why pedestrians wishing to cross these low-volume legs have to be controlled by a signal, seeing that neither the vehicle volumes on the minor road legs nor the pedestrian volumes warrant it.

These issues are somewhat wider than those addressed by the authors and may perhaps be dealt with more appropriately by the traffic engineering profession as a whole. So long as these questions remain unanswered, the warrant will suffer from a credibility problem.

Authors' Closure

We appreciate Todd's comments and his interest in our paper. The fact that his comments are from the "public's point of view" is refreshing, and we will attempt to address each of the questions raised.

First of all, Todd suggests that acceptability by the public might be added as a criterion in selecting an appropriate pedestrain signal warrant. Although we agree that citizen input is often useful in many transportation-related areas, the average citizen does not have an adequate understanding of how traffic signals affect the safety and operations of pedestrians and vehicles. In terms of public preferences, we have found that pedestrians are likely to favor the installation of traffic signals to aid their crossings, while motorists would prefer not to be stopped at a traffic signal due to pedestrians, particularly since pedestrians are unjustly treated as second-class citizens by many motorists.

Todd's next question involves the minimum pedestrian volume criterion of 110 pedestrians/h on the major street. We believe that some minimum pedestrian volume level is needed as part of the warrant, and our analyses indicated that 1200 pedestrians/day was an appropriate level. This daily volume translates into 110 pedestrians/h during the peak hour based on 24-h distributions of pedestrian volume. When Todd asks why a signal could be warranted for lower volumes, we assume he is referring to our 2-h volume criterion (90 or more pedestrians/h) and our 4-h criterion (60 or more pedestrians/h). It should be understood that all three criteria correspond to approximately the same daily (24-h) pedestrian crossings. Therefore, public agencies can use our warrant with as little as 1 h of data or as much as 4 h of data.

The next two issues pertain to the number of adequate gaps. Because traffic and pedestrian volumes commonly fluctuate greatly throughout the day, the development of any type of signal warrant must consider some specified time period due to practical considerations in applying the warrant. Thus, traffic signal warrants involve criteria for some critical or high-volume period when safety and operational problems are most prevalent (i.e., peak 8 h of an average day). For signalized intersections with low nighttime traffic, a flashing amber (caution) signal may be used on the main street with a flashing red (stop and proceed when clear) signal on the side street.

Regarding the need to stop pedestrians and traffic on the side street (in Todd's third major point), the installation of a traffic signal on the main street implies that traffic will be given a green interval for probably most of the cycle. Thus, stop control (i.e., a red light) is needed for side street vehicles to prevent large numbers of right-angle collisions with main street through vehicles. Also, pedestrian street crossings should be guided by the signal control (preferably WALK/DON'T WALK signals, if present) for obvious safety reasons.

With regard to warrant credibility, we have found that the current MUTCD minimum pedestrian volume warrant suffers from a severe credibility problem. At a recent TRB conference session, one city traffic engineer stated that the existing minimum pedestrian volume warrant is so unrealistically high that it casts a shadow of doubt on the other signal warrants, even though most of the other warrants seem to be reasonable for most real-world conditions. This comment was typical of comments we have received from traffic engineers throughout the country.

Based on our research efforts, we believe our proposed warrant to be superior to the current warrant. The similarities of our warrant to the Canadian warrant (60 pedestrians/h for 4 h), although coincidental, are encouraging. Also, the concept of a minimum pedestrian volume criterion and consideration of the number of acceptable gaps is shared by other researchers. Perhaps further research could be useful to determine the number of new traffic signal installations (and corresponding costs) that would result from the adoption of our warrant. The effect of such new signal installations on safety and operations under various conditions should also be determined for signals installed based on a revised warrant.

One other important point to consider is that the current warrant is so unrealistically high that little possibility exists in many cities for warranting a traffic signal based on pedestrian considerations, as discussed in our paper. This indicates that the needs of pedestrians may be unjustly ignored, and that pedestrians are too often considered merely as a hindrance to traffic flow in our society. We hope that the results of our study, as well as other related studies, will be helpful in the modification of the minimum pedestrian volume warrant.

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Measurements and Analysis of Degradation of Freight Car Reflectors in Revenue Service

JAMES L. POAGE AND JOHN B. HOPKINS

Accidents at railroad-highway crossings in which a motor vehicle ran into the side of a train during dawn, dusk, and darkness accounted for 13.6 percent of all fatalities and 21 percent of all injuries in crossing accidents during 1980. A possible remedial action for this problem is to mount retroreflective material on the sides of freight cars that, when illuminated by vehicle headlights, may give an indication of the presence of a train in the crossing. Results of measurements conducted on freight-car-mounted reflectors to provide data on the durability of reflectors in revenue service are presented. Reflective intensity measurements were made on engineer-grade retroreflective sheeting on Canadian freight cars; this material has been installed on the side sills of Canadian freight cars since 1959. Reflective intensity measurements were also made over a six-month period on high-intensity reflective sheeting on 19 Boston and Maine Railroad freight cars. The Canadian reflector measurements on engineer-grade reflectors indicated rapid deterioration in the reflective intensity. Data from tests on the Boston and Maine Railroad strongly indicate that high-intensity reflectors deteriorate in the railroad environment at a similar rate, although the limited time for the high-intensity tests precludes absolute conclusions on high-intensity reflector durability. The rapid rate of degradation in reflective intensity has implications for the size of reflectors that might be mounted on freight cars, the useful life of the reflectors, the importance and scheduling of washing of the reflectors, and the cost and cost-effectiveness of reflectorization. Equations that describe the trade-off between reflector size and washing interval are developed.

Accidents at railroad-highway crossings in which a motor vehicle ran into the side of a train during dawn, dusk, and darkness accounted for 13.6 percent of all fatalities and 21 percent of all injuries in crossing accidents during 1980. A possible remedial action for this problem is to mount retroreflective material on the sides of freight cars that, when illuminated by vehicle headlights, may give an indication of the presence of a train in the crossing. Previous research (1) has addressed the issues of potential benefits and required reflector brightness. However, a major remaining uncertainty concerning this safety measure is the rate at which dirt and age affect the reflectors. This paper presents the results of measurements performed on freight-car-mounted reflectors to provide information on the durability of reflectors on cars in revenue service. The rate of degradation in reflective intensity has implications for the size of reflectors that might be mounted on freight cars, for the useful life of the reflectors, for whether washing of the reflectors would be necessary, and thus for the cost-effectiveness and practicality of reflectorization. This paper also investigates the relation between reflector size and frequency of washing.

OVERVIEW OF REFLECTIVE INTENSITY MEASUREMENTS

Several types of tests of freight car reflector durability are described in this paper. In one test, the reflective intensity of reflectors on 208 Canadian freight cars was measured. Since May 1959, the Canadian Transport Commission (CTC) has required that reflective markings be installed on the side sills of Canadian freight cars. Observations of the visibility of reflectors on Canadian trains at night were also made at three railroad-highway crossings. The tests on Canadian freight cars were conducted jointly by the Transportation Systems Center (TSC) and CTC. The reflectors measured on Canadian freight cars are engineer-grade retroreflective sheeting. In another test, high-intensity retroreflective sheeting was placed on 33 Boston and Maine Railroad (B&M) freight cars during spring and summer 1981. Reflective intensity measurements on 19 of these cars were made during a six-month period.

The reflective intensity measurement tests on Canadian freight cars suggest a rapid decline in reflector reflective intensity to an average of 23 percent of initial value after six months, to 14 percent after one year, and to 5 percent after two years. The night-observation tests also indicate a rapid decline in reflector reflectivity. On at least 61 percent of the Canadian cars observed, reflector reflectivity was rated poor. The test period for measuring reflective intensity on the B&M freight cars was not long enough to develop estimates concerning the long-term wear of high-intensity reflective sheeting on railroad cars. However, the results for the first six months indicate deterioration rates that are similar to those obtained from the Canadian measurements.

CANADIAN REFLECTORIZATION PROGRAM

During the late 1950s, the Canadian Board of Transport Commissioners (BTC) studied railroad-highway crossing data that indicated that a large percentage of accidents in which motor vehicles struck a train occurred at night. BTC concluded that the reflectorization of freight cars might reduce this type of accident. BTC recommended to the Canadian federal cabinet that the Railway Act be amended to permit grants to be made from the Railway Grade Crossing Fund toward the cost of the installation of reflectors. The amount of the grants was established at 80 percent of the cost, which was the same percentage granted for other improvements to public crossings. The amount of the grant cannot exceed $8.00/car.

BTC, and later CTC, have issued several orders, beginning in 1959, that have required four reflectors to be applied to each side of cars of 52 ft or less and six reflectors to each side of cars more than 52 ft in length. All reflectors measured in the tests are Scotchlite Brand Reflective Sheeting manufactured by the Minnesota, Mining, and Manufacturing (3-M) Company of Canada. The reflectors are engineer-grade silver 4-in discs used on Canadian National Railway (CN) cars and 4-in squares on Canadian Pacific Railway (CP) cars.

CTC has from time to time attempted to evaluate the effectiveness of the program. The railways are required to report all accidents that occur at public crossings at grade, and the Railway Transport Committee (RTC) investigates those that involve casualties. However, statistics are not maintained that differentiate between those accidents in which the vehicle ran into the side of a train and those in which the train struck the vehicle.

MEASUREMENT OF REFLECTIVE INTENSITY ON CANADIAN FREIGHT CARS

The reflective intensities of reflectors on 208 freight cars were measured in CN and CP yards near Montreal during the week of October 19, 1981. The measurements were made by using a Gamma Scientific, Inc., model 910F retroreflectometer. This instrument consists of (a) an optical head with an optical system, detector, and light source, and (b) a control unit with readout display, operating controls, and rechargeable battery power supply. The instrument is operated by pressing the optical head against the surface to be measured, which activates the device's light source. The instrument is calibrated against a secondary standard and can make measurements during either day or night. Units of reflective intensity are measured in candela per footcandle per square foot. Reflectivity was measured for reflectors on both sides of 120 cars and on one side of 88 cars. Samples of new reflective sheeting of the type installed on Canadian freight cars were measured and showed an average reflective intensity of 94 cd/footcandle/ft^2.

For the data analysis, cabooses and work cars were excluded because the type of service of work cars and the frequent washing of cabooses provide a different environment for the reflectors than that experienced by typical freight cars. The average of reflective intensity measurements for each of the remaining 195 cars is shown in Figure 1. As can be seen from this figure, the reflective intensity of the reflectors decreases rapidly within a year after installation. The reflective intensity continues to decrease into the second year, when it becomes a relatively constant value of less than 10 cd/footcandle/ft^2.

An exponential curve was fitted to the reflectivity data measurements obtained for reflectors that had been in service for less than 2.5 years. The resulting curve (Figure 2) shows a rapid decline in reflective intensity for reflectors in railroad revenue service, with an average reflective intensity that is 23 percent of the original value after six months, 14 percent after one year, and 5 percent after two years. Figure 2 also shows the 95 percent confidence interval for the curve.

A second method was also used to analyze the reflectivity measurement data. The data were averaged over three-month periods and plotted at six-month intervals (Figure 3). As shown, these averages are similar to the exponential regression curve (Figure 2) and imply the same rapid decline in reflective intensity with age.

After the initial reflectivity measurements, reflectors on 24 freight cars were washed and the measurements were repeated. The average reflectivity of the reflectors on each of the 24 cars before and after washing are given in Table 1. The average reflectivity for cars with reflectors that have the same time in service was calculated and expressed as a percentage of the reflective intensity measured for new reflectors (Table 2).

The data suggest that the reflective intensity of the reflector does increase after washing, as expected. The data also indicate that the reflectors deteriorate in the railroad environment at a rate such that, after three years of service, washing of the reflectors restores less than 25 percent of the original reflectivity.

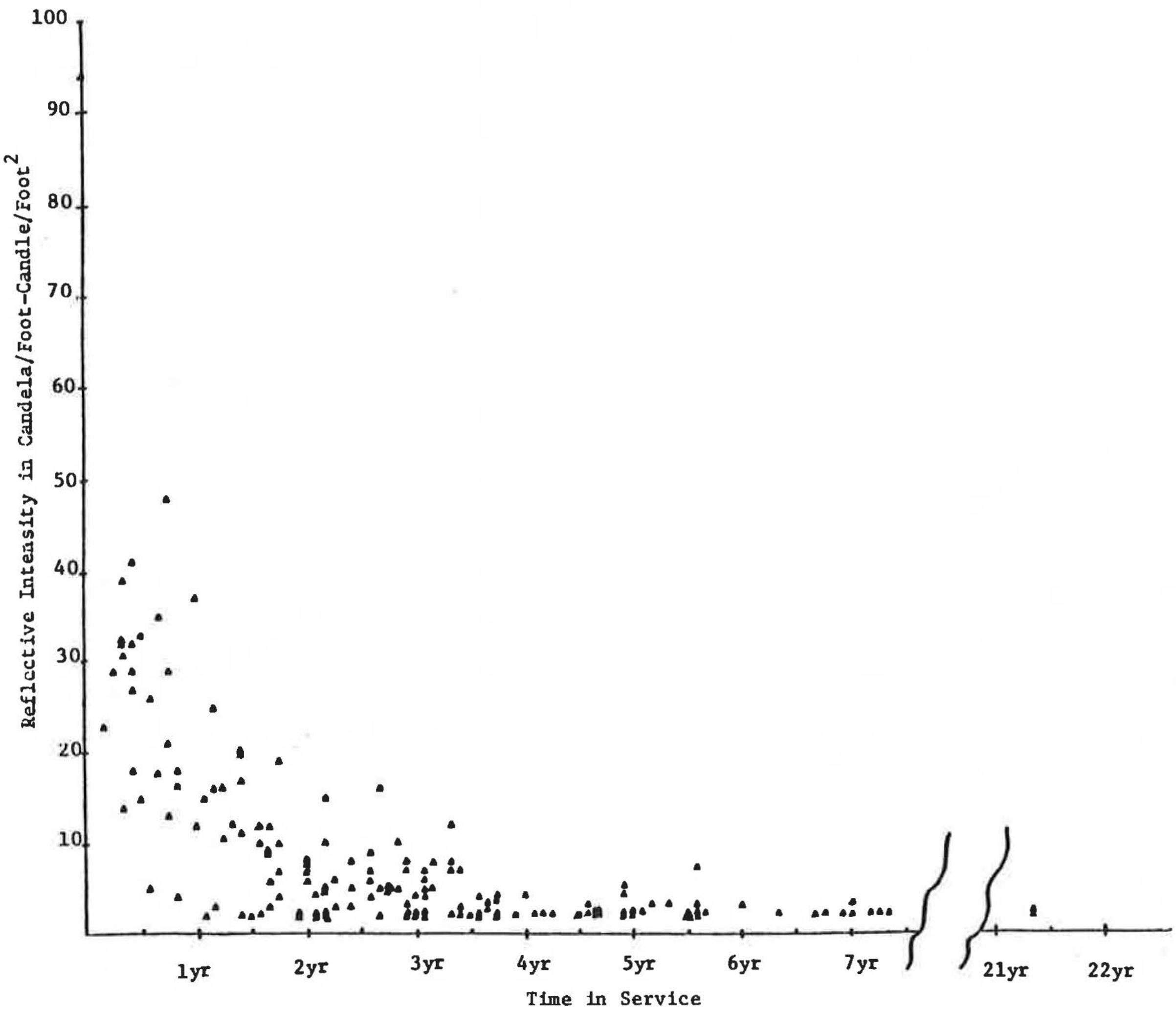
NIGHT OBSERVATION OF REFLECTORS ON CANADIAN FREIGHT CARS

To observe freight car reflector conspicuity under actual railroad operating conditions at railroad-highway crossings, night-observation tests of reflectors mounted on freight cars were made at three Canadian railroad-highway crossings in the vicinity of Montreal during the week of October 19, 1981. The test crossings had minimal automobile traffic, an intersection angle of road and track of 90°, and relatively flat approach grades.

An automobile was parked 300 ft from the crossing such that headlights illuminated the crossing. High beams were used for all tests. An observer sat in the front seat and recorded observations of the visibility of reflectors on each car of the passing trains. A new reflector was posted at the crossing to provide a reference for the observer. An observation of good, fair, or poor was recorded by the observer for each car. A car was rated good if the reflectors were clearly visible, fair if the reflectors were only moderately visible, and poor if barely visible or not visible at all. This test was conducted under the best of conditions, with the observer stationary and anticipating the presence of a train.

The night-observation test results are summarized in Table 3, which gives the percentage of cars with reflectors observed as good, fair, or poor in seven trains with a total of 480 cars. Of the cars ob-

Figure 1. Reflective intensity measurements for each Canadian freight car.



served, 14.2 percent had reflectors with good visibility, 16.7 percent were fair, and 69.1 percent were poor.

Canadian trains typically include freight cars owned by U.S. railroads; these cars usually do not have reflectors. Representatives of CTC, CN, and CP estimated that 20 percent of the cars in Canadian trains are of U.S. ownership. To account for U.S. ownership, results shown in Table 3 were modified to provide values for only Canadian cars. This process results in 17.8 percent of cars having reflectors with good visibility, 20.9 percent with fair visibility, and 61.3 percent with poor visibility.

The second line of data in Table 3 identifies a CN train with 76 cars. The dates on which the cars were built or rebuilt were recorded from markings on the cars after this train entered a nearby classification yard. The reflector visibility rating--good, fair, or poor--is shown in Table 4, along with the built/rebuilt date and car type. Most of the reflectors that were rated as good or fair are less than four years old. These results of the observation of reflectors at night support the measurements of reflective intensity discussed previously in showing a rapid decline in reflective intensity in the first few years.

B&M REFLECTOR TEST

Scotchlite Brand Reflective Sheeting, High-Intensity Grade, was installed on 33 sand-and-gravel hopper cars on the B&M between May through July 1981. Four reflectors, each 4x12 in, were installed on each side of the cars just above the side sill. The material has alternating silver and orange colors, such that each 12-in piece applied to the cars is a composite of both colors. The reflective intensity of the silver portion of the material was measured to be 290 cd/footcandle/ft^2 prior to installation. The B&M sand-and-gravel cars are high-use cars in dedicated service between Boston, and Ossipee, New Hampshire.

During October through December 1981, reflectivity measurements were collected on 19 of the sand-and-gravel hopper cars. The dirt observed on the reflectors was of a sandy, dusty nature, which would be expected from the type of service experienced by the cars. Table 5 gives the average reflector reflective intensity for each car by time in service and the lowest and highest reflector reflective intensity for each car.

The average reflective intensity of reflectors in service for four months was 106 cd/footcandle/ft^2. Reflectors in service for five and six months had average reflective intensities of 55 and 22 cd/footcandle/ft^2, respectively. These data suggest a decline in reflective intensity to 37 percent of the initial reflective intensity after four months in service, to 19 percent after five months, and to 8 percent after six months, as given in the table

Figure 2. Exponential curve fitted to data for reflector reflective intensity measured on Canadian freight cars.

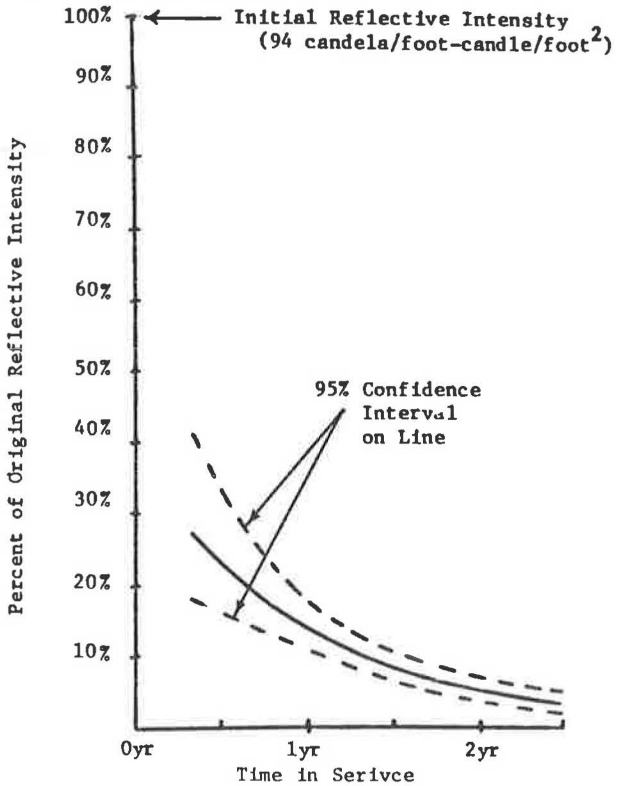


Figure 3. Three-month averages of reflector reflective intensity measured on Canadian freight cars.

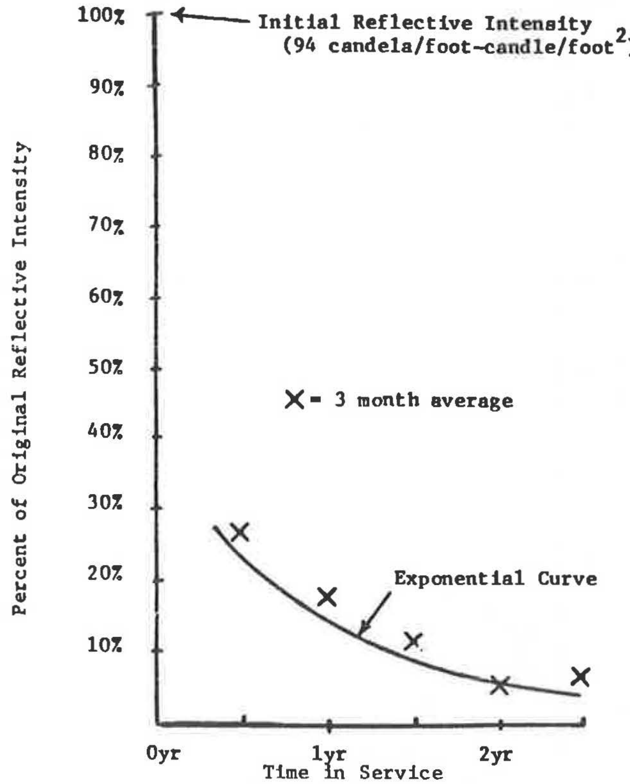


Table 1. Measurement of reflective intensity before and after washing reflectors.

Date Car Built or Rebuilt	Reflectivity (cd/footcandle/ft^2) Before Washing	Reflectivity (cd/footcandle/ft^2) After Washing	Date Car Built or Rebuilt	Reflectivity (cd/footcandle/ft^2) Before Washing	Reflectivity (cd/footcandle/ft^2) After Washing
1981	37	67	1979	11	20
	38	67		9	16
	39	64		9	27
	34	63	1978	6	16
	45	72		4	5
	41	73		8	15
	28	82	1977	5	17
	35	67	1975	2	2
	51	85	1972	3	5
1980	18	55	1969	3	10
	8	28		3	5
	3	14			
	43	66			

Note: Measurements listed are averages of the reflective intensity of all reflectors on each freight car.

Table 2. Reflective intensity of reflectors before and after washing as a percentage of original reflective intensity.

Year Car Built or Rebuilt	No. of Cars Washed by Year	Original Reflective Intensity (%) Before Washing[a]	Original Reflective Intensity (%) After Washing
1981	9	41.2	76.6
1980	4	19.1	42.7
1979	3	10.3	22.3
1978	3	6.4	22.3
1977	1	5.3	18.1
1976	–[b]	–[b]	–[b]
1975	1	2.1	2.1
1974	–[b]	–[b]	–[b]
1973	–[b]	–[b]	–[b]
1972	1	3.2	5.3
1969	2	3.2	8.0

[a]Percentages listed are averages of all reflectors measured by year car was built or rebuilt.
[b]No data.

Table 3. Night observation of reflectors on freight cars.

Item	Railroad	No. of Cars in Train	Rating of Reflector Visibility by Car (%) Good	Fair	Poor
Test date					
10/19	CN	89	8.9	3.4	87.7
	CN	76	15.8	26.3	57.9
10/20	CP	108	18.5	16.7	64.8
10/21	CP	20	15.0	60.0	25.0
	CP	65	13.8	4.6	81.6
	CP	74	14.9	23.0	62.1
	CP	48	10.4	14.6	75.0
Total for all cars		480	14.2	16.7	69.1
Total modified to show Canadian cars only		384	17.8	20.9	61.3

Table 4. Ratings of reflector visibility by age and type of car.

Item	No. of Cars by Observed Reflector Visibility			No. of Cars by Observed Reflector Visibility by Type of Car								Total No. of Cars by Time
				Good		Fair		Poor				
	Good	Fair	Poor	Box	Tank	Box	Tank	Box	Tank	Hopper	Refrigerator	
Date[a]												
1981	4	2	3	1	3	2	-	1	2	-	-	9
1980	6	-	3	4	2	-	-	1	2	-	-	9
1979	1	2	2	1	-	2	-	-	2	-	-	5
1978	1	11	2	-	1	11	-	2	-	-	-	14
1977	-	-	1	-	-	-	-	1	-	-	-	1
1976	-	1	1	-	-	1	-	1	-	-	-	2
1975	-	2	5	-	-	-	2	1	4	-	-	7
1974	-	-	2	-	-	-	-	-	1	1	-	2
1973	-	-	1	-	-	-	-	-	1	-	-	1
1972	-	2	1	-	-	-	2	-	1	-	-	3
1971	-	-	-	-	-	-	-	-	-	-	-	-
1970	-	-	3	-	-	-	-	-	3	-	-	3
1969	-	-	2	-	-	-	-	-	-	-	2	2
1968	-	-	3	-	-	-	-	-	3	-	-	3
1967	-	-	-	-	-	-	-	-	-	-	-	-
1966	-	-	4	-	-	-	-	-	3	-	1	4
1965	-	-	1	-	-	-	-	1	-	-	-	1
1064	-	-	-	-	-	-	-	-	-	-	-	-
1963	-	-	-	-	-	-	-	-	-	-	-	-
1962	-	-	1	-	-	-	-	1	-	-	-	1
1961	-	-	-	-	-	-	-	-	-	-	-	-
1960	-	-	-	-	-	-	-	-	-	-	-	-
1959	-	-	-	-	-	-	-	-	-	-	-	-
<1959	-	-	3	-	-	-	-	2	1	-	-	3
Total	12	20	38	6	6	16	4	11	23	1	3	70
Non-Canada cars	-	-	6	-	-	-	-	2	2	-	2	6
Total	12	20	44	6	6	16	4	13	25	1	5	76

[a]Date cars built or rebuilt.

Table 5. Reflective intensity of silver reflectors.

Time in Service (months)	Avg Reflective Intensity on Car (cd/footcandle/ft^2)	Range of Reflective Intensity on Car (cd/footcandle/ft^2)	
		Low	High
4	196	139	232
	15	2	45
	29	13	42
	163	85	202
	103	36	164
	97	64	127
	221	214	227
	70	33	98
	135	67	168
	117	110	123
	58	22	102
	72	55	89
5	28	19	38
	94	78	119
	44	29	56
6	11	5	17
	19	4	25
	2	2	2
	58	33	87

below (note, the initial reflective intensity of the silver portion of reflectors was measured to be 290 cd/footcandle/ft^2):

Time in Service (months)	No. of Cars Measured	Avg Reflective Intensity (cd/footcandle/ft^2)	Avg Reflective Intensity of Initial Value (%)
4	12	106	37
5	3	55	19
6	4	22	8

For comparison purposes, the reflective intensity, as a percentage of the initial value, is given in the table below for reflectors measured on Canadian cars:

Age (months)	Regressive Analysis: Reflective Intensity of Initial Value (%)	Avg for Cars Measured by Month	
		No.	Reflective Intensity of Initial Value (%)
4	27	5	32
5	25	4	29
6	23	2	26

The decline in the percentage of the initial value with time is given by both the curve developed through a regression analysis (Figure 2) and the average of the reflective intensities measured in each month (Figure 1).

SUMMARY OF FREIGHT CAR REFLECTOR MEASUREMENTS AND OBSERVATIONS

Both the measurement of reflective intensity on Canadian freight cars and the night observation of reflectors on Canadian freight cars suggest a rapid rate of deterioration in the railroad environment. The average reflective intensity measurements made on 208 Canadian freight cars imply that a reflector's reflective intensity is reduced to 23 percent of its initial value after six months in service. After one and two years in service, the reflective intensity is reduced to 14 and 5 percent, respectively, of the initial value. In the night observation of reflectors, 61 percent of the cars were observed to have reflectors that were poor (i.e., barely visible or not visible at all). The reflectors in these tests were engineer-grade reflective sheeting.

An insufficient amount of data and the limited time available for the B&M reflectorization tests prohibit the development of absolute conclusions regarding the durability of high-intensity reflectors in the railroad environment. Also, the measurement of reflectors on B&M gravel cars represents only one type of service environment. However, the data indicate that high-intensity reflectors deteriorate in the railroad environment at a rate similar to that observed of engineer-grade reflectors in use in Canada.

These measurement results have implications for the size of reflectors that would be necessary for freight car reflectorization. A large reflector would compensate for a degradation in reflective intensity in providing a visible warning to motorists. Other ways to compensate for the degradation in reflective intensity are to wash or replace the reflectors periodically. The following analysis investigates some of the relations between reflector size, wash cycles, and replacement cycles.

ANALYTICAL CHARACTERIZATION OF REFLECTOR DEGRADATION

In order to use the measurements described above for the determination of necessary reflector size, it is convenient to express these results in terms of a degradation equation. This is done in a sequence of steps.

First, the reflective intensity deterioration rate for engineer-grade reflectors tested in Canada is examined to separate the deterioration due to dirt alone from that due to age. The dirt deterioration rate obtained for engineer-grade reflectors is then combined with an appropriate aging factor for high-intensity material to determine an overall expected deterioration rate for high-intensity reflectors. In the next section, trade-offs between reflector size, wash interval, and replacement interval for high-intensity reflectors are considered.

The engineer-grade material tested in Canada was found to have an initial reflective intensity of 94 cd/footcandle/ft², which dropped dramatically in the first few months (Figures 1 and 2) and then appeared to diminish exponentially with time. The exponential curve fitted to the data (Figure 2) began with a value of 35.8 cd/footcandle/ft². Letting $R(t)$ stand for reflective intensity and t for time (in years), the reflective intensity at time t, based on a least-squares fit of the Canadian data as shown in Figure 2, is given by the following equation:

$$R(t) = R_o(35.8/94)\exp(-0.9872t) = 0.3809R_o\exp(-0.9872t) \quad (1)$$

where R_o is the initial reflective intensity.

The decay coefficient of -0.9872 combines the effects of dirt accumulation and material aging. In order to determine the effect of dirt alone, the effect of deterioration with age must be quantified. The reflective intensity of engineer-grade material is specified to drop to no less than half its original value in seven years under normal conditions of use, according to a pamphlet from the Traffic Control Materials Division, 3-M Company, St. Paul, Minnesota [pamphlet LH-HIBCB (71.75) MP]. This implies decay at a rate given by

$$R = R_o\exp(-0.099t) \quad \text{(engineer grade; normal conditions)} \quad (2)$$

The specifications are intended for highway traffic-control signs. In this analysis, the assumption is made that the reflectors deteriorate twice as rapidly in a railroad environment. Thus, in railroad use, as in Canada, expected deterioration due to age alone can be described by

$$R = R_o\exp[2 \times (-0.099)]\, t = R_o\exp(-0.198t) \quad \text{(engineer grade; age alone; railroad conditions)} \quad (3)$$

Thus, the Canadian result, which shows a total deterioration time constant of -0.9872, is assumed to be composed of an age effect that contributes -0.198 and a dirt effect of [-0.9872 - (-0.198)] = -0.7892. These results show that, for material with initial reflective intensity of R_o, deterioration due to dirt alone can be represented by the following equation:

$$R(t) = 0.3809\, R_o\exp(-0.7892t) \quad (4)$$

High-intensity reflective material also shows deterioration with age but at a substantially slower rate than engineer grade. High-intensity sheeting is specified to retain 80 percent of its original reflective intensity after 10 years of service (according to the 3-M Company pamphlet). In view of the harshness of the railroad environment, it is again assumed that deterioration with age is twice as fast for reflectors on railcars as it is for reflectors in highway applications. A drop to 80 percent in 10 years implies a decay constant of -0.0223; doubling this value to adjust for the railroad case yields for the aging effect the following equation:

$$R(t) = R_o\exp(-0.0446t) \quad \text{(high intensity; age alone; railroad environment)} \quad (5)$$

Reflectors mounted on freight cars will be subjected to periodic cleaning and replacement. If t_w denotes the time since the last cleaning and t_r the time since the last replacement, the reflective intensity for high-intensity reflectors with an initial intensity of R_o is given by

$$R(t_w,t_r) = 0.3809\, R_o\exp(-0.7892t_w)\exp(-0.0446t_r) \quad (6)$$

Federal Highway Administration (FHWA) specifications and manufacturers' guarantees state that, when new, silver/white high-intensity material will have an R_o of 250 cd/footcandle/ft² (2,3), so that

$$R(t_w,t_r) = 95.23\exp(-0.7892t_w - 0.0446t_r) \quad (7)$$

REFLECTOR BRIGHTNESS

The determination of reflector area requires specification of the brightness assumed to be required to attract the attention of a motorist. For a 90° intersection angle between the roadway and the track, the amount of light received by an observer from a retroreflector is given by the equation below (1):

$$E_e = I_s A B t^{2d} W H/d^4 \quad (8)$$

where

- E_e = illuminance received by the observer (footcandles),
- I_s = intensity of the light beamed toward the reflector (cd),
- A = area of the reflector (ft²),
- B = reflective intensity of the reflector (cd/footcandle/ft²),
- t = transmissivity of the atmosphere (per ft),
- W = windshield transmittance,
- d = distance between observer and reflector (ft), and
- H = headlight efficiency.

According to the Transportation and Traffic Engineering Handbook (4), for a level approach grade, a wet pavement, and a vehicle speed of 50 mph, the desired sight distance so that the motorist can stop before reaching the tracks is approximately 50 ft

(1,4). Based on a previous study, a 30 percent reduction of light by the windshield and a 15 percent reduction of light by dirt on the headlights is assumed (1).

Studies have shown that motorists typically use the low headlight beam even when the high beam would be appropriate (1). Under the assumption of a straight and level road with the reflector mounted on the side sill of the freight car, the beam pattern for typical automobile headlights on low beam will provide a source intensity of 3000 cd incident on the reflector.

The Federal Aviation Administration (FAA) has established criteria for the detection of lights in darkness. Based on these standards, it is assumed that an illumination level of 2.3×10^{-6} footcandles will make the reflector sufficiently visible to virtually all motorists (1). Atmospheric conditions are assumed to be clear, with light attenuated 50 percent due to haze in a distance of 5 miles. (This is the situation normally described as "5 miles visibility".) This implies an atmospheric transmittance (t^d) of 94 percent (one-way) at the assumed range of 500 ft. The parameters for the reflector brightness calculation are summarized below:

Item	Value
Required level of illuminance (footcandle)	2.3×10^{-6}
Required detection distance (ft)	500
Windshield transmittance	0.70
Headlight efficiency	0.85
Headlight intensity (per light) (cd)	3000
Atmospheric transmittance (one-way, 500 ft)	0.94

The required reflector brightness can be determined from the above equation for E_e with the assumed values presented above. The results of applying that equation indicate that, for a straight and level roadway, the reflector must return at least 45 cd/footcandle of incident light in order to attract the attention of virtually all motorists (except those incapacitated by alcohol, drugs, fatigue, etc.) at a distance from the crossing sufficient to permit stopping the vehicle safely.

The reasonableness of this theoretical finding is shown by comparison with two other devices used to warn motorists of obstacles in the highway: the emergency triangle and vehicle marker lights.

1. The emergency triangle "is to be carried in commercial motor vehicles and used to warn approaching traffic of the presence of a stopped vehicle" (5). Triangular in shape, it includes both orange fluorescent material for daytime visibility and red reflective material for night visibility. The basic specification for the reflective portion is that it return 80 cd/footcandle of incident light. If dirt accumulation leads to deterioration of this value in use by as much as a factor 2, the light returned would be 40 cd/footcandle, which is very close to the value of 45 developed above.

2. A variety of white and amber lamps are required on motor vehicles to serve as marker, parking, and clearance lights. All have the basic function of alerting drivers to the presence of a vehicle in or near the road. The minimum intensity required for these lights is 1 cd for white devices and 0.68 cd for amber (6). A reflector that returns 45 cd/footcandle of incident light, determined above to be appropriate for the freight car application, has a brightness of 0.87 cd when illuminated and observed as described in the table above. This brightness is midway between the specified minimum intensity for white and amber vehicle lights.

REFLECTOR AREA AND WASHING INTERVAL

Given the constraint that the reflector must return at least 45 cd/footcandle of incident light, the area (A) of the reflector must be large enough to satisfy the relation: A x (reflective intensity) $\geqslant$ 45. By using the earlier expression for $R(t_w,t_r)$, $A \times 95.23 \exp(-0.7892t_w - 0.0446t_r) \geqslant 45$, or $A \geqslant 0.4725 \exp(0.7892t_w + 0.0446t_r)$.

In the following analysis, it is assumed that the replacement interval is a multiple of the wash interval. This is not strictly necessary but has practical advantages as well as simplifying the analysis. For specified wash and replacement intervals T_w and T_r, the area is determined from the condition that the above constraint on the area A is met as an equality immediately prior to replacing the reflectors; that is, when $t_w = T_w$ and $t_r = T_r$.

This situation is illustrated by Figure 4, which is a graph of the intensity of a reflector with area A that is replaced after T_r years and washed every T_w years. (In this example, $T_r = 6T_w$.) At time T_r, the intensity has degraded to 45 and the reflector is replaced. The necessary area A is thus determined by the following equations:

$$A \times [95.23 \exp(-0.7892T_w - 0.0446T_r)] = 45 \tag{9}$$

or

$$A = 0.4725 \exp(0.7892T_w + 0.0446T_r) \tag{10}$$

Figure 5 shows the area calculated from this expression, graphed as a function of washing interval T_r, for a 10-year replacement interval. The assumption of a 10-year replacement interval is consistent with manufacturers' specifications for high-intensity material. It can be seen that the required reflector area is quite large--greater than 1 ft² for most washing intervals--and substantially exceeds sizes considered in previous studies.

The choice of the reflector size to be used for a particular situation can be based on minimizing the total lifetime discounted cost of reflector material, installation and replacement, and washing. For material with a given service lifetime (replacement interval), use of larger reflectors increases initial cost, since more reflective sheeting is used, but maintenance expense is lowered because less frequent washing is necessary to prevent reflective intensity from falling below the required 45 cd/footcandle. The least-costly choice of reflector size is that that balances these two effects to attain the lowest total expense. The life-cycle cost to reflectorize a freight car can be written in terms of three components: cost = material cost plus installation labor cost plus maintenance (washing) cost.

This cost is a function of reflector size, the washing interval, and the replacement interval. Once labor and material costs are known, the above equations that relate area, wash interval, and replacement interval could be used to determine the size and maintenance schedule to minimize life-cycle costs. Practical considerations, such as integrating wash and replacement intervals into freight car maintenance schedules and the possible need for stenciling to indicate when a car's reflectors were last washed, would have to be included in a realistic economic analysis.

Figure 4. Reflector brightness versus time for wash period T_w and replacement period T_r.

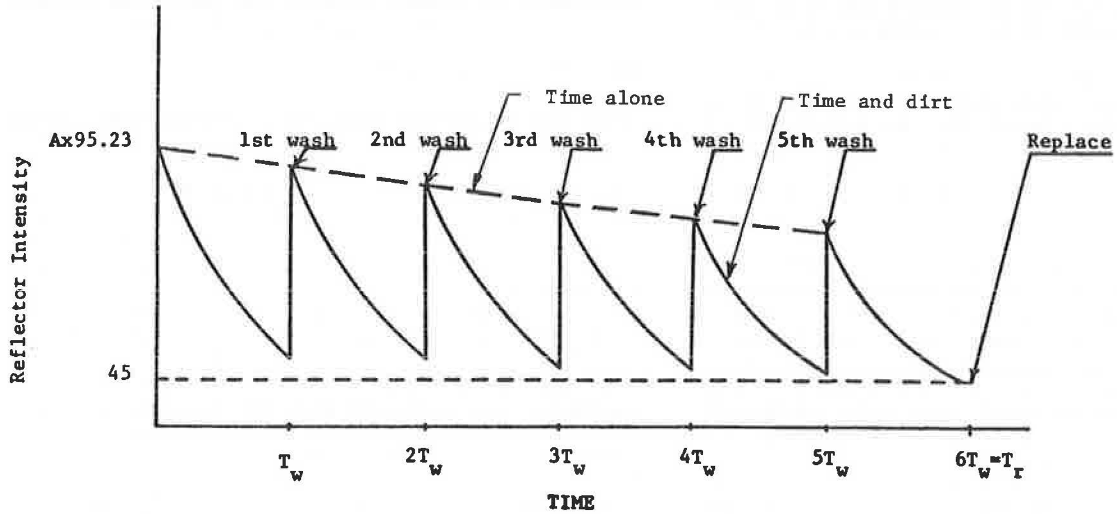
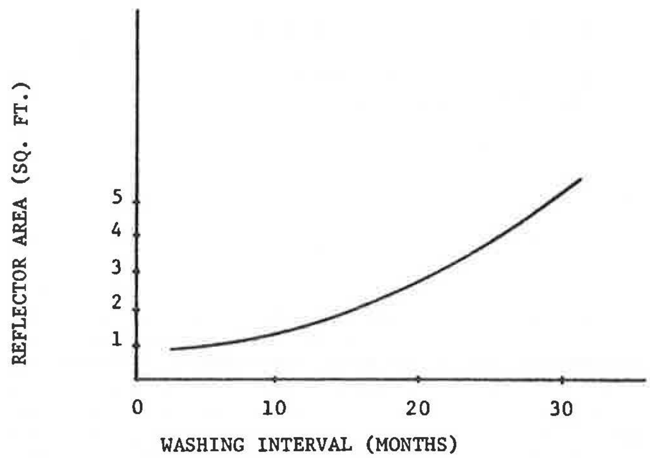


Figure 5. Required reflector area versus washing interval.



CONCLUSIONS

The Canadian reflector measurements on engineer-grade reflectors indicated rapid deterioration in the reflective intensity to 23 percent of original value after six months in revenue service, 14 percent after one year, and 5 percent after two years. Data from tests on the B&M strongly indicate that high-intensity reflectors also deteriorate in the railroad environment and at a rate similar to that observed of engineer-grade reflectors in Canada. However, the limited time for the high-intensity reflector tests precludes absolute conclusions on high-intensity reflector durability.

The decline in reflectivity has important implications for the size, lifetime cost, and cost-effectiveness of reflectors necessary if one is to be confident of attracting a motorist's attention. Reflectors would have to be substantially larger than prior studies have assumed and the expense of reflectorization would be correspondingly greater. For example, as shown in Figure 5, it can be seen that a reflector area of approximately 1.5 ft^2 would be sufficient for a one-year wash interval with replacement after 10 years; a reflector more than 3 ft^2 would be required for a two-year wash interval and 10-year replacement. The choice of a particular combination of size and washing frequency would depend on material, installation and washing costs, and other practical constraints.

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Does Roadway Luminance Correlate with Visibility Metric of CIE 19/2?

BILLY LEE SHELBY

The latest mathematical model for predicting visibility of an object was published in 1980 by the International Commission on Illumination (CIE) as technical report 19/2. It is based on a visibility index (VI), which is defined as the product of equivalent contrast, relative contrast sensitivity, disability glare factor, and transient adaptation factor and is modified by the reciprocal of the constant 0.0923. The visibility of a small square target (18 cm on a side) with a totally diffusing surface when viewed against an R3 roadway surface background has been evaluated by using this model. The Roadway Lighting Committee of the Illuminating Engineering Society of North America has just approved a new standard practice for roadway lighting. The design criteria are based on two equally acceptable metrics—pavement luminance or illuminance. The decision of which to use is left to the discretion of the designer or specifier. This paper evaluates the correlation between the pavement luminance criteria and the visibility model (VI) of CIE 19/2.

The Roadway Lighting Committee (RLC) of the Illuminating Engineering Society of North America (IESNA) on August 7, 1982, approved a revision to the Standard Practice for Roadway Lighting (ANSI RP-8). This revision incorporates design criteria based on pavement luminance, luminance uniformity, and veiling glare as the preferred and equally acceptable metric for roadway lighting. The previous edition (1) was based solely on horizontal illuminance.

The objective of fixed roadway lighting and automobile headlights is to make visible the objects and cues on the roadway to permit the driver to evaluate the visual scene for the purpose of driving safely.

In 1980, the International Commission on Illumination (CIE) published report 19/2 (2), which defines a mathematical model for describing the influence of lighting parameters on visual performance.

The purpose of this paper is to evaluate the correlation between the visibility model of CIE 19/2 and the luminance performance criteria proposed in the IESNA-recommended standard practice.

THE TARGET

To apply the visibility model of CIE 19/2, we need a target to simulate an object on the roadway to be detected by the driver. A variety of targets, from simple discs to three-dimensional objects of various sizes and shapes, have been used in different roadway lighting research projects. For this evaluation, a flat two-dimensional target (18 cm on a side) with a perfectly diffusing surface has been chosen because it was desired to have the following attributes:

1. A simple flat target whose brightness is easy to predict;
2. A target of about 4 minutes in visual size at 150 m, since much of CIE 19/2 was based on a 4-minute target; and
3. A size and shape that would make it easy to produce semirealistic objects by combining a number of the 18-cm targets.

VISIBILITY MODEL

One of the earliest applications of a visibility model for roadway lighting was developed by Gallagher (3, p. 85). He called the visibility metric a visibility index (VI), which was defined as

$$VI = C \cdot RCS \cdot DGF \qquad (1)$$

where

C = contrast,
RCS = relative contrast sensitivity, and
DGF = disability glare factor.

CIE 19/2 also used the term visibility index to define the purely physical measures of visibility. Their formula is slightly different:

$$VI = (C \cdot RCS \cdot DGF \cdot TAF)/0.0923 \qquad (2)$$

where TAF is the transient adaptation factor.

In this paper, Equation 2, which is from CIE 19/2, was used. The terms C, RCS, DGF, and TAF are as defined in CIE 19/2. (Note, the definition of the terms C, RCS, and DGF are the same for both Equations 1 and 2.) The computer program used to develop these values was intended to include TAF in the calculations. However, some problems arose as to how large an area to use as the background in the glance at either the lightest or darkest roadway areas just prior to fixation on the target. Some exploratory calculations gave a TAF between 0.97 and 1.0, regardless of the interpretation used. Therefore, it was decided to set TAF equal to 1.0 (4, p. 151) for all the calculations of VI, i.e.,

$$VI = (C \cdot RCS \cdot DGF)/0.0923 \qquad (3)$$

This is the formula used in all visibility calculations for this paper. The physical variables required for the parameters of VI are as follows:

Lt = luminance of task,
Lb = luminance of the background around the task,
d = task size (angular minutes),
A = age of the observer (years), and
Lv = veiling luminance

ROADWAY GEOMETRY AND ANALYSIS PROCEDURE

Figure 1 shows the elevation and plan view for a typical lighting system as used in this study. The observer is located in a vehicle at 1.45-m eye height and (headlights off) one-quarter of the lane width from the left edge of the lane. The target (task) is always located on the ocular line of sight that is always parallel to the lane width markers and curb. The fixed lighting is always arranged so that the first unit is on the same transverse roadway line as the observer and on his or her left

Figure 1. Typical lighting system.

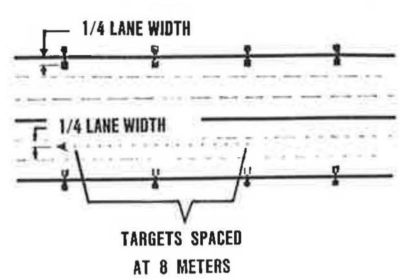


Figure 2. Driver's view of road.

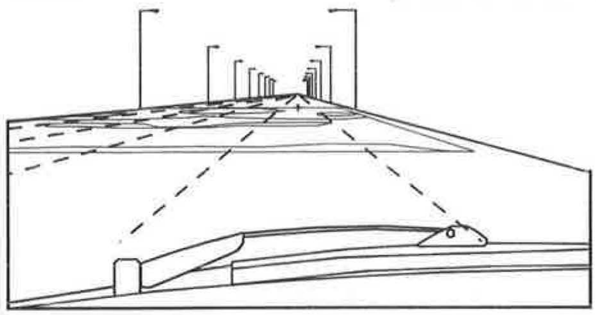
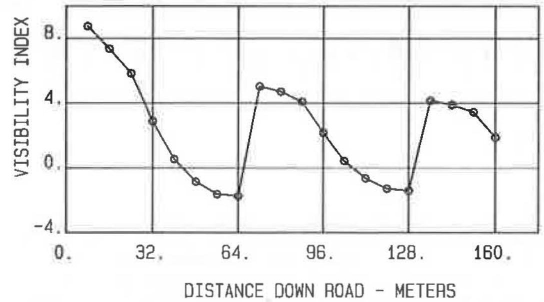


Figure 3. Typical VI.



side. The luminaire overhang is always one-quarter of the lane width. The roadway pavement is considered to have the directional reflectance characteristics of a CIE R3 pavement (5).

The targets are always spaced down the road at 8-m intervals. Because the target is relatively small, one location on the target was used to calculate Lt and one location on the pavement (target removed) to calculate Lb (target shadow ignored). Pavement luminance, both when used as a background for the target and when expressed as an average luminance over the entire roadway (Lave), was calculated by the method recommended in the new proposed Standard Practice for Roadway Lighting (RP8). Veiling luminance for the stationary observer is the maximum found in any lane as the observer is moved through the luminaire cycle and is calculated as recommended by the RLC in RP8. Figure 2 shows the typical lighting arrangement from a driver's viewpoint.

Following are the variables of the typical lighting arrangement and luminaire characteristics used in the analysis:

1. Target reflectance,
2. Observer and target locations,
3. Lamp lumens,
4. Luminaire spacing,
5. Luminaire mounting height,
6. Photometric distribution, and
7. Opposite versus staggered.

As can be readily understood, there is an infinite number of combinations of the variations listed above. It would be equally impossible to cover the impact of all of these variations in this paper. The reasoning used to select the specific details investigated to date is explained next.

This paper is devoted to exploring the visibility of multiple targets as the application techniques commonly used to improve the level and uniformity of luminance and illuminance are applied. It is hoped that this will give a better understanding of the possible and probable detection of a target that might suddenly occur ahead of the observer out to a distance of 160 m, which is well beyond the distance required for safe stopping at 88.5-km/h driving speed. The target is in the same lane as the observer in all cases, since it is felt that the driver will take no drastic evasive or braking action on seeing a stationary target in another lane. Also, the analytic model for CIE 19/2 is not useful for predicting the visibility of an object in motion.

Once such a target occurs, the observer, who is really in a moving vehicle, then approaches the target at his or her driving speed. As the driver does so, the visibility of the target will vary due to the following facts:

1. The targets angular size increases.
2. Lt will increase as the observer's headlights approach the target. This may increase or decrease contrast, depending on whether the initial detection occurred under positive or negative contrast conditions.
3. The background against which all or part of the target is seen will change.
4. Lv will increase and decrease in a rhythmic cycle as the observer passes through the fixed lighting system.

CIE 19/2 as well as most other work in lighting has defined contrast as follows:

$$C = |(L_t - L_b)/L_b| \tag{4}$$

Because it is felt that item 2 above is a very important factor in explaining the ability to detect an approaching target, the sign of the contrast (+ or -) was maintained:

$$C = (L_t - L_b)/L_b \tag{5}$$

Carrying this into the VI calculation gives the possibility of either a positive or negative VI.

DATA PRESENTATION AND DISCUSSION

The same geometric arrangement is used as the base for comparison throughout the study. The list below gives the particulars for this arrangement:

1. Road width = 22 m,
2. Number of lanes = 6,
3. Arrangement = opposite,
4. Mounting height = 10.67 m,
5. Spacing = 64 m,
6. Overhang = 0.92 m,
7. Target location = lane 5,
8. Target reflectance = 20 percent,
9. Target spacing = 8 m, and
10. Luminaire distribution = type III medium semicutoff.

Figure 3 shows that the VI varies from positive to negative as the target is moved through the luminaire spacing cycle. The positive maximums occur just beyond the luminaires and the negative maximums occur with targets located just ahead of the luminaires. The value of the maximums decreases as the target goes farther down the roadway. This is due to the decrease in angular size of the target as the distance between observer and target increases.

To make sense of the comparisons with pavement luminance criteria, we need to establish methods of evaluating the merit of the visibility performance. Although it is not the intent of this paper to argue for or against a particular method, two arbitrary figures of merit for use in this study have been

Figure 4. Typical VI for visibility figure of merit.

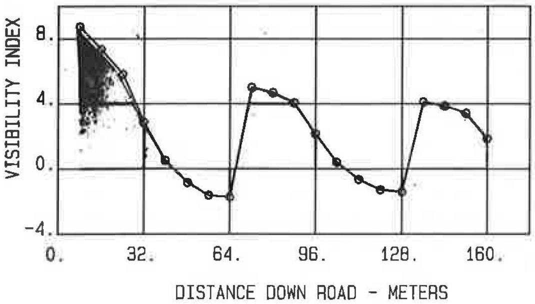
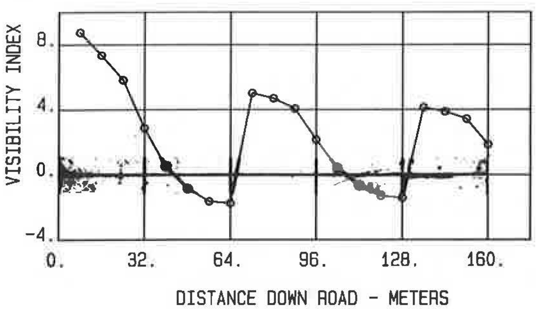


Figure 5. Visibility figures of merit for percent less than ABS one.



chosen. Figure 4 shows the one called VI Ave., which is the numerical average of the sum of the VI for each target position, disregarding the sign of the individual VI value. Figure 5 shows the figure of merit based on values that are less than absolute one. This is the percentage of the total number (20) of target positions that have a VI whose absolute value is less than one (referred to as %<|1|).

Although we know that the observer's vehicle headlights will have a large effect on target visibility when within a few meters of the vehicle (up to about 60 m), we also know that the effect beyond approximately 60 m will be negligible. Unfortunately, the directional reflectance tables for the pavement do not have values for the angles that define the positional relation of the headlights to the pavement, so we cannot calculate the headlights' effect on pavement luminance. Therefore, all of this was done on the basis that the vehicle's headlights were off.

TARGET REFLECTANCE EFFECT

Figure 6 shows the results with target reflectances of 5, 20, and 40 percent. The table below shows that, as the theory indicates, the VI Ave. goes up and the %<|1| goes down as target reflectance goes up (note, Ave./min = average-to-minimum ratio):

Target	Visibility		Luminance	
Reflectance (%)	VI Ave.	$\%<\lvert 1\rvert$	Lave	Ave./min
5	0.84	55	1.0	2.5
20	3.14	20	1.0	2.5
40	6.89	10	1.0	2.5

Observer and Target Location Effect

Figure 7 shows the results with the observer and target located in lanes 4, 5, and 6. The table below shows that, as the observer and/or target position is moved closer to being in line with the

Figure 6. VI versus target reflectance.

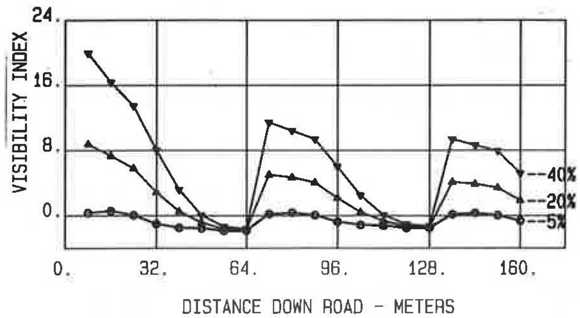


Figure 7. VI versus target position.

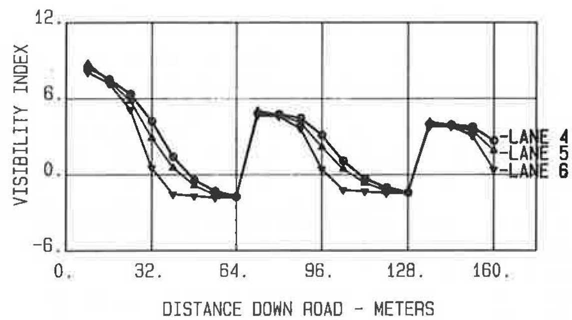
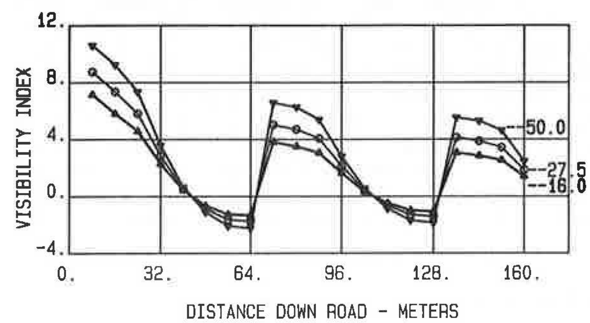


Figure 8. VI versus lumens.



luminaires on the right side of the roadway, the VI Ave. decreased but the %<|1| was not consistent:

Target	Visibility		Luminance	
Position	VI Ave.	$\%<\lvert 1\rvert$	Lave	Ave./min
4	3.35	10	1.0	2.5
5	3.14	20	1.0	2.5
6	2.88	15	1.0	2.5

This needs some further study to determine the significance.

Lamp Lumens Effect

Figure 8 shows the results as the lamp lumen package is changed from 16 000 to 27 500 to 50 000 lumens. The table below shows that, just as theory would predict, the Lave increased in direct proportion to the lamp lumens:

Lamp Lumens	Visibility		Luminance	
(000s)	VI Ave.	$\%<\lvert 1\rvert$	Lave	Ave./min
16.0	2.42	25	0.6	2.5
27.5	3.14	20	1.0	2.5
50.0	4.04	15	1.8	2.5

However, VI Ave., while it increased, did not increase as much as the percentage change in lumens, and the change in %<|1| was not in the same proportion as the lumen change.

Luminaire Spacing Effect

Figure 9 shows the results of increasing the spacing-to-mounting height ratio (MH) from 3 to 8 to 12. The table below gives the numerical changes in pavement luminance and VI figures of merit due to changing the spacing:

Luminaire Spacing (MH)	Visibility VI Ave.	%<|1|	Luminance Lave	Ave./min
3	2.28	25	1.7	2.4
6	3.14	20	1.0	2.5
8	3.63	10	0.8	6.6
12	3.45	25	0.5	109

The change in the Lave and luminance uniformity are as expected. However, we see that the VI Ave. increases as the spacing is increased up to about 8 MH and then decreases as the spacing is increased further. The %<|1| values follow this same trend. Because this increase in VI Ave. up to 8-MH spacing is contrary to what we have traditionally assumed (we always thought visibility was directly correlated with average pavement luminance), some further analysis is warranted.

Figures 10 and 11 help to explain what is happening to the VI value at each individual target position as the spacing is increased from 3 to 6 MH. Target position 1, with a spacing of 3 MH, has a large contribution to background luminance from luminaires 3 and 4. The greatest contribution to target luminance comes from luminaire 2. When the spacing is increased to 6 MH, in effect you have eliminated luminaires 1 and 3. The loss in background luminance is greater than the loss in target luminance; therefore, the contrast has increased. Also, the veiling luminance has decreased due to the removal of luminaire 3, which results in the DGF increasing. The overall result is an increase in the VI value for target position 1. Going through this same exercise for all target positions shows that the total sum of the absolute VI values increases, which results in the VI Ave. increasing.

Luminaire Mounting Height Effect

Figure 12 shows the result of increasing the luminaire mounting height from 7.9 to 10.7 to 13.7 m. The table below shows that the figures of merit for pavement luminance and VI move in the same direction with changing MH, but the VI Ave. changes at a faster rate:

Luminaire Mounting Height (m)	Visibility VI Ave.	%<|1|	Luminance Lave	Ave./min
7.9	4.41	0	1.1	3.1
10.7	3.14	20	1.0	2.5
13.7	2.1	15	0.9	1.6

The pavement luminance ratio is 1.1/0.9 = 1.22, while the VI Ave. ratio is 4.41/2.1 = 2.1. This could be a significant factor in designing roadway lighting geometry for an optimum VI figure of merit.

Luminaire Distribution Effect

Figure 13 compares the VI difference between a luminaire with a II long-cutoff distribution to one with

Figure 9. VI versus spacing.

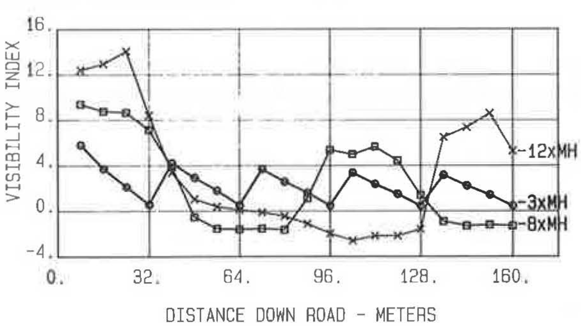


Figure 10. Three-MH spacing.

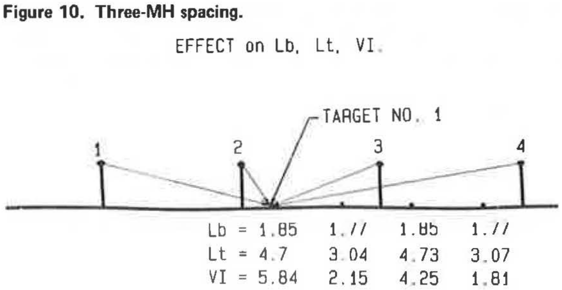


Figure 11. Six-MH spacing.

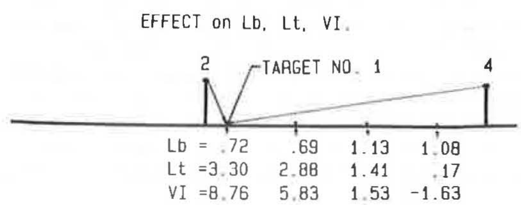


Figure 12. VI versus mounting height.

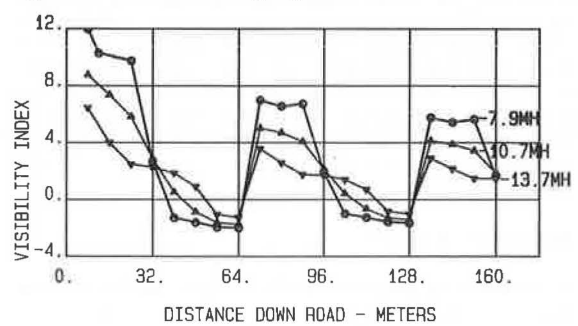
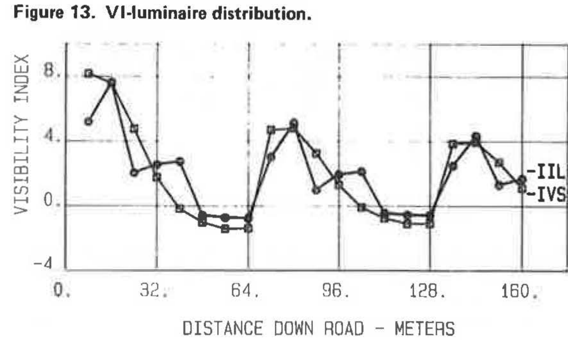


Figure 13. VI-luminaire distribution.



a IV short-semicutoff distribution. The table below gives the numerical results:

Luminaire Distribution	Visibility VI Ave.	Visibility %<|1|	Luminance Lave	Luminance Ave./min
IV short semicutoff	2.75	15	0.7	2.0
II long cutoff	2.37	30	1.2	4.0

As we would expect, the change in the pavement luminance figures of merit are significant. The change in VI Ave. was very small, but the change in %<|1| was 2 to 1. This also warrants some further study.

Arrangement Effect

Figure 14 shows the results of changing the luminaire arrangement on the roadway from single-sided left to opposite to staggered. In this arrangement analysis, the roadway was changed to 13.4-m width with four 3.35-m-wide lanes. The observer and targets are located in lane 3 (numbered from the left side). Figure 14 shows the results with spacing of 64 m. The table below gives the numerical results that correspond to Figure 14:

Luminaire Arrangement	Visibility VI Ave.	Visibility %<|1|	Luminance Lave	Luminance Ave./min
Single left	3.1	25	0.95	7.6
Opposite	4.5	0	1.9	2.0
Staggered	1.7	25	1.9	2.0

The pavement luminance figures of merit, as expected, do not significantly change going from the opposite to the staggered arrangement but do significantly change when going to the single-sided arrangement.

However, the effect is just the opposite for the VI figures of merit. The change from opposite to single-sided VI Ave. decreased by 30 percent, but going to staggered the VI Ave. decreased by 60 percent.

In this study, the word correlation is not used in the sense of a rigorous statistical relation but is used to indicate that VI criteria consistently move in the same direction as luminance criteria. On this basis, when the physical parameters (i.e., spacing, mounting height, etc.) that are expected to effect both visibility and pavement luminance are varied, no correlation is found in four out of seven of the parameters. This is shown graphically in Figure 15.

FIELD STUDY

To check the correlation between the CIE 19/2 model and an actual roadway installation, the research and visibility subcommittee of RLC of IESNA conducted a field study on a roadway in Chicago in October 1982. (Although the committee report has not yet been made public, permission to use the results was given.)

The roadway was two 3.7-m lanes with 400-W mercury type III medium semicutoff luminaires mounted at 10.7 m and with 64-m spacing. Targets with the same physical dimensions as described above and of 5, 17, 20, and 30 percent reflectances were used. These targets were located at three different positions with two different observer positions.

Measurements of target luminance and background luminance were made with a Pritchard photometer by using the 6-minute aperture. The visual task evaluator (VTE) was used by an observer experienced in its use to get a measure of the $\tilde{C}$. (Note, $\tilde{C}$ is a weighted measure of contrast by the VTE.)

Figure 14. Luminaire arrangement.

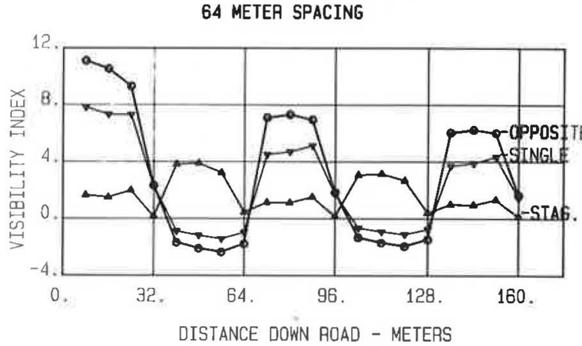


Figure 15. Correlation of luminance and VI.

	LUMINANCE AVE	LUMINANCE A/M	VISIBILITY INDEX AVE	VISIBILITY INDEX %< |1|	CORR?
TAR. REFL ↑	–	–	↑	↑	N.A.
TAR. LOC. ↑	–	–	↓	↕	N.A.
LUMENS ↑	↑	–	↑	↓	NO
SPACING ↑	↓	↑	↑	↓	NO
MT HT ↑	↓	↓	↓	↓	YES
PHOT DIST ↔	↑	↑	↓	↑	NO
ARRANG ↔	↑	↓	↕	↕	NO

Table 1. Field study, Chicago.

Test	Target No.	Target Percent	Results from Field Study $\tilde{C}$	Results from Field Study VL	Results from Field Study VI	Computer-Predicted Results VI	Computer-Predicted Results C[a]
1	T1	5	-0.71	-1.73	-1.53	-0.89	-0.53
2	T1	17	-0.25	-0.61	-0.13	1.02	0.60
3	T1	30	T[b]	–	–	3.10	1.83
9	T2	17	-0.56	-1.43	-2.44	-1.84	-0.97
14	T3	30	T[b]	–	–	3.53	1.96

[a] C is a measure of contrast from physical parameters.
[b] T = threshold.

The measurements were used to calculate visibility level (VL) and VI as defined in CIE 19/2 and in the IES report (6, p. 40). Table 1 gives the VIs calculated from the VTE measurements as compared with the VIs predicted by the computer program that used the CIE 19/2 model. One observation is that the VI from both methods moved in the same direction: As the target reflectance was increased, the VI increased in the positive direction.

However, the VI from the VTE measurements never went positive, even though the contrast measurements from the Pritchard showed 30 percent of the targets to have positive contrast. It should be noted that the VTE indicated the same as visually perceived by several observers at the test site. Further to the point is that the computer runs that used the CIE model predicted that the VIs for both the 17 and 30 percent targets would be positive at target locations T1 and T3.

How can these differences be explained? The computer prediction program for the VI has been checked and, with the exception of the TAF, it does compute the VI based on the formulation from CIE 19/2. The visual scene in the field study has a significant number of light sources in the field of view as well as periodically heavy traffic in both directions that seemed to be affecting target contrast and veiling luminance. None of these effects are accounted for in the computer-predicted VI. Do these factors influence our adaptation in a way that is not accounted for in the CIE 19/2 model?

These last questions indicate the need for further study to correlate field conditions with the measurement techniques and with prediction models. It is hoped that this additional study can be done within the next year.

CONCLUSIONS

Although further study is warranted to fine-tune the results of varying specific physical parameters related to pavement luminance and VI figures of merit, the study to date was rigorous enough to draw some basic conclusions. These are as follows:

1. Average pavement luminance is not a predictor of average visibility of the small target used based on the model of CIE 19/2,
2. Uniformity of pavement luminance is not a predictor of improved VI figures of merit based on the model of CIE 19/2 for this target, and
3. Many application techniques commonly used to improve the figures of merit for pavement luminance cause reductions in the figures of merit for VI based on CIE 19/2 for this target.

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Methodology for Determining Pavement Reflectivity for Roadway Luminance Calculation

RONALD N. HELMS

The field test procedures for determining the reflective properties and the macrostructure of a pavement surface are described. The light-reflecting characteristics of roadway surfaces affect the quantity and quality of light reflected (luminance) from the pavement. The luminance of the pavement surface and its uniformity affect the visibility of objects on the roadway surface. The uniformity of luminance patterns reduces confusion and clutter and stabilizes the driver's adaptation state. Improved visibility through better roadway lighting can result in increased safety for the nighttime driver. The luminance of the pavement is the product of the light arriving at a point (illumination) multiplied by the luminance factor. The luminance factor represents the directional reflectivity of the pavement, which is a function of light source location, observer location, and macrostructure or microstructure of the pavement. The parameters used for test-site selection and a matrix of test-site variables are presented. Once a test site has been selected, the test-site locations for reflectance measurements, British portable tester, and sand patch are defined, and the preparation and measurement techniques are described. As reported in the paper, problems were discovered in the reflectometer mechanism electronics that invalidated the test data. The methodology presented provides insight into the complexities of making pavement reflectance measurements. The procedures described lay the foundation for subsequent field measurements. The measurements are required to establish a simplified pavement reflectance classification system that is crucial to roadway luminance calculations.

Fixed roadway lighting is important to the nighttime driver. The safety of a driver at night is dependent on visibility, visual comfort, and driver alertness. These factors affect the driver's ability to see objects on the roadway with sufficient warning to take appropriate and safe action. The visibility of an object on the roadway is dependent on the contrast between the object and its background (the pavement). The size and shape of the object, the state of driver adaptation, and the movement (versus stability) of the object also contribute to the visibility of the object.

Headlight penetration limits the driver's visibility to distances that are marginal for safe stopping. Vehicle stopping distance consists of reaction distance (reaction time x velocity) and braking distance. The reaction time is directly related to object visibility. The object must be detected and recognized by the driver. The driver must respond to the recognition of the object and then initiate some form of action. The response to the recognition of the object and the initiation of action are related to the physiology of the individual driver. The detection and recognition are dependent on the visibility of the object. The size and shape of the object and whether the object is stationary or moving are beyond the control of the driver or the agency responsible for safety on the highway. However, fixed lighting systems can increase object contrast, driver adaptation state, and detection distance. The improved visibility can reduce reaction time and hence reaction distance. The increased detection distance will result in an increased margin of safety to allow the driver to take appropriate and safe action.

The level of background luminance and its uniformity affect the contrast discrimination and one's sensitivity to contrast. The background luminance is a function of the reflective properties of the pavement. Light that strikes the macrostructure or microstructure of the pavement will cause different quantities of light to be reflected toward the driver. The quantity of light leaving (luminance) the roadway is a function of the direction from which the light is arriving (illumination) and the reflecting properties of the surface for a given viewing direction and angle. The term "luminance factor" is used to describe the reflected component from a nondiffuse reflecting surface. The luminance factor, which is denoted by the symbol β, varies as a function of the location of the light source, the location of the observer (driver), and the macrostructure or microstructure of the pavement. This means that the luminance of a point (Lp) is the sum of the products of the illumination that arrive at the point multiplied by the corresponding luminance factor, i.e.,

$$L_p = \Sigma\beta(v_o, v_s, h) \times E \quad (1)$$

where $\beta(v_o, v_s, h)$ defines a specific luminance factor for a given viewing position (v_o) and light source location (v_s, h) (see Figure 1).

Reid and Channon (1,2), Kraehenbuehl (3), Finch and Marxheimer (4), Wyatt (5), and King (6,7) are some of the researchers in the United States that have made laboratory measurements of pavement reflectivity and developed classification schemes for calculating pavement luminance. The major problem in applying these techniques is the lack of a method for quickly relating the existing roadway conditions to those pavements measured in the laboratory. Past measurements of the reflectivity properties of pavements have been accomplished by removing the pavement sample to the laboratory or by extensive and bulky field instrumentation. When samples are removed from the pavement, there can be gross changes in the macrostructure of the sample that could affect its reflecting properties and hence the validity of the data.

Problems with the instrument electronics necessitated a change in electronics subcontractors six months into the project. Because problems with the electronics had already delayed data collection by three months, it was decided that the new subcontractor should salvage as much of the existing reflectometer mechanism electronics as possible. In hindsight, this was a mistake. The change in subcontractors and the modification of the electronics resulted in a six-month delay in data collection. This resulted in a reduction in the test period from the original 12 months to only 7 months.

Data collected during the first three months of field measurements were consistent with previous laboratory data (6,7). The subcontractor, who was responsible for data analysis, got behind in analyzing data due to delays in scheduled computer system maintenance and modification. The data collected during the last four months were analyzed during the seventh month of testing. The analysis of the last four months of data indicated that another malfunction had occurred in the original reflectometer mechanism electronics during the time the data-analysis computer was down.

This paper describes the instrumentation and methodology developed under a research grant from the Federal Highway Administration (FHWA) (8) for

Figure 1. Measurement terminology.

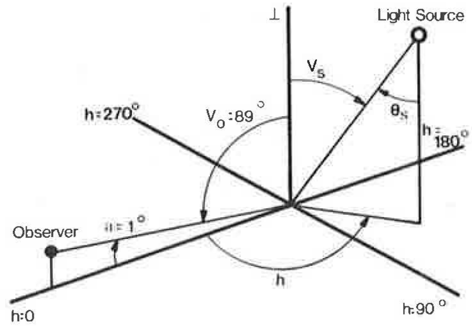
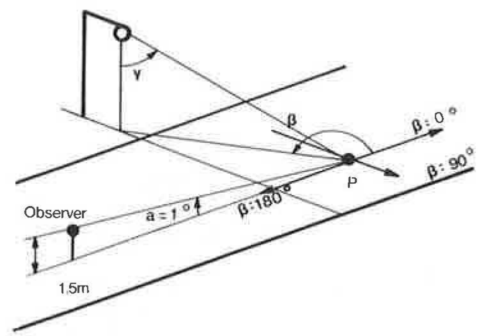


Figure 2. CIE definition of beta and gamma.



making in situ measurements of the reflecting properties (luminance factors) for roadway surfaces. The paper does not deal with the data collected during the seven-month test period. The basic reflectometer mechanism operation, control electronics (interface), and data-collection system were flawless in their operation. The concepts and methodology were judged significant enough that FHWA chose to fund a second phase of this research project. The second phase involves the modification of the reflectometer mechanism electronics, which is under way at this time.

The instrument used to measure the luminance factors is called a gonio-reflectometer. By using the procedures described in this paper and the gonio-reflectometer, a large in situ data base could be developed. These data, combined with the large base of existing laboratory data, could be used to develop a pavement reflectance classification system and a light-weight, simple field instrument for classifying unlit roadways.

In Europe, researchers (9-11) have developed a classification system that has evolved into the International Commission on Illumination (CIE) method (12). The CIE method uses the terminology "luminance coefficient" rather than "luminance factor" to describe the reflecting characteristics of nondiffuse surfaces. The symbol β is used to describe a horizontal angle in the CIE method (Figure 2), while β is the symbol used to denote luminance factors in the United States. In the CIE method, each luminance coefficient (luminance factor) is multiplied by $\cos^3\gamma$ (Figure 2) and 10^4 to produce a reduced luminance coefficient, which is reported in a tabular form called an r-table. The CIE method uses a viewing angle (a) of 1°. Figure 3 shows the standard format for an r-table in the CIE method. The data reported in an r-table are limited to a portion of the hemispherical data, which limits luminance calculations to points that fall within a defined area. Luminance calculations that involve

Figure 3. CIE beta and gamma angles.

		β																		
tanγ	γ	0	5	10	15	20	25	30	35	40	45	60	75	90	105	120	135	150	165	180
0	0	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x
0.25	14.04	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x
0.5	26.57	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x
0.75	36.87	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x
1	45.00	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x
1.25	51.34	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x
1.50	56.31	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x
1.75	60.26	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x
2	63.43	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x
2.5	68.20	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x
3	71.57	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x
3.5	74.05	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x
4	75.96	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x
4.5	77.47	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x
5	78.69	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x
5.5	79.70	x	x	x	x	x	x	x	x	x										
6	80.54	x	x	x	x	x	x	x	x											
6.5	81.25	x	x	x	x	x	x	x												
7	81.87	x	x	x	x	x	x	x												
7.5	82.41	x	x	x	x	x	x													
8	82.87	x	x	x	x	x	x													
8.5	83.29	x	x	x	x	x	x													
9	83.66	x	x	x	x	x														
9.5	83.99	x	x	x	x	x														
10	84.29	x	x	x	x	x														
10.5	84.56	x	x	x	x	x														
11	84.81	x	x	x	x	x														
11.5	85.03	x	x	x	x															
12	85.24	x	x	x	x															

Figure 4. Block diagram of gonio-reflectometer.

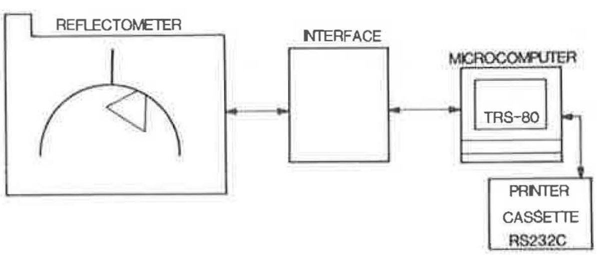
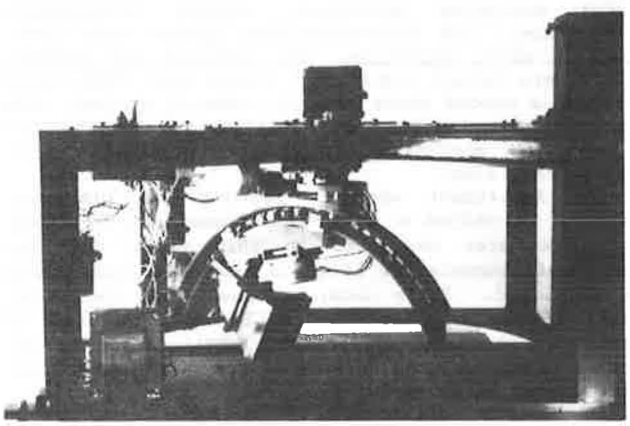


Figure 5. Photometer housing, declination arm, and illuminator.



headlights, larger surface areas, and curved roadways fall outside the CIE r-table data. This paper recommends the reporting of complete hemispherical data in terms of luminance factors rather than reduced luminance coefficients. The gonio-reflectometer described in this paper makes complete hemispherical data possible. The storage of that data in a computer makes it practical and logical to use complete hemispherical data. The paper also establishes terminology and symbols consistent with photometrics and calculations used in the United States.

TESTING EQUIPMENT

The testing equipment consists of the gonio-reflectometer, sand patch equipment, and a British portable tester (BPT). The gonio-reflectometer measures the complete hemispherical luminance factor data. The sand patch equipment (13) is used to measure the texture depth or macrostructure of the pavement. The BPT (13) is used to measure the surface frictional properties of the roadway surface.

The gonio-reflectometer is composed of three subsystems (Figure 4). They are (a) the reflectometer mechanism, (b) the interface between the mechanism and controller, and (c) the controller, which is a microcomputer and its peripherals.

The reflectometer mechanism (Figure 5) consists of five basic units: mainframe, leveling base, azimuth drive, declination drive, and illuminator. The mainframe is a weldment built of aluminum to ensure structural rigidity and repeatability of measurements. The mainframe is attached to a leveling base that sits on three support pads that are adjustable for leveling. The azimuth drive, which consists of a stepper motor, gear-reduction transmission, and declination arm, is attached to the top inside of the mainframe. The stepper motor rotates the declination arm and illuminator nine steps per degree from 0 to 355 in azimuth (horizontal angle). The gear-reduction transmission contains a worm gear and wheel to prevent any overrun or movement during transportation. The declination drive, which is attached to the declination arm, consists of a stepper motor and gear transmission. The illuminator is attached to the declination drive. The declination gear produces 24 steps per degree of declination (vertical angle). The illuminator produces a uniform collimated beam of light 4 in on a side. The collimated beam of light is reflected off the pavement through a series of baffles and a lense in the photometer housing to a photodiode. The photometer looks at a circular area 3 in in diameter in the center of the illuminated area.

The second subsystem is the interface unit. The interface receives analog voltages from the illuminator and the photometer and converts these voltages to digital numbers under command of the microcomputer. It also converts computer motor commands to the appropriate voltages for operation of the azimuth and declination stepper motors.

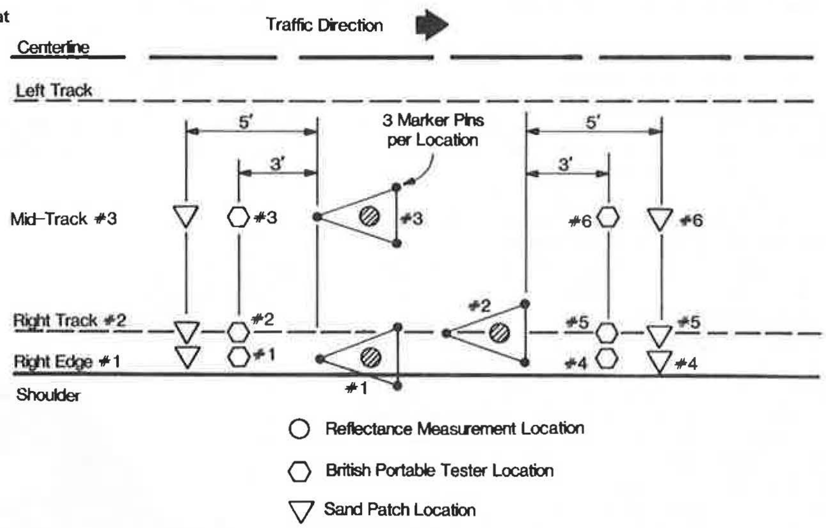
The controller, which is the third subsystem, is a microcomputer. A Radio Shack, TRS 80 Level II microcomputer is used to control reflectometer movement and data acquisition. The TRS 80 Level II system consists of a keyboard, 48k memory, floppy disc input, cathode-ray tube (CRT) display, and a line printer. The controller/operator program is loaded into the computer from a floppy disc. The

Figure 6. Colorado test site criteria matrix.

Date of Pavement Installation	Concrete	Asphalt*				
	Portland Cement	3/4" Max. on existing, solid, base PMSC Plant Mixed Seal Coats (Surface Coating)		3/4"+ layers (multiple layers) on old, poor base structure, HBP Hot Mix Bituminous Pavement (Overlay)		
		A	B	E	EX	F
New 1977 to Present				3/4"	1/2" $\boxed{3}$	
Middle Age 1972 to 1977						
Old Prior to 1972						

A - Open Grade; B - Medium Grade (Similar to Open Grade in Other States);
E - Medium Grade; F - Fine Grade (See Table 3).

Figure 7. Typical measurement locations at each test site.



computer output (data) is displayed on the CRT as it is being stored in memory. Once the data have been collected for a single test location, it can be printed on the line printer and stored on a RS232 compatible digital cassette tape recorder. The raw data, which are recorded on a cassette tape, can be read into another computer for conversion to luminance factors.

TEST SITE SELECTION

The following is a list of parameters for test site selection:

1. Minimum of 3000 average daily traffic (ADT); prefer 5000 ADT or more;
2. Minimum of two lanes in the same direction;
3. Roadway surface visible at least 1500 ft in front of the test site;
4. Preferable that recent skid test data and/or outflow data exist for the test site; and
5. Test locations at the test site must be flat.

The two major variables are pavement type and the age of the pavement. A matrix of variables results in the development of Figure 6. A new pavement is defined as two years old or less. A middle-aged pavement is two to seven years old, and an old pavement is greater than seven years. These definitions were based on meetings between the research team and representatives of the Colorado Department of Highways (CDOH) and FHWA.

The test sites should represent variations in surface macrotexture and microtexture, aggregate size, surface finish grading, and mixture. Variation in climatic conditions such as seasonal changes, relative humidity, temperature (surface and ambient), and relative humidity need to be considered.

TEST LOCATION

Gonio-reflectometer measurements should be taken at a minimum of three test locations (Figure 7) per test site. The first test location (no. 1) is near the right-hand edge of the driving lane. This location is expected to show the least amount of wear due to lower traffic use. The second location (no. 2) is the right-hand tire track, which is expected to be the greatest area of change because of the greater traffic use. The third location (no. 3) is between the two tire tracks (midtrack) in the driving lane. This location is expected to have an intermediate level of use, since it is most commonly used during lane changes.

Figure 7 shows the recommended test locations for sand patch and BPT measurements. A minimum of six measurements for sand patch and six for BPT is recommended with three in front of the gonio-reflectometer test locations and three behind.

TESTING PROCEDURE

The testing procedure can be described in three phases or parts. The first phase involves the pretraffic control setup for closure of the right-hand lane. The second phase involves data collection with the gonio-reflectometer, sand patch, and BPT. The third phase consists of posttraffic control, which involves removal of the barricades and signs.

The pretraffic control consists of the placement of signs and barricades to close the right-hand lane of traffic. Sign and barricade placement must meet the guidelines of the local highway safety engineer. Once the lane-closure guidelines have been established, a pretest site visit allows the position of each of the signs and barricades to be marked with safety orange paint. Pretest marking of sign and barricade placement speeds up placement on subsequent visits.

Before the first data-collection visit, the test site and test locations are determined during the pretest site visit. The pretest site visit involves the preparation and marking of the three gonio-reflectometer test locations, the six sand patch test locations, and the six BPT test locations. Once the right-hand lane is closed, the pretest site preparation begins. The three gonio-reflectometer test locations are selected based on macrotexture, levelness, and consistency of pavement surface. A template (Figure 8), which has three machined steel drill guides that match the three gonio-reflectometer support pads, is placed at the test location. Three holes are drilled into the pavement by using the drill guides in the template. Round magnetic stainless-steel pins are driven into the drilled holes in the pavement. These pins will be used in subsequent site visits to allow for more precise positioning of the gonio-reflectometer at each of the test locations. The pins are painted with safety orange paint. Orange markings are painted along the shoulder to help identify pin, sand patch, and BPT locations. This completes the initial or pretest site visit.

The data-collection phase begins during the next site visit once the right-hand lane has been closed. The gonio-reflectometer is removed from the transport vehicle and made ready for operation. The test locations are prepared by a second member of the research team. Test-location preparation consists of washing the reflectance measurement area with a mild soap solution, then thoroughly rinsing and air-drying the area. Once the area has dried, the pavement marker template is positioned over the three stainless-steel pins. The template is used to position a circular masking disc over the measurement area (Figure 8). With the masking disc in position, the template is removed and the pavement area enclosed by the gonio-reflectometer is sprayed with a fast-drying flat black paint. The masking disc is removed (Figure 9), which leaves the natural pavement in the measurement area clean and ready for measurements. The flat black paint is used to minimize stray light from reaching the photodiode.

While the test locations are being prepared, the project director is connecting the various computer components and the gonio-reflectometer. The gonio-reflectometer is positioned on the calibration platform and the operating program is read into the computer. The reflectance working standard, which was calibrated against a known National Bureau of Standards reflectance tile, is placed in the support block that is fixed to the calibration platform (Figure 10). A reading is taken by the computer of the calibration valve.

The gonio-reflectometer is moved from the calibration platform to the first test location. To prevent light leaks between the base of the gonio-reflectometer and the pavement, a black velvet cloth

Figure 8. Base pin template and hammer drill.

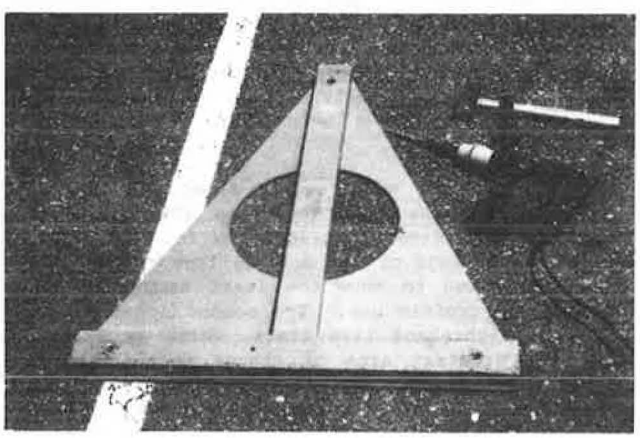


Figure 9. Painted test location and painting disc.

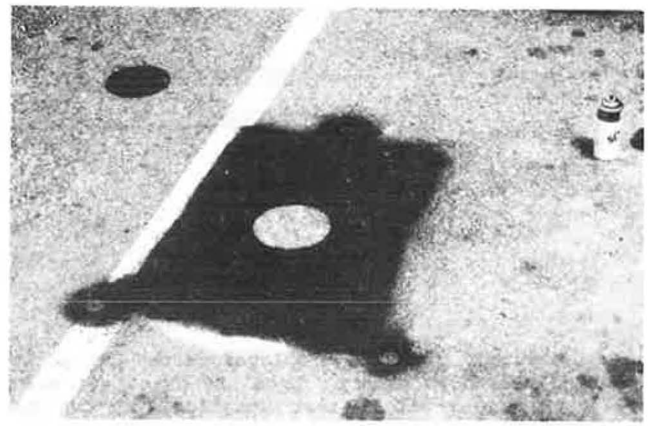
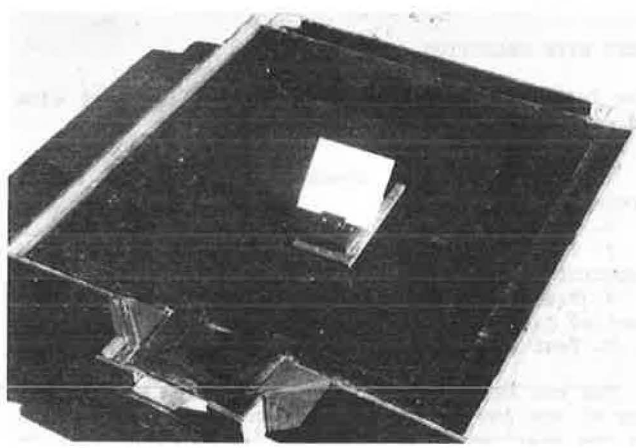


Figure 10. Working standard in calibration platform.



is placed around the base of the instrument (Figure 11). The operating program is initiated and the reflectance measurement sequence begins. The operating program moves the light source in preprogrammed steps and records the data. Figure 12 shows the measurement angles and increments for both the horizontal (azimuth) and vertical (declination) angles. This results in 44 horizontal planes and 34 declination angles per horizontal plane for a total of 1496 data points. Once the data are collected, they are printed on the line printer and recorded on a cassette tape for later analysis. This completes the reflectance measurement at a single test location. The gonio-reflectometer is moved back to the calibration platform and the entire process is repeated for each of the remaining two test locations at each test site.

While the gonio-reflectometer measurements are being taken, one assistant is performing sand patch measurements at each of the six sand patch test locations and another assistant is making BPT measurements at the six BPT test locations. The sand patch data are recorded on a field data sheet, as shown in Figure 13. Figure 14 shows a field data sheet used with the BPT. Field test site data and testing conditions are recorded on a project test data sheet (Figure 15).

Once all the data have been recorded and the equipment stored in the transport vehicle, the painted test position markers are repainted with orange paint. The posttraffic phase is completed when the signs and barricades are removed. This completes a single test site visit.

Figure 11. Ginio-reflectometer in testing position.

Figure 12. Measurement angles and increments.

Horizontal Angles (h)	Vertical Angles (V_S)
0° to 140°, 10° 145° to 220°, 5° 230° to 350°, 10°	0° to 70°, 5° 71° to 89°, 1°

CONCLUSIONS

The purpose of this paper is to describe the instrumentation and methodology for making pavement reflectance measurements. Modifications to the reflectometer electronics are currently under way. The

Figure 13. Sand patch field data sheet.

Test Site No.: 3

Test Date: 11/6/80

Pavement Type: Asphalt-EX

Remark:

Sand Patch Location No.	Diameter of Circle (cm) Mean Individual Rad. 1	2	3	4	5	Mean	Radius of Circle (cm)	Texture Depth (mm)	Test by
1	30	30.5	29.5	30	30	30	15.0	.376	AG
2	31.5	32	31	32	30.5	31.4	15.7	.343	AG
3	30	29.5	29	30	31	29.9	14.95	.378	AG
4	27	27	28	29	28	27.8	13.9	.437	AG
5	34	34	34	34	34	34	17.0	.292	AG
6	30	30	29.5	28	29.5	29.4	14.7	.391	AG

Figure 14. BPT field data sheet.

Test Site No: 3

Test Date: 1/6/80

Pavement Type: Asphalt-EX

Remark:

Location No.	Surface Texture	Temperature of Water (°C)	Skid Resistance Mean Individual Rad. 1	2	3	4	5	Mean
1	E	20.79	68	67	68	68	68	67.8
2	E	20.03	65	65	64	65	65	64.8
3	E	20.57	63	63	65	65	63	63.8
4	E	16.18	63	63	62	63	63	62.8
5	E	17.41	60	59	60	61	61	60.2
6	E	17.63	67	67	68	68	68	67.6

Figure 15. Sample field test data sheet.

FHWA
Contract DOT-FH-11-9482
Project Test Data Sheet
File Name - D00311

Test Site No. 3 Test Date 11/06/80 Project Director RNH

Weather Conditions Warm Pavement Type EX-ASPH

Tape I.D. D12B BPT Sheet No. 11 SP Sheet No. , 11

SEQ NO.	CAL NO.	LAMP	AMB-TEM	PAV-TEM	BOX-TEM	DIODE-TEM	TIME
317	14464.	7568.	22.20	17.54	18.80	.00	1115
327	14232.	7308.	22.20	19.19	21.70	.00	1205
337	13880.	7440.	27.20	22.57	25.50	.00	1345

BPT SUMMARY

LOC NO.	SURFACE TEXT	H20 TEMP	MEAN SKID RESIST
1	E	20.79	67.96
2	E	20.03	64.81
3	E	20.57	63.91
4	E	16.18	63.04
5	E	17.41	60.68
6	E	17.63	68.13

SP SUMMARY

LOC NO.	MEAN DIAM	RADIUS CIRCLE	TEXTURE DEPTH
1	30.00	15.00	.38
2	31.40	15.70	.34
3	29.90	14.95	.38
4	27.80	13.90	.44
5	34.00	17.00	.29
6	29.40	14.70	.39

modified gonio-reflectometer will be used in subsequent research projects to collect in situ reflectance data by using the methodology described in this paper. Collection of reflectance, sand patch, and BPT data in conjunction with existing laboratory data can result in the development of a pavement reflectance classification system. The pavement reflectance classification system can be used to establish the reflecting properties of a roadway surface. Luminance factor data are used to calculate the luminance of the pavement, which is critical to object visibility on the roadway. Pavement luminance (background luminance) affects adaptation state and contrast discrimination, which in turn affect visibility and safety on the highway at night.

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Automated Facility for Measurement of Pavement Sample Reflectance Characteristics

M.G. BASSETT, S. DMITREVSKY, P.C. KREMER, AND F.W. JUNG

Design methods for roadway lighting based on the concept of luminance require information on the light-reflecting properties of pavement surfaces. To evaluate and classify typical pavement materials used in North America, an automated test facility was designed and built at the University of Toronto for the Ontario Ministry of Transportation and Communications. The reliability of the measurements has been assessed in terms of equipment stability and the measuring procedure followed. Results indicated that the measuring accuracy was not limited by the equipment but by the measurement procedure. An estimate of system accuracy based on the limited number of samples measured to date is included. Measurements of the variation of pavement characteristics with variation of angle of observation showed that the direction of traffic need not be identified with accuracy greater than ±5°. Measurements were also made on the effect of the sample size on the characteristics. Results indicated that, for samples with a particle size of about 3 mm, two 15-cm-diameter samples that yield four sets of data would provide reliable values for the relevant characteristics. A measuring program to assess a larger number of samples and a variety of surface textures is in progress.

Roadway lighting design methods of past decades were based on illuminance (i.e., incident light), and simple manual calculations were possible by using precalculated utilization curves issued for each luminaire.

Design methods based on the concept of luminance (1) (i.e., reflected light) are more closely related to the night driving task than illuminance but were not generally used because of the complexity of the calculations. The development of the electronic computer has removed this obstacle, and luminance methods are now being incorporated in design procedures. This new approach, however, requires the evaluation and classification of pavement surfaces with respect to their light-reflecting properties (2).

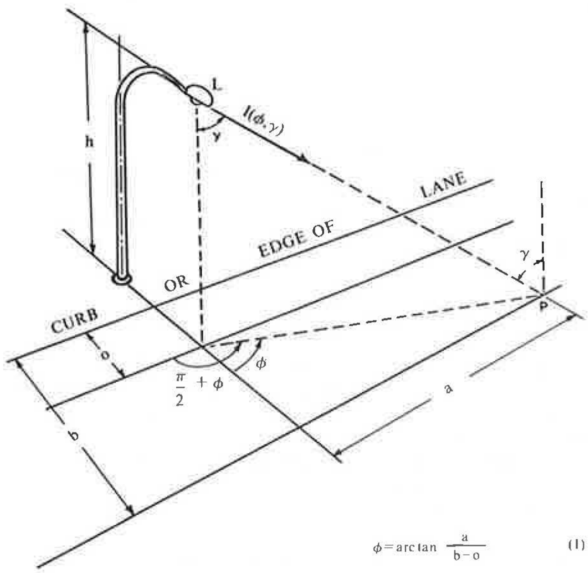
An automated apparatus for the measurement of the reflection properties of highway pavement samples has been developed at the University of Toronto for the Ontario Ministry of Transportation and Communications (3). The measurements are made in accordance with the recommendations of the International Commission on Illumination (CIE) (4). A pilot sequence of measurements on approximately 30 samples was carried out to determine both the equipment accuracy and the scattering of the measured data for different specimens of the same pavement.

LUMINANCE CONCEPT IN COMPUTER METHODS OF ROADWAY LIGHTING DESIGN

The point-by-point calculation of illuminance at a grid point P (E_p) is shown in Figure 1. Luminance is calculated from the corresponding illuminance value for the same grid point (P), but only that portion of the light that is reflected toward the driver's eyes is considered. As shown in Figure 2, the illuminance contribution (E_p) from each luminaire is multiplied by a coefficient q, which depends on the light-reflecting properties of the pavement surface. For each driver position, or each lane, the calculated luminance arrays are different (unlike illuminance arrays, which remain the same).

The luminance coefficient (q) is defined as the ratio between the luminance at point P and the horizontal illuminance at the same point (E_p). It is a function of four angles: α, β, γ, and δ, as shown in Figure 2. Thus,

Figure 1. Illuminance parameters.

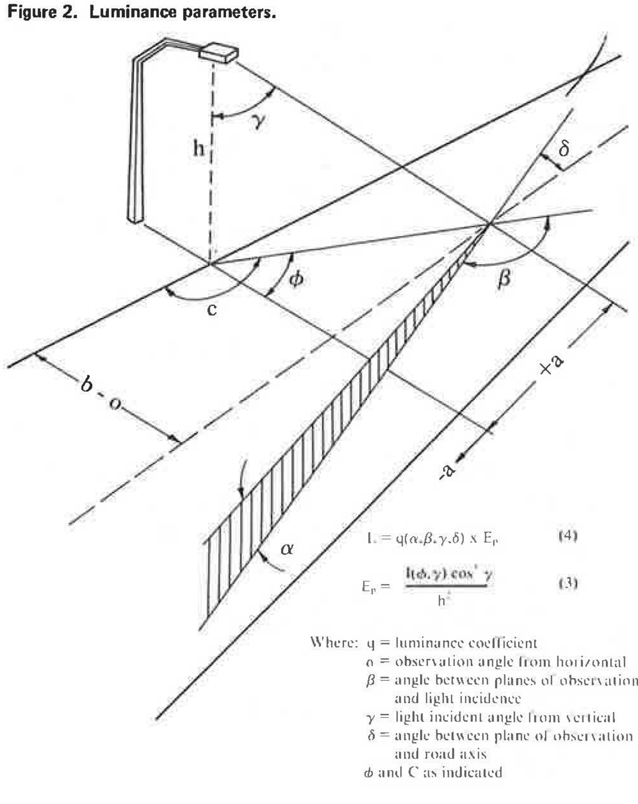


$$\phi = \arctan \frac{a}{b-o} \qquad (1)$$

$$\gamma = \arctan \frac{\sqrt{a^2+(b-o)^2}}{h} \qquad (2)$$

$$E_p = \frac{I(\phi,\gamma)\cos^3\gamma}{h^2} \qquad (3)$$

Figure 2. Luminance parameters.



$$L = q(\alpha,\beta,\gamma,\delta) \times E_p \qquad (4)$$

$$E_p = \frac{I(\phi,\gamma)\cos^3\gamma}{h^2} \qquad (3)$$

Where: q = luminance coefficient
α = observation angle from horizontal
β = angle between planes of observation and light incidence
γ = light incident angle from vertical
δ = angle between plane of observation and road axis
φ and C as indicated

$$L = q(\alpha, \beta, \gamma, \delta) \times E_p \tag{1}$$

where q is the luminance coefficient [$(cd/m^2)/lx$], being a function of the angles α, β, γ, and δ for each particular pavement surface, and E_p is illuminance. In the metric system, the luminance coefficient [expressed as $(cd/m^2)/lx$], has a maximum value for perfect white diffuser of $q_{max} = 1/\pi = 0.318$.

At each grid point P, values of luminance must be calculated for each luminaire and then added together. The luminance created by one luminaire at a given point (subscript i) according to Equation 1 is as follows:

$$L = q(\alpha_i, \beta_i, \gamma_i, \delta_i)\, E_p\, (\phi_i, \gamma_i) \tag{2}$$

This is in accordance with Figure 2.

Adding the values for n luminaires, substituting for E_p, neglecting the influence of δ, and setting $\alpha = 1°$, the following equation for luminance is obtained:

$$L = \sum_{i=1}^{n} q(\beta_i, \gamma_i) \left\{ [I(\phi_i, \gamma_i) \cos^3\gamma_i] / h^2 \right\} \tag{3}$$

In conjunction with the work that went into the CIE recommendations, standard reflectance tables have been established in the form of reduced coefficients: $r = q \cos^3\gamma$ for $\alpha = 1°$. This combination of $\cos^3\gamma$ and q simplifies the reflectance measurements. Also, the combination $r = q \times \cos^3\gamma$ leads to tabulated values of r, which decrease with an increase in γ or $\tan\gamma$, whereas the pure reflectance function (q) alone increases tremendously for β being equal to or close to zero (5).

The number (n) of luminaires to be taken into account should include a longitudinal distance of 12 x h or more beyond the point P. Additional luminaires beyond this range contribute insignificantly. By substituting $r = q \times \cos^3\gamma$, Equation 3 can be rewritten as follows:

$$L = \sum_{i=1}^{n} \left\{ [I(\phi_i, \gamma_i) r(\beta_i \tan\gamma_i)] / h^2 \right\} \tag{4}$$

CLASSIFICATION OF ROAD SURFACES

The results of measurements from a sample pertaining to a particular road surface are tabulated in terms of $\tan\gamma$ and β and can be used as a design input after storage in a computer memory array. Figure 3 represents a typical array for a dark asphalt pavement surface. CIE-sponsored research in Europe has addressed the question of classifying such measurements into a manageable number of specularity classes. Usually there are four classes for dry pavements, although some suggestions go as far as eight in this category. More recently, there has been work done on four additional classes pertaining to wet pavement surfaces (6,7).

The most commonly known dry classes are designated RI, RII, RIII, and RIV, and an alternative system is known as N1, N2, N3, and N4--all proposed for European-type pavements (4,8). In the United States and Canada, the same or similar classes may apply, which will be subject to some research investigations. The example in Figure 3 is for RIII.

Classification of measured road surfaces is carried out by using certain parameters, which are Q0 (or q_0), S1, and S2, where Q0 or q_0 equals an average luminance coefficient over a defined roadway ground area or space angle; S1 = r(0,2)/r(0,0) (i.e., $\beta = 0$, $\tan\gamma = 2$; and $\beta = 0$, $\tan\gamma = 0$); and S2 = Q0/r(0,0).

The value Q0 can be understood as a parameter correlated to the average luminance of the pavement surface. The values S1 and, especially, S2 are parameters that indicate the degree of specularity.

It is stipulated that surfaces with identical parameters are equivalent, with very little differ-

Figure 3. Standard reflection table R-III.

tan γ \ β°	0	2	5	10	15	20	25	30	35	40	45	60	75	90	105	120	135	150	165	180
0	294	294	294	294	294	294	294	294	294	294	294	294	294	294	294	294	294	294	294	294
0.25	326	326	321	321	317	312	308	308	303	298	294	280	271	262	258	253	249	244	240	240
0.5	344	344	339	339	326	317	308	298	289	276	262	236	217	204	199	199	199	199	194	194
0.75	357	353	353	339	321	303	285	267	244	222	204	176	158	149	149	149	145	136	136	140
1	362	362	352	326	276	243	226	204	181	158	140	118	104	100	100	100	100	100	100	100
1.25	357	357	348	298	244	208	176	154	136	118	104	83	73	70	71	74	77	77	77	78
1.5	353	348	326	267	217	176	145	117	100	86	78	72	60	57	58	60	60	60	61	62
1.75	339	335	303	231	172	127	104*	89	79	70	62	51	45	44	45	46	45	45	46	47
2	326	321	280	190	136	100	82	71	62	54	48	39	34	34	34	35	36	36	37	38
2.5	289	280	222	127	86	65	54	44	38	34	25	23	22	23	24	24	24	24	24	25
3	253	235	163	85	53	38	31	25	23	20	18	15	15	14	15	15	16	16	17	17
3.5	217	194	122	60	35	25	22	19	16	15	13	9.9	9.0	9.0	9.9	11	11	12	12	13
4	190	163	90	43	26	20	16	14	12	9.9	9.0	7.4	7.0	7.1	7.5	8.3	8.7	9.0	9.0	9.9
4.5	163	136	73	31	20	15	12	9.9	9.0	8.3	7.7	5.4	4.8	4.9	5.4	6.1	7.0	7.7	8.3	8.5
5	145	109	60	24	16	12	9.0	8.2	7.7	6.8	6.1	4.3	3.2	3.3	3.7	4.3	5.2	6.5	6.9	7.1
5.5	127	94	47	18	14	9.9	7.7	6.9	6.1	6.7										
6	113	77	36	15	11	9.0	8.0	6.5	6.1											
6.5	104	68	30	11	8.3	6.4	5.1	4.3												
7	95	60	24	8.5	6.5	5.2	4.3	3.4												
7.5	87	53	21	7.1	5.3	4.4	3.6													
8	83	47	17	6.1	4.4	3.6	3.1													
8.5	78	42	15	5.2	3.7	3.1	2.6													
9	73	38	12	4.3	3.2	2.4														
9.5	69	34	9.9	3.8	3.5	2.2														
10	65	32	9.0	3.3	2.4	2.0														
10.5	62	29	8.0	3.0	2.1	1.9														
11	59	26	7.1	2.6	1.9	1.8														
11.5	56	24	6.3	2.4	1.8															
12	53	22	5.6	2.1	1.8															

Values tabulated with $\bar{Q}_0 = 0.07$

S1 = 1.11, S2 = 2.38

Note: Table of reflectance data: $q \cos^3\gamma \times 10^4$

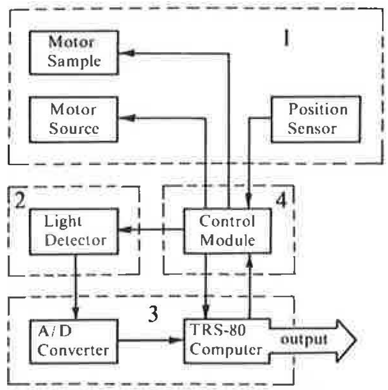


Figure 4. System block diagram.

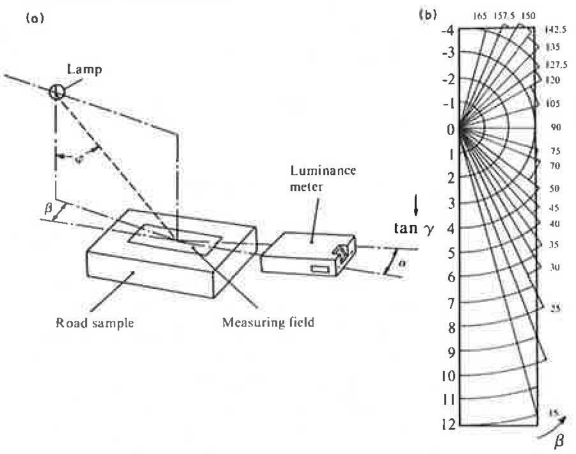


Figure 5. (a) Coordinates of lamp sample luminance meter system and (b) range of angles β and γ.

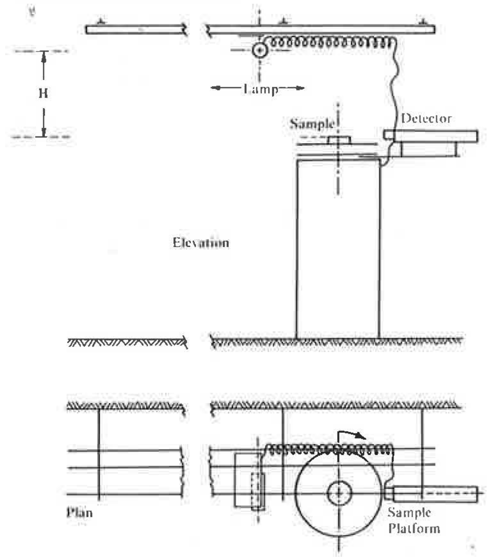
ences in the remaining r values, as measured and tabulated in the format of Figure 3.

DESCRIPTION OF APPARATUS

The Ontario road reflectance matrix photometer is an instrument designed to measure r-values as a function of β and tanγ. The apparatus consists of four major subsystems, as shown in Figure 4 and described below:

1. A mechanical system with photoelectric sensors that position the light source and the sample in accordance with the specified range of parameters β and γ as defined in Figure 5, thereby maintaining a constant height of the source.
2. An optical detection system that is rigidly attached to the sample and maintains the constant viewing angle α [see Figure 6 (note that scale is approximately 1:20)]. The core of this system is a 5x0.15-cm sector of a spherical lens of 35-cm focal length with a 1x0.5-mm iris located in the focal plane and an RCA 6217 photomultiplier tube, the cathode of which is located in the proximity of the focal plane. The dimensions of the lens sector define a field of view of approximately 5x8 cm while the focal plane iris defines the acceptance angle of 5' in the vertical and 10' in the horizontal planes.
3. An integrated data-acquisition and dedicated microcomputer system that stores the elements of the measured r-matrix, applies appropriate weighting factors, numerically integrates them, and evaluates the parameters S1, S2, and Q0. The output is in the form of a printout of the r-matrix with the associ-

Figure 6. Apparatus layout.



ated values of the three derived parameters. If desired, there is a provision for storing the output on an audio tape. This feature is about to be augmented by providing the option for storing the output on a magnetic disc.
4. A control module that uses the primary inputs from the photoelectric sensors of the mechanical subsystem and, in conjunction with the microcomputer, governs the operation of the other subsystems.

The module operates in an open-loop mode. Experience has shown that the intrinsic stability of the system is of the order of 1 percent in a controlled environment (temperature and humidity). The introduction of a closed-loop control system with continuous calibration monitoring is currently under consideration.

APPARATUS PERFORMANCE

The major performance parameters of the system are described in this section. The light source employed is a 400-W quartz halogen lamp moving on horizontal rails 60 cm above the sample level. The range of lamp movement is from directly above the sample to approximately 7 m away. With the integration time of about 15 ms, the signal-to-noise ratio provided by the detector employed (RCA 6217 photomultiplier) was about 10 dB for the weakest signal observed.

The choice of size of the field of view was dictated by the requirement that the equipment be designed to operate on circular samples of about 15 cm in diameter. The relative sizes of the field of view and the sample allow one to obtain two independent sets of measurements from one sample.

Experience has shown that the measurement cycle for one sample (two viewing fields), which includes mounting and alignment time, is about 1 h.

ASSESSMENT OF MEASUREMENT RELIABILITY

The reliability of measurements by the apparatus is affected by two phenomena:

1. Equipment stability and
2. Measurement procedure, such as mounting and alignment of the sample.

Table 1. Standard deviation of displaced measuring field characteristics.

Sample	Q0/$\bar{Q}$0			S1/$\bar{S}$1			S2/$\bar{S}$2		
	Left	Right	Δ (%)	Left	Right	Δ (%)	Left	Right	Δ (%)
EB 2-1	1.03	0.97	4.3	1.04	0.95	6.4	1.00	1.00	0
EB 6-1	1.01	0.99	1.4	0.95	1.06	7.8	0.98	1.02	2.8
EB 6-2	1.05	0.94	7.8	1.02	0.97	3.5	1.07	0.92	10.5

Table 2. Standard deviations of relative values of displaced measuring field characteristics evaluated for varying number of samples.

No. of Samples	Δ(Q0/$\bar{Q}$0) (%)	Δ(S1/$\bar{S}$1) (%)	Δ(S2/$\bar{S}$2) (%)
3	4	4.7	4.9
2 (EB2-1, EB6-1)	2.6	5.8	1.6
2 (EB6-1, EB6-2)	4.6	4.9	6.3
2 (EB2-1, EB6-2)	5.1	4.1	6.1

Equipment stability was tested by performing repeated measurements on a pavement sample in situ in the apparatus. The standard deviations for the characteristics Q0, S1, and S2 based on four measurements were as follows: Q0 = 0.7 percent, S1 = 2.2 percent, and S2 = 0.9 percent.

The repeatability of measurements on a sample being removed and remounted was checked on 13 samples. The standard deviations of relative variations of the three characteristics were as follows: Q0 = 2.7 percent, S1 = 5.9 percent, and S2 = 4.0 percent.

Comparison of the two sets of standard deviations indicates that the measurement accuracy is not limited by the equipment but by the measurement procedure.

Assuming that the values given above are good approximations of their limits for a large number of measurements, the results obtained can be interpreted by stating that the ranges of relative deviations from the means that occur with 95 percent probability are limited by the following values: 5.5 percent for Q0, 12 percent for S1, and 8 percent for S2.

It was then deemed desirable to test the sensitivity of the measured data with respect to the variation of the direction of observation or, from the operational point of view, the accuracy with which the traffic direction has to be specified. To this end, measurements were performed on samples in the normal traffic direction (0°) and rotated ±5° from this nominal position. The average values of the standard deviations for the three directions of observation were as follows: Q0 = 3.5 percent, S1 = 5.3 percent, and S2 = 3.5 percent.

The standard deviations for the S1 and S2 values for the misorientated observation directions, although close, are somewhat smaller than the corresponding set of values for the repeatability measurements. This may be due to the fact that 13 samples were measured in the repeatability measurements and 25 in the misorientation measurements. These results indicate that the traffic direction need not be specified with an accuracy greater than ±5°.

The next feature of the measurement process to be considered was the effect of the size of the viewing field on the results obtained. Measurements were made of two adjacent measuring fields on three of the samples investigated. Tables 1 and 2 list the relative value of the characteristics Q0/$\bar{Q}$0, S1/$\bar{S}$1, and S2/$\bar{S}$2 for each sample. Also included in the tables are the standard deviations of the relative values evaluated for all six measurements of the three samples and for three groups of four measurements on three different pairs of samples. The standard deviations range from 1.6 to 6.3 percent.

The particle size in the surfaces measured was of the order of 3 mm, and from the above data it is estimated that measurements performed on two 15-cm-diameter samples that yield four sets of data would provide reliable values for both the parameters (Q0, S1, and S2) and the elements of the r-matrix. The accuracy of these measurements would be consistent with the accuracy of the equipment and the measuring procedure.

No results are available currently for surfaces with a larger particle size. It is expected that either a larger field of view may be required or a larger number of samples would be necessary to obtain reliable data. The latter option would be preferable due to the convenience of obtaining and handling smaller-sized samples.

PILOT MEASUREMENTS FOR ONTARIO PAVEMENTS

The Lindsay, Ontario, test sections (9) were chosen to carry out pilot measurements on pavements of known composition. About 20 samples were measured from various sections, and each sample was measured at least three times in slightly changed angular position (+5°, 0°, and -5°). Comparing the results with the Erbay Atlas data in Figures 7a and b, it was found that, in terms of specularity (S1), the surfaces range from S1 = 0.49 to S1 = 1.45 (i.e., from below the standard surface RII to above RIII). The most specular surfaces contain at least 60 percent limestone coarse aggregates, and the least specular surfaces contain more than 70 percent igneous rocks. In the latter, the igneous stone projections feel gritty; in the former, the limestone projections feel smooth and polished. The surfaces were exposed to moderate traffic from summer 1978 to fall 1980.

The test sections had been skid tested by the old, nonstandard skid trailer in summer 1980. In Figure 8, the skid numbers (SNs) from these tests are plotted versus the specularity parameter (S1).

There is a weak negative correlation (-0.64) between the SN taken at 80 km/h and the parameter S1. Stronger correlations may be expected between low-speed or "zero-speed" SNs and S1. In any case, it is certain that the degree of specularity of pavement surfaces is related to the degree of smoothness or polish of coarse aggregates.

CONCLUSIONS

The Ontario road reflectance matrix photometer, as designed and tested to date, has provided measurement results that are well within the accuracy of existing design methods. The instrument provides reliable, rapid, and automatic measurements of pavement samples as small as 15 cm in diameter. Although the viewing angle (α) is fixed at 1° in the present apparatus, modifications are in progress to vary the range of α.

The availability of this test facility with its rapid measuring time and small sample size should assist highway agencies in the acquisition of relevant data for modern highway lighting design by the luminance method.

Figure 7. Plot of log S1 versus log S2.

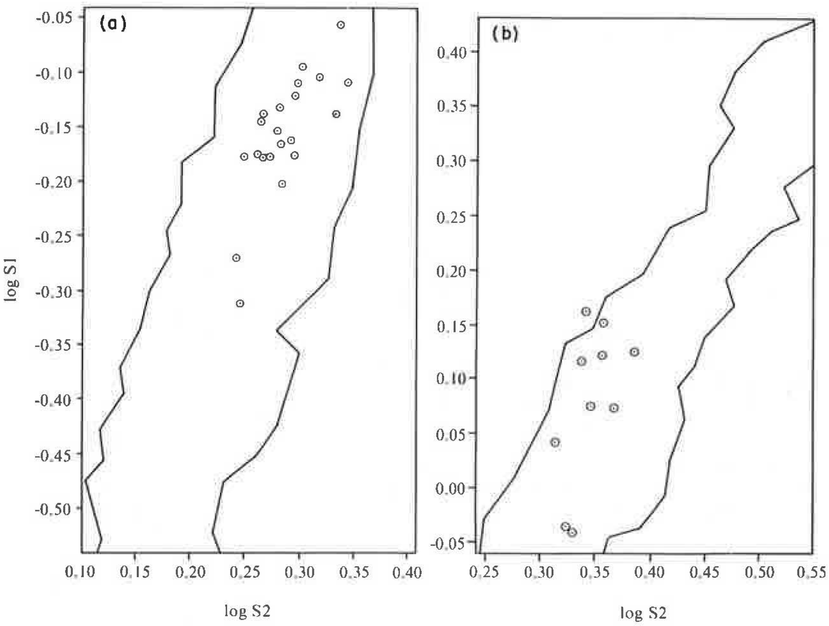
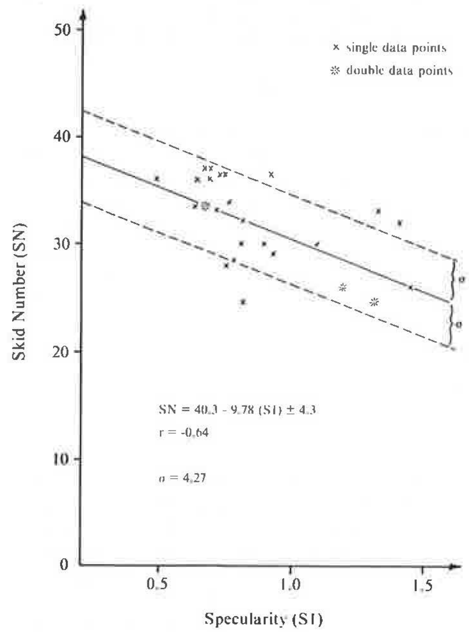


Figure 8. Plot of SN versus specularity.



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Effects of Partial Lighting on Traffic Operations at a Freeway Interchange

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The objective of this paper is to report the results of an experiment that evaluated the effects of partial interchange lighting, complete interchange lighting, and no lighting on traffic operations at a freeway interchange. A freeway interchange that possessed a modern and complete lighting system was chosen for the experiment. The lighting was temporarily modified so that two levels of partial lighting and unlit conditions could be provided. Traffic operational data that consisted of velocities, accelerations, brake occurrences, gore and shoulder encroachments, high-beam use, and diverging and merging patterns were collected for five study conditions—daylight, complete lighting, partial lighting (two types), and no lighting—for both an exit and an entrance ramp by using a newly designed data-collection system capable of recording complete trajectory information on individual vehicles. The results indicated that complete interchange lighting is superior to partial interchange lighting in providing smoother and safer nighttime operations at the interchange.

As a means of facilitating the driving task and reducing the potential for accidents, partial lighting of interchanges has been used for areas where complete or continuous lighting was not deemed to be justified. The use of partial lighting is based on the premise that it will provide many of the benefits attributable to complete interchange lighting at lower costs. This premise has never been substantiated, and there are many who question the effectiveness of partial lighting. There has been a long-standing need for information that could form a basis for guidance concerning the effectiveness and conditions that favor the use of partial lighting of interchanges.

RESEARCH OBJECTIVE

The objective of this research, which was sponsored by the National Cooperative Highway Research Program (NCHRP), is to determine the effectiveness of partial lighting of interchanges in comparison with complete interchange lighting and no lighting and to develop recommendations for its use. For purposes of this research, partial lighting is defined as lighting that consists of a few luminaires located in the general areas where entrance and exit ramps connect with the through traffic lanes of the freeway.

To accomplish part of this objective, an interchange with appropriate geometric design, traffic flow, and a modern complete interchange lighting (CIL) system was identified. Permission was obtained from the highway lighting authority to temporarily modify the lighting system so that both partial interchange lighting (PIL) and no-lighting conditions could be provided during any chosen night at the interchange. In this manner, all other variables (geometry, traffic, environment, etc.) remained fixed so that the effect of the different lighting conditions on traffic operations could be assessed.

This paper is a report on the results of an experiment that evaluated the traffic safety and operational effects of PIL, CIL, and no lighting on traffic operations at a freeway interchange located near Philadelphia.

The remainder of this paper is organized into five parts: (a) experimental design, which included site selection, definition of independent variables and selection of dependent measures; (b) design and development of the data-collection system; (c) installation of equipment and data collection; (d) data analysis; and (e) results and conclusion.

EXPERIMENTAL DESIGN

Site Selection

A freeway interchange was required that possessed the following characteristics:

1. Modern CIL lighting system that could be physically modified to produce a variety of configurations of PIL and no-light conditions;
2. Cooperative authorities who would allow us to temporarily change the lighting from CIL to PIL and no lighting;
3. Modern geometric design with good markings, signing, surfaces, and shoulders and no severe grades or curves;
4. Suburban environment;
5. Sufficiently high night traffic volumes to allow for a high data-collection rate (200 vehicles/night); and
6. Close proximity to our offices.

A site was selected on the Pennsylvania Turnpike (PATPK) at the junction of I-276 (east-west route) with PA-9 (the turnpike's northeast extension). (This is not a tolled exit but a true interchange.) The interchange is a three-leg design that satisfies all of the criteria and was only 5 miles from our offices. Figure 1 illustrates the site. The lighting consists of 250-W high-pressure sodium (HPS) luminaires at a 30-ft mounting height and spaced about 180 ft apart.

Selection of Independent Variables

PIL Configurations

Based on the results of a nationwide survey of interchange lighting practices (1), two PIL configurations were selected. These included two lights per ramp, called PIL-2 (the California system), and four lights per ramp, called PIL-4 [the American Association of State Highway and Transportation Officials (AASHTO) system]. The location of these luminaires was derived from the same survey. They are illustrated in Figure 2 for the exit ramp. (The exit ramp is used for illustrative purposes; the entrance ramp configurations are almost identical.)

Independent Variables

The primary independent variable was the lighting configuration (day, CIL, PIL-4, PIL-2, and no light), and this was quantified by the following photometric measures: number of luminaires (and total flux) and average illuminance and uniformity. In the future, we will also determine glare (disability), pavement luminance (average), and visibility index.

Selection of Dependent Measures

The dependent measures selected for analysis cover a wide spectrum and include the following:

Figure 1. Pilot site (I-276/PA-9).

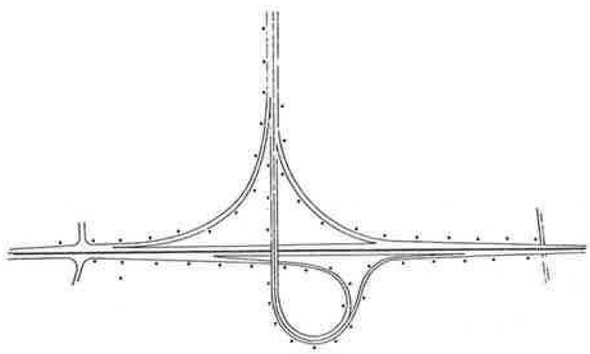
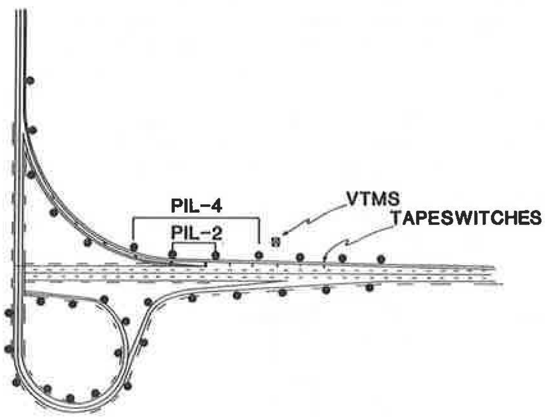


Figure 2. Exit ramp.



1. Velocity, acceleration, and acceleration noise in the deceleration and acceleration lanes and in the ramps themselves;
2. Diverge and merge distributions of drivers who use the exit and entrance ramp, respectively; and
3. Erratic maneuvers such as frequencies of braking, use of high beams, and gore and shoulder encroachments.

These measures can be stratified into two categories: those related to the beacon effect of interchange lighting (i.e., the lighting serves to identify where the interchange is) and those related to the driver-control effect of interchange lighting. The first type includes the diverge and merge point and velocity and acceleration in these areas, while the second type includes velocity and acceleration in the ramps and erratic maneuvers.

It was hypothesized that, as the lighting conditions were modified (CIL to PIL to no lighting), and since no other variables were changed, any measurable differences in the dependent measures would result from the change in lighting.

DESIGN AND DEVELOPMENT OF DATA-COLLECTION SYSTEM

The data-collection system designed and fabricated for this study--the vehicle trajectory measurement system (VTMS)--is a simplified version of the Federal Highway Administration (FHWA) traffic evaluator system (TES). VTMS has the capability of recording the time-position history of an individual lead vehicle traversing an instrumented section of roadway--in this case, the exit or entrance ramp. (TES has the capability of simultaneously recording the time-position history of all vehicles in a section of multilane roadway.) Because our experimental plan required the analysis of only individual lead vehicles (the behavior of following drivers is influenced more by the lead driver than by lighting), the VTMS was sufficient for our study.

VTMS consists of four parts: electronic logic and control circuit, high-speed printer, power supply, and roadway sensors that consist of tapeswitches and connecting cable. Seventeen switches were employed at each ramp: five single switches at 100-ft spacing in the acceleration and deceleration ramps and six pairs at 150-ft spacing in the ramps, beginning at the tip of the painted gore. The total length of roadway covered by the system was 1250 ft (Figure 2).

INSTALLATION AND USE OF VTMS

Determination of Instrument Placement

After visiting the site and observing traffic operations at both ramps, the tapeswitch configuration illustrated in Figure 2 was selected. The exact locations of these switches were dictated by both the type of measure (e.g., velocity and acceleration versus location), the availability (and cost) of connecting cable and tapeswitches, and past experience with data-collection systems of this type.

Installation of Equipment

Beginning about midnight on the day before data collection began, three Ketron personnel met with PATPK personnel who provided a heavy truck with a trailer-mounted flashing-arrow board, a pickup truck with an on-board flashing-arrow board, and three persons--two drivers and a flagman--to divert traffic. The tapeswitches were affixed to the roadway surface, in sequence, moving from upstream to downstream and covered with heavy duct tape. Traffic was either stopped temporarily (e.g., switches in the ramp) or diverted around the area (switches upstream of the ramp for the exit and downstream for the entrance).

Cabling was then placed along the shoulder, connections were made, and the system was tested. The entire procedure took about 4-5 h: 2-3 h for installation and the remainder for cabling and connections.

Data Collection

Data were collected by using a team of two observers. One observer, stationed upstream of the instrumented length of highway, announced to the other observer the arrival of each lead car vehicle. The second observer controlled and monitored the output of VTMS and also manually input data concerning erratic maneuvers onto the output of VTMS by pressing the appropriate push buttons on the manual input extension of the VTMS. The two observers communicated by citizen-band (CB) radio.

For the exit ramp (Figure 2), the upstream observer identified the lead car as it entered the deceleration lane. If no other lead car was being tracked by the system (i.e., within 1250 ft downstream), the vehicle's trajectory was tracked automatically. High-beam use and brake activations were spotted by either observer and recorded on the printer output by the downstream observer. The upstream observer kept a total count of all high-beam and brake use of all lead cars (whether tracked by VTMS or not).

Data collection continued for at least one entire night or for two nights if 200 vehicles were not ob-

Table 1. Study conditions.

	Sample Size		Day of Week	
Condition	VTMS	Manual	VTMS	Manual
Exit				
Day	204	216	Wednesday	Wednesday
CIL	147	189	Sunday	Sunday
PIL-2	153	212	Friday	Friday
PIL-4	269	321	Thursday/Saturday	Thursday/Saturday
No light	189	207	Wednesday	Wednesday
Entrance				
Day	176	732	Thursday	Wednesday
CIL	152	388	Monday	Wednesday
PIL-2	193	291	Saturday	Saturday
PIL-4	197	427	Friday	Friday
No light	176	334	Wednesday	Wednesday

served during the first night. For the exit, only the PIL-2 condition required two nights, and this resulted from a rain period during the first night. The PIL-4 condition for the exit was terminated after 212 data points were recorded by 11:00 p.m. because of a temporary system problem, and this is the only case with less than a full night's data.

Table 1 gives the 10 study conditions, the sample sizes, the number of nights, and the day of the week for each condition.

For the entrance ramp, location of the upstream observer proved to be problematic. During the first two lighting conditions (day and no lights), we noticed an exceedingly small percentage of brakers in VTMS data. This did not agree with observations made on three different nights by senior members of the research team. We repositioned the upstream observer and, after some trial and error concerning the exact position, finally determined an optimum placement. For this ramp, the manual observation (counts) of erratic maneuvers was normally performed on different nights than the nights on which the trajectory measurements were made. This was accomplished by recording total lead car volume and the number of lead cars that made erratic maneuvers.

Traffic volume counts on the main road and the ramps were initially desired, but malfunctions in the counters produced error-prone data, so the volumes were discarded. All nighttime study conditions had very similar traffic volumes and did not differ on different days by more than 10-15 percent at any given hour of the night. Daytime volumes were much heavier.

Data collection began on September 23, 1981, after deployment of VTMS on September 22. It continued through October 28. Rain caused complete washouts on four different nights (requiring redeployment of most of the switches) and halted data collection on three other nights. A major system failure, caused by nearby lighting, occurred on the second night of data collection for the entrance ramp and caused a two-week delay to obtain the necessary parts.

DATA ANALYSIS

The data analysis was divided into two parts: (a) manual analysis of high-beam use and brake-light occurrences and (b) computerized analysis of gore and shoulder encroachments, velocity, acceleration and acceleration noise, and the distribution of diverging patterns (for the exit) and merging patterns (for the entrance).

Manual Analysis

High-Beam Use

The table below gives the frequency of high-beam use by study condition for both the exit and entrance:

	Frequency (%)	
Study Condition	Exit	Entrance
Day	0	0
CIL	0	1.8
PIL-4	2.4	8.9
PIL-2	4.2	5.4
No light	5.8	12.6

As expected, the frequencies increased as the illumination decreased from day to CIL to PIL to no light. The greater use of high beams under PIL conditions is indicative of a lack of visibility (i.e., drivers activate their high beams to increase their seeing distance).

Brakers

The data on brakers were derived in two ways. For the exit, only VTMS-recorded lead cars were classified as either brakers or nonbrakers. For the entrance (because of observer placement problems described previously), all lead cars were classified as either brakers or nonbrakers (on nights other than those when VTMS was used). The type of data collected at the exit is a subset of the type collected at the entrance (it consisted of 75 percent of the early night volume and up to 95-100 percent of the late night volume).

The table below gives the braker data for both the entrance and exit:

	Frequency (%)	
Study Condition	Exit	Entrance
Day	40.1	74.0
CIL	45.0	73.2
PIL-4	49.1	85.9
PIL-2	48.5	84.8
No light	52.2	86.8

Again, as the lighting decreased, the percentage of brakers increased.

The higher frequencies of brake-light occurrences under the two PIL conditions are again indicative of a lack of visibility and an uncertainty associated with the task required to negotiate the interchange ramps.

The braker data were further classified by time period--early (8:00-11:00 p.m.) and late (1:00-5:00 a.m.)--which corresponded to higher and lower (than average) volumes. It was disclosed that the frequency of brakers normally increased during higher volumes and decreased during lower volumes (for the four night conditions).

Other Manual Analysis

We attempted to observe and record gore and shoulder encroachments manually, but this proved to be impossible under the PIL and no-light conditions because the gore markings could not be seen by the observers under these low-light conditions. These data were instead analyzed from VTMS data.

Computer Analysis

The vehicle trajectory data (time in milliseconds at which the front wheels of each vehicle encountered each switch) were manually transcribed onto coded data-reduction forms and then keyed to a disk file. Preliminary data screening was accomplished during the transcription process to remove those data records with gross errors.

The computer program was designed to perform four different operations: error checks, individual record calculations (for each vehicle), pooled data calculations (for each location of each study condition), and summary statistics (for each study condition), as follows.

Error Checks and Reconstruction

1. Data screening for miscodings or logical errors and reasonableness tests to identify times that are excessively out of range (e.g., a time of 5 s between two switches when the average time between preceeding switches is only 1 s), and
2. Reconstruction of time profiles when one or more switches were temporarily out of operation.

Individual Record Calculations

1. Compute spot speed across each pair of switches and average speed between consecutive individual switches or pairs for each record,
2. Compute spot acceleration from consecutive spot speed and average accelerations from consecutive average speeds for each record,
3. Compute average acceleration noise from consecutive average accelerations, and
4. Compute individual diverge or merge points for each vehicle that traverses the exit or entrance ramp and identify unusual or dangerous exit points from the system--both exit and entrance (i.e., a record in which the first switch hit was located downstream of the beginning of the gore on the exit ramp, or before the point of the gore on entrance ramp; these are equivalent to gore and shoulder encroachments).

Pooled Data Calculations

1. Compute means of spot speed and frequencies of switch hits across each pair of switches and compute means of average speed and frequencies between consecutive switches or pairs for each study condition,
2. Compute means of spot acceleration and frequencies between consecutive pairs of switches and compute means of average acceleration across three consecutive pairs or individual switches for each study condition,
3. Compute means of average acceleration noise across four consecutive switches or pairs for each study condition,
4. Compute mean diverge (merge) speed and frequencies at each diverge (merge) point for all study conditions for exit (entrance) ramp, and
5. Compute frequencies and locations of unusual exit points for all study conditions.

Summary Statistics

The following summary statistics were given for each study condition: number of records, accepted records, and errors (rejected data), and number of cars and trucks.

RESULTS

Eight sets of data were output by the computer program for each study condition:

1. Average velocity by location by study condition,
2. Average acceleration by location by study condition,
3. Average acceleration noise by location by study condition,
4. Spot velocity by location by study condition,
5. Spot acceleration by location by study condition,
6. Average diverge velocity (exit) and merge velocity (entrance) by location by study condition,
7. Distribution of diverging patterns (for exit) and merging patterns (for entrance), and
8. Frequencies of unusual (dangerous) exiting vehicles from VTMS (equivalent to gore and shoulder encroachments).

Velocity, Acceleration, and Acceleration Noise

It was hypothesized that, as the lighting conditions changed, a systematic measurable change in these variables would be noticed. This proved to be false. All three of these variables, whether computed as an average value between switch locations or as spot value at switch pairs, proved to be insensitive to the changes in the independent variable. Although certain differences were found (e.g., day velocities higher than night velocities; night no-light velocities greater than all night-lighted velocities), these differences were quite small (e.g., 1-3 ft/s) and did not distinguish between CIL and PIL conditions.

Diverge and Merge Patterns

The table below summarizes the diverging and merging patterns for the exit and entrance:

	Frequency (%)	
Study Condition	Exit (diverging late)	Entrance (merging early)
Day	51.0	37.5
CIL	20.0	36.2
PIL-4	25.5	47.7
PIL-2	26.1	47.2
No light	26.9	26.1

It is evident from the data that, at the exit, drivers diverge farther downstream under PIL conditions than under the CIL condition, and at the entrance they merge earlier under the PIL conditions than under the CIL condition.

This later divergence at the exit is probably indicative of more uncertainty concerning where the exact location of the ramp is, while the earlier merging at the entrance is probably indicative of the uncertainty concerning where the end of the acceleration ramp is.

Unusual Exiters

Drivers who left the VTMS after the gore (for the exit) or before the gore (for the entrance) either (a) crossed the gore into the main stream of traffic or (b) encroached on the left shoulder. Either condition would be indicated by a VTMS output that stopped after the last contacted switch.

An example of a gore encroachment for the exit would consist of a vehicle passing over switches 1 through 7, then not passing over switches 8 through 17. A typical shoulder encroachment would consist of a driver passing over switches 1 through 11 and missing switches 12 through 17. For the entrance, a shoulder encroachment would consist of a driver passing over switches 1 through 4, 1 through 6, or 1 through 8, and missing the rest, while a gore encroachment would consist of passing over switches 1 through 10 and missing the rest.

The table below summarizes the frequencies of unusual exiters for both the exit and entrance ramp:

	Frequency (%)	
Study Condition	Exit	Entrance
Day	3.9	1.7
CIL	4.8	1.3
PIL-4	8.5	10.4
PIL-2	6.0	9.6
No light	4.3	2.8

Table 2. Summary of results.

Measure	Result: Exit	Result: Entrance	Interpretation
Frequency of high-beam use	Increases with each decrease in lighting: CIL to PIL-4 to PIL-2 to no lighting	Increases as lighting decreases: CIL to PIL to no lighting (reversal in PIL cases)[a]	CIL better than PIL
Frequency of brakers	Increases as lighting decreases: day to CIL to PIL to no lighting (PIL cases the same)	Increases as lighting decreases: day to CIL to PIL to no lighting (reversal in PIL cases)[a]	CIL better than PIL
Frequency of unusual exiters[b]	Increases as lighting decreases: day to CIL to PIL to no lighting (reversal in PIL cases)[a]	Increases as lighting decreases: CIL to PIL to no lighting (reversal in PIL cases)[a]	CIL better than PIL
Diverge and merge patterns	Drivers diverge later under PIL than under CIL	Drivers merge earlier under PIL than under CIL	CIL better than PIL

[a]PIL-2 better than PIL-4.
[b]For gore and shoulder encroachments.

Again, the frequencies under the PIL conditions are higher than under the CIL condition.

Both of these maneuvers are indicative of erratic or dangerous behavior and probably result from either lack of control caused by inadequate visibility or uncertainty.

INTERPRETATION OF RESULTS AND STATISTICAL ANALYSIS

Table 2 summarizes the results for the four maneuvers (high beams, brakes, unusual exiters, and diverge and merge patterns) and provides an interpretation of each result.

For each of the four variables, CIL is better than PIL for both the exit and entrance ramp and, in one-half of the eight study conditions (three of four for entrance; one of four for exit), PIL-2 is better than PIL-4.

Statistical analyses (t-tests) revealed that, except for the occurrence of brakers at the exit, CIL performs significantly better than PIL. Further, although the difference between CIL and PIL was not significant for the breakers at the exit, the trend in this case clearly indicates the same pattern. In addition, there is a significant difference between the two PIL conditions in only three of the eight tests, which indicates little difference in performance between the two PIL conditions.

Additional statistical comparisons revealed the following:

1. Significantly better performance during daylight than at night (five of six comparisons),
2. Significantly better performance under CIL than under no light (seven of eight comparisons),
3. Better performance under PIL than under no light in only five of eight comparisons, and
4. Better performance under light than no light in only five of eight comparisons.

CONCLUSIONS

Based on the analyses performed on the four types of data described in the previous section, it appears that driver performance under CIL is significantly better than under either of the PIL conditions. The frequency of the erratic-behavior measures all increase under PIL in comparison with CIL and frequently decrease again under the no-light condition (see, for example, the tables under the headings of Unusual Exiters and Diverge and Merge Patterns). In addition, driver performance under PIL-2 is often superior to that under PIL-4 (see in Table 2 those entries that are referenced with a footnote).

It is hypothesized (but as yet unproven) that drivers are experiencing transitional visibility problems under the PIL conditions when they are forced to drive from dark to light to dark areas and at the same time perform a relatively complex maneuver: diverge or merge plus track a 90° curve. The problem is made more difficult under the PIL-4 condition when the lighted area is about 550 ft long (8 s at 70 ft/s) than under PIL-2 when the lighted area is only 180 ft long (2.5 s).

Photometric measurements were made at the exit ramp under CIL and PIL-2 conditions. Illumination, pavement luminance, object luminance, and glare luminance were all measured under both lighting conditions. Illuminance and all other luminances were roughly proportional to the number of lights under each lighting condition, as was visibility, which was calculated from the three luminances. No systematic relation was found between either of the photometric measurements and any of the driver-behavior measures.

This experiment will be repeated at a second interchange (a full cloverleaf type) by using the same measures and a slightly modified tapeswitch arrangement, which will cover a longer section of roadway.

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