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Motorist Information Needs and Visibility Factors

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Abridgment

Oregon's Motorist Information System

L. E. GEORGE and BRENDA HOLMAN

ABSTRACT

The Oregon billboard removal program became active in 1970. To date the Oregon State Highway Division has bought and removed approximately 2,000 billboards. The Oregon Travel Information Council, established in 1971, and the Oregon State Highway Division jointly developed informational signing for travelers. The travelers signing service systems used on Interstate and non-Interstate highways, rest areas, and scenic overlooks are described.

Oregon took action under the Federal Highway Beautification Act to remove outdoor advertising billboards on rural and certain urban sections of the state highway system in the early 1970s. The highway division is responsible for the administration of the billboard removal program. Although established earlier, the program became active in 1970. To date the highway division has brought and removed approximately 2,000 billboards. Interstate routes are almost free of commercial advertising signs except for on-premise signs and those located in areas zoned commercial or industrial. In any event, no new billboards can be installed. As time passed and a larger percentage of the billboards were removed, demand increased for some form of motorist services signing and for signing in the related sectors of the tourism business. Also there was an indication from the general public that some form of directional information was needed for those drivers seeking services, but who were not familiar with the area.

The state highway motorist information sign system was implemented in 1971 when the Oregon Legislature passed laws establishing a Travel Information Council. The 13-member council consisted of representatives from the travel industry and from the general public appointed by the governor. Under the legislation the Travel Information Council retained administrative authority over several types of information signing on the state highway system. Informational signing of interest to travelers placed in rest areas, scenic overlooks, or in gazebo-type shelters or plazas, as well as logo signs and more recently tourist-oriented directional signs, were placed under the administrative authority of the council. Outdoor advertising signs, directional signs (a special type of advertising sign), and on-premise signs erected or maintained outside the right-of-way along state highways and visible to the traveling public from a state highway were also placed under the council's jurisdiction. The council method was chosen rather than establishing a program within the Oregon Department of Transportation because it was believed that credibility and effectiveness would be improved by using an independent council to administer any type of motorist service sign system.

Under the current law, the Oregon State Highway Division provides staff engineering services to the council and establishes all signing standards. Highway division sign crews also install all logo signing not previously installed under the original contracts. The chief counsel of the Oregon Department of Transportation provides the Travel Information Council necessary legal services. As might be expected, these interlocking services result in a well-coordinated, cooperative working arrangement with the council and its administrator. Such an arrangement is absolutely necessary to provide an information system that is safe, uniform, that meets its intended objectives, and is successful.

Oregon's motorist information system consists of several elements:

1. Generous use of the general services signs (gas, food, lodging);
2. Effective use of visitors' information center signing for those centers contained in various chamber of commerce offices throughout the state;
3. An almost totally completed logo sign program on Oregon's Interstate system;
4. A healthy, popular, off-Interstate logo sign program;
5. Oregon's new off-Interstate experimental tourist-oriented directional sign program; and
6. A series of motorist services information gazebos at 22 Interstate and off-Interstate rest area locations throughout the state.

Interstate logo backboards are fabricated from aluminum extrusions and galvanized steel supports. Off-Interstate logos are a scaled-down version of the Interstate logo signs (backboard and business panel). State crews have installed all non-Interstate backboards, which are fabricated from plywood and use wood supports. The tourist-oriented directional sign is constructed of plywood with wood supports and has a word legend and directional information that consists of the business name, turn arrow, and mileage. The information gazebo design was derived from competitive architectural designs. The gazebos are constructed of wood and are usually pleasing designs that contain interior-illuminated panels. The panels are translucent and contain information about various businesses. Forty percent of the panel space must be devoted to public information--scenic attractions in the area, wild flowers, and so forth.

The majority of Interstate logo backboards were installed on the 600 plus miles of Interstate routes under three major contracts. Material stockpiled from these contracts was used by state crews over the next several years to make installations as development occurred at interchanges. Each business furnished its own logo panel under strict specifications. The units on the logo backboards are installed by state sign crews. Off-Interstate logos are handled the same way; however, both the logo panel and the backboard are installed by state sign crews. Supplemental logo signing for proper traffic operation is required as determined by an engineering investigation.

Except for general service signs, all of the components of Oregon's motorist information system are user-fee supported. Both Interstate and non-Interstate logos require a permit fee of $75 per year and a rental fee of $10 per month. The same fee is required for supplemental logo signs and the tourist-oriented directional sign. The information gazebos are privately operated under contract to the state. Space in the gazebos is rented based on a fee structure established by the private operator. General motorist services signs are installed by the highway division without charge, because they serve many businesses over large areas. Both general service signs and logo signs may be installed at the same interchange.

As with any signing program of this magnitude, some difficulties have been encountered. Early in the 1970s start-up problems with sign material requirements and spacing requirements resulted in a meeting between state and FHWA officials to review possible changes in the National Standards for Specific Information Signs contained in the FHWA program manual transmittal. To eliminate in the future problems similar to the ones that occurred during the first months of the program, the national standards were revised based on information gained from the Oregon experience. These revisions are still contained in the national standards and provide a more practical approach for motorist services sign installations.

Although Oregon ranks 30th in population (2,656,000 in 1982) and has not realized its full potential in the tourism industry, an estimated 11.8 million pleasure travelers entered the state in the last year. To provide these visitors with information related to their travel needs, 1,100 Interstate and 260 off-Interstate logo signs have been installed.

The tourist-oriented directional sign program is just beginning, so there is no measure of its impact. A 2-year study of the experimental sign program will run concurrently with sign installation. A final report will be published when the study is completed.

Publication of this paper sponsored by Committee on User Information Systems.

Abridgment

Information Sign Color Evaluation Using a Video Presentation

H. L. WOLTMAN, R. H. STANTON, and R. A. STEARNS

ABSTRACT

Signs that provide guidance or navigational information to the motorist are color coded green to facilitate rapid identification and to ensure a clear and unambiguous meaning of the nature of the information. The green color code is not always mandatory at night. The sign backgrounds may have nonreflective green backgrounds that appear black at night unless separately lighted. A laboratory method of evaluating the effectiveness of the green nighttime sign color is presented. This method isolates only the variables of interest--the effectiveness of white-on-green versus white-on-black as a means of providing attention value or target value. The method described uses a video presentation of six identical pairs of highway scenes in which only the color of the guide sign varied. Scenes were presented for a time period of 3.5 seconds, which is comparable to detection and recognition models. The sequence was shown to 313 subjects--all licensed drivers representing all age groups at a variety of locations nationwide. The analysis of results indicated that greater scene complexity and increasing driver age contributed to an increased error rate for both color combinations. There were fewer errors in the recognition and identification of the white-on-green guide signs than the white-on-black signs. For a combination of scenes, this accuracy was 3.2 times greater for the white-on-green signs and is attributable to the green night color of the signs.

Green was selected for the guide sign color following the accepted practice of applying a distinctive yet uniform color to designate the category of information presented by a traffic sign. This selection followed testing by the Bureau of Public Roads in 1957. The tests included full-scale, outdoor tests of various sign colors during both day and night and included color recognition, legibility, and various other measures of sign and color performance using a public audience of professional and lay drivers (1).

Later testing by Forbes (2), and evaluations conducted by departments of transportation (3,4), also dealt with day and night aspects of guide sign color, including subjective reactions to color determined from interviews and driving tests. Woltman (5,6) has reported typical day and night luminance levels for sign copy, sign backgrounds, and surrounds and has attempted to identify various factors that significantly affect sign luminance such as stream traffic, rainfall, and headlamp modification.

Target value, or attention value, is a characteristic that enhances detection and recognition of the sign as a source of guidance information in a frequently complex surround, a dynamic process that must work equally well for the driver, day or night. Good target value implies a conspicuous target. Cole's (7) operational definition of a conspicuous target appears appropriate: "A conspicuous object will attract attention such that the object is seen with certainty within a short observation time regardless of the location of the object with respect to the line of fixation. A conspicuous object then is one that commands attention and requires no search for it to be noticed." The green color is mandatory during the day but may be black at night if nonreflective and unlighted.

EVALUATION

The evaluation of target value for a white-on-green versus white-on-black system in various nighttime settings on a satisfactorily large audience is a task confounded by many variables:

1. Extreme variations in the sign surround. Sign surround consists of the foreground and visible background, including oncoming vehicle headlights and roadway lighting. The night-to-day variation is extreme, and may vary in significant detail from one trial to the next.
2. Sign luminance. Substantial variations in night luminance may occur due to headlamp alignment, the presence of immediately preceding or following vehicles, roadway alignment, and the proximity of luminaires. The contrast of signs to surround is established by their relative luminances. The maintenance of a consistent level of contrast throughout a field test is unlikely.
3. Traffic. Attention to the driving task is highly influenced by the proximity of adjacent vehicles, familiarity with the vehicle, the route, and other external variables.
4. Sign size. Green information signs allow the largest variation in size of any standardized traffic control sign ranging from small street name signs to large guide signs for freeways. Although all signs have the same purpose, that is, to inform about distance or location, lack of size standardization within the series complicates the interpretation of test results.
5. Sign placement. Placement of information signs is more variable than placement of signs for other series, such as regulatory or warning series. The positions include overhead bridges, span wires, right shoulders, wide offsets to the right or left side, medians, and dead ahead at intersections and ramps. Signs placed within one or two degrees of the normal visual axis are relatively easy to detect; however, many information signs are mounted outside this range. In a previous study, Hanson et al. (5) reported an angular separation of guide signs in excess of 10 degrees horizontally on roads with curvature, as in hilly and mountainous terrain. Such signs require more time for visual search.
6. Viewing time. The driver has only a limited time to find, read, and react to guide sign information, and, in the limited time available, must be able to maintain lane position, safe headway, and cope with other distractions. Large vehicles ahead may obstruct the view of critical guide signs, which can result in extremely abbreviated periods of exposure.
7. Subjects. The subjects should represent a reasonable cross section of drivers, who can also be tested simply. Subjects must be relatively unprejudiced with respect to the issues.

The sole evaluation of the green color cue at night must contend with all of the previously mentioned variables whenever a field test is used. A suitable laboratory method that can present the principal variables of interest--that is, white-on-green and white-on-black--is far more likely to derive reliable results.

A video presentation technique for evaluation of guide sign colors is a laboratory method that overcomes most of the variables listed earlier. Unlike most laboratory test methods, the video presentation equipment is portable so that a large number of subjects in diverse locations can be tested. It cannot be assumed that a video image will provide a completely faithful representation of the real visual stimuli. Stimulus of movement and movement parallax is absent, and the luminance scale is compressed. However, these shortcomings are common to any laboratory driving or sign simulation. The video presentation has the unique advantage of extremely uniform presentations to small groups or single subjects.

THE VIDEO SIMULATION

Night driving scenes were first photographed from the driver's point of view using 35 mm color slide film. (ASA 640 color transparency film exposed at 1/15 second at f 2.8 using the lower headlamp beams.) Scenes were chosen that were free of the types of signs to be tested, but were otherwise typical of street or highway scenes at night.

Of more than 100 night scenes photographed, 6 were chosen that represented downtown streets, city arterials, urban arterial highways, suburban arterial highways, and rural Interstates. Scenes included typical white, yellow, and red light sources commonly encountered in night driving.

Next, miniature guide signs were prepared by photographing 10 typical guide signs from the Manual on Uniform Traffic Control Devices (MUTCD) (8) ranging from the advance guide sign (E1-1A) to street name sign (D3). These were produced in white-on-green, as illustrated in the MUTCD, and in white-on-black. (The appearance of nonreflective unlighted green sign backgrounds at night is black.) Five sizes of each of the 10 signs were produced to provide a choice to fit the scene.

The night slide scenes were next projected through a field lens to a video camera. A selection of miniature guide signs were then arranged on a black background, photographed by a second video camera, and superimposed by mixing both signals to form a composite image. By adjustment of the camera's zoom lens, rearrangement of signs, and selection of sign size appropriate to the specific scene, a night scene was composed with signs of logical size and location. Scenes were constructed with all white-on-green or white-on-black signs.

After the composite scene with white-on-black signs was judged satisfactory, it was recorded on 1-in. tape using a computer-controlled editing system. This permitted positioning the scene on the tape in a predetermined sequence for later presentation. A duplicate scene was next constructed with white-on-green sign counterparts, identical in size, message, and position. This segment was recorded at another predetermined space on the tape. Each scene was preceded by a 6-second blank space--2 seconds for recording the answer for the previous scene, 2 seconds in which a letter appeared to identify the next scene, and 2 seconds for that scene to appear. The test scene then appeared for 3.5 seconds. The 3.5-second interval is quite consistent with the detection and recognition times listed for various sign types by Perchonok and Pollack (9). During the

3.5-second interval, the subject was instructed to count the number of signs (white-on-black or white-on-green) that appeared in the picture and write the number of signs on the answer form. Scenes were arranged so that black and green background signs were alternated, then sequenced so that each scene was randomly exposed with the paired scenes some distance apart. This precluded learning the location of signs or their number.

Twelve such scenes were prepared, identified A through L, consisting of six black and six green segments. Following the generation of the 1-in. master tape, a duplicate 0.75-in. tape cartridge was prepared for field use.

Subjects

A standard answer form was prepared that requested information on age, sex, years of driving experience, and color blindness. Spaces following the letters A through L were provided for answers.

Subjects represented a college age group, a number of highway patrol officers, a group of retired senior citizens, and drivers renewing their driver's licenses at a state licensing station. The total number of subjects tested was 313. The subjects all held valid driver's licenses and were from Phoenix and Sun City, Arizona; Atlanta, Georgia; and St. Cloud, Minnesota. Subject age, sex, and location are given in Tables 1 and 2.

TABLE 1 Observers by Location

Location	Number	Percent
St. Cloud, Minnesota	58	18.530
Atlanta, Georgia	84	26.837
Phoenix, Arizona	80	25.559
Sun City, Arizona	91	29.073
Total	313	100.000

TABLE 2 Age Distribution of Observers

Age Group	Number	Percent
0 to 25	81	25.879
26 to 35	75	23.962
36 to 45	41	13.099
46 to 55	19	6.070
56 to 65	26	8.307
66 to 75	62	19.808
76 and up	9	2.875

Actual testing required a video tape player (0.75 in.) and a 19-in. monitor. The monitor was selected for color quality, brightness contrast, and proper horizontal and vertical linearity. The same equipment was shipped to each location to ensure consistency of picture color and quality. Subjects were seated 12 ft from the monitor. Only sufficient light (approximately 1 to 2 footcandles illumination) was provided so that the answer form could be seen. From two to eight subjects were tested simultaneously. There was no discussion during the test. Subjects were uniformly instructed to provide the personal information requested and were told that a series of night scenes would be presented containing either white-on-black or white-on-green signs. They were instructed to count the number of signs in each scene. Subjects were then shown 3 sample scenes to familiarize them with the test after which they were shown the 12 test scenes.

Results

Profiles of the subjects are given in Tables 1 and 2. It is acknowledged that subjects in the 66- to 75-year age group are overrepresented in the sample. Only 2.5 percent of the subjects indicated a color blindness problem. Table 3 gives the number of miscounted black and white versus green and white signs and the distribution of both. Table 4 gives the six pairs of duplicate scenes with the number of subjects having correctly counted the number of black and white signs versus green and white. The percentage of correct counts is significantly higher for white-on-green signs for all comparisons.

TABLE 3 Distribution of Miscounted Black and White Versus Green and White Signs

No. of Signs Missed	Frequency of Black and White Scenes Miscounted: Number	Percent	Frequency of Green and White Scenes Miscounted: Number	Percent
-7	5	0.27	0	0
-6	4	0.21	1	0.05
-5	18	0.96	3	0.16
-4	41	2.18	9	0.48
-3	184	9.80	17	0.91
-2	455	24.23	80	4.26
-1	666	35.46	266	14.16
0	427	22.74	1,374	73.16
1	64	3.41	104	5.54
2	7	0.37	16	0.85
3	3	0.16	5	0.27
4	1	0.05	2	0.11
5	1	0.05	1	0.05
6	1	0.05	0	0
7	1	0.05	0	0
Total	1,878	100.00	1,878	100.00

Note: Number of signs missed by sign color. Zero missed represents number of scenes where signs were correctly counted. Negative values represent one or more signs that were missed. Positive values represent the number of signs overcounted. Total is number of black or green scenes presented times number of subjects.

TABLE 4 Number and Percent of Subjects Seeing Correct Number of Black Versus Green Signs for Each Scene and Totals for All Scenes

No. of Signs	Sign Color	No. of Subjects	Correctly Counted	Percent Correct
2	Black	313	113	36.10
2	Green	313	278	88.82
4	Black	313	30	9.58
4	Green	313	189	60.38
5	Black	313	47	15.02
5	Green	313	231	73.80
7	Black	313	16	5.11
7	Green	313	152	48.56
4	Black	313	103	32.91
4	Green	313	256	81.79
5	Black	313	118	37.70
5	Green	313	268	85.62
All Scenes	Black	1,878	427	22.74
	Green	1,878	1,374	73.16

A paired comparison T-test determined that there was a significant difference between the number of white-on-green signs missed and the number of white-on-black signs missed in the same traffic scenes. The results were significant, ($P \leq 0.0001$), indicating that there was less than 1 chance in 10,000 that the observed difference was due to chance.

The final result for all subjects and all scenes indicates that white-on-green signs were identified

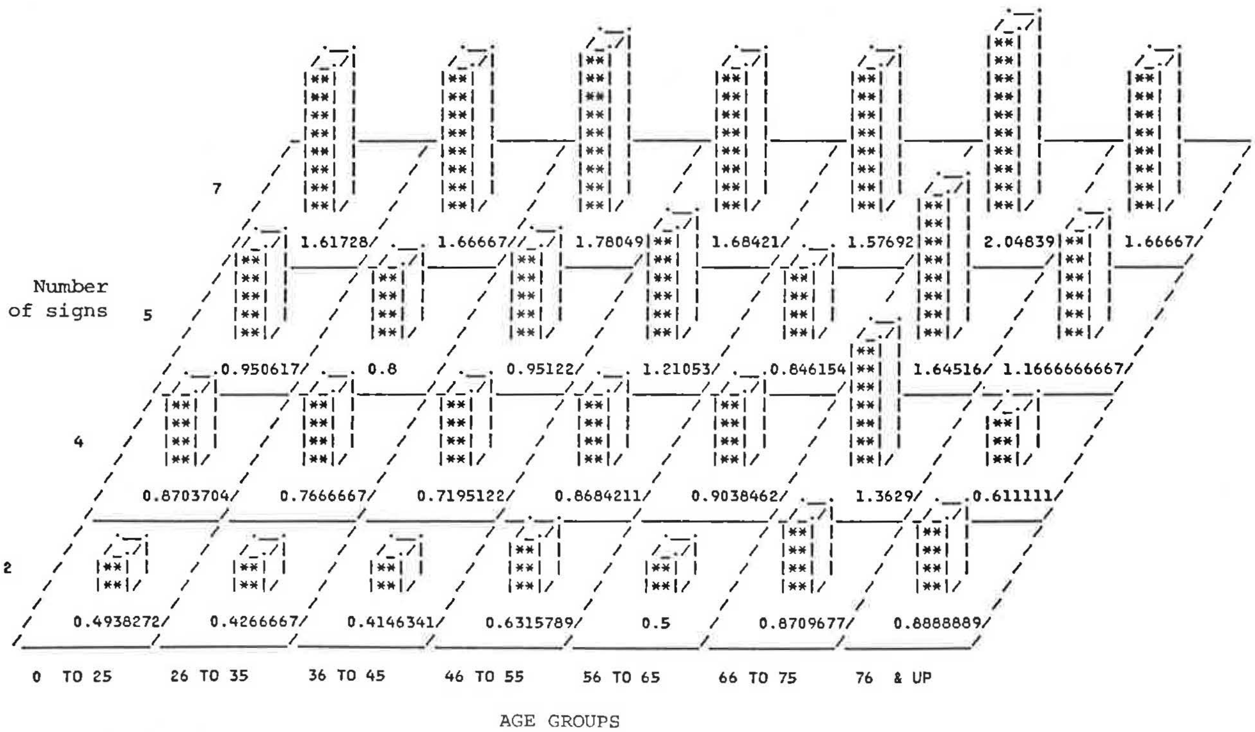

FIGURE 1 Scene complexity and increase in subject age contribute to increase in rate of error.

correctly 73.16 percent of the time versus 22.74 percent for white-on-black signs. Such signs were either undercounted, that is, were missed; or were overcounted, that is, were confused with other visual elements in the scene. Undercounting (signs missed) was far more frequent than overcounting.

Scenes with larger numbers of signs were counted incorrectly more often than scenes with fewer signs. This is apparent in the data presented in Table 4. Two scenes with similar numbers of signs have different results (scenes with four and five signs, respectively) because of different surrounds.

Older subjects had more difficulty counting the scenes correctly than did younger subjects as shown in Figure 1. The data indicate that scene complexity and increasing subject age contribute to an increased error rate.

CONCLUSIONS

A night driving simulation is described that uses a video presentation to evaluate the attention value or target value of white-on-green versus white-on-black guide signs. This technique permits a uniform presentation of the variables of interest to a large number of subjects in diverse locations. The external variables found in field testing in night settings can be eliminated or controlled by using this technique. A series of six identical pairs of highway scenes were synthesized varying only the guide sign background color. Scenes were presented for 3.5 seconds comparable to detection and recognition times reported elsewhere. The sequence was shown to 313 subjects, all having a driver's license, and representing all age groups at a variety of locations nationwide. Subjects were requested to count the white-on-green or white-on-black signs in the scene.

The analysis of results indicated greater numbers of signs, increasing scene complexity, and increasing driver age contributed to an increased error rate. There were fewer errors in the recognition and identification of the white-on-green guide signs than the white-on-black signs. The most common error was not finding, and therefore undercounting, the white-on-black signs. For all scenes combined, the accuracy for the white-on-green guide signs was 3.2 times greater and is attributable only to the green night color of the signs.

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Level of Service Evaluation of Freeway Guide Signing

ROGER W. McNEES and CARROLL J. MESSER

ABSTRACT

The methodological basis for a freeway guide signing level of service evaluation is presented. This level of service evaluation was developed using the level of service concept in the Highway Capacity Manual as the prototype. The level of service evaluation can be performed in the engineer's office on all types of signs, both overhead and ground mounted, either individually, in a series, or sequentially along a freeway. The methodology is divided into four sections: (a) navigational, (b) work load, (c) response, and (d) overall level of service.

The opportunity now exists to critically examine the urban freeway guide signing system and to improve those areas found deficient. To make optimum use of existing resources, a proficient evaluation procedure has been developed that identifies probable trouble areas without requiring an excessive amount of staff time or data collection. The various techniques used in the past (1-5) will still be used to study the effects of signing changes, but they will not be used to evaluate probable problems in freeway signing.

LEVEL OF SERVICE CRITERIA

The criteria used to evaluate the level of service of urban freeway guide signing include the navigational information needs of the motorist, the motorists work load, and the response distance provided to the motorist. The level of service concept developed in this paper was designed by using the same format as the level of service of freeway operations contained in the Highway Capacity Manual. This continuous scale signing level of service may be performed in the engineer's office on a single sign or on a series of signs along a particular freeway.

Motorists Navigational Information Needs

The navigational level of service of a particular guide sign on an urban freeway is determined from a consideration of several principal navigational related factors. These factors are (a) sufficiency, (b) consistency, (c) expectancy, and (d) relatability. These four factors all relate to separate concepts that are embodied in the navigational task. As pointed out in the following discussion of each factor, a certain amount of overlap exists among these factors, but they are separate as they relate to the navigational task. The degree to which these factors contribute to the task of navigation has not been field tested.

Sufficiency

Sufficiency is a term used to denote whether the information presented on each guide sign should be sufficient to satisfy an unfamiliar motorist's navigational information needs. The basic issues are whether the guide signing elements believed necessary are present and in accordance with accepted national guide signing principles. The Manual on Uniform Traffic Control Devices (MUTCD) is used as the chief yardstick of sufficiency. As the number of manual violations increase, the poorer the rating for sufficiency.

Consistency

Destination names are a principal navigational information source; therefore, it is imperative that consistent use of destination names be achieved. Three criteria have been identified as affecting the consistency of destination names:

1. Name familiarity consistent with route priority,
2. Number of names consistent with number of exits, and
3. Names of route destinations consistent area-wide.

As the number of violations of these three criteria increase, the poorer the rating for consistency.

Expectancy

Expectancy evaluation addresses guide signing problems that may occur within the signing sequence for a particular freeway exit. Violation of short-term expectancy is the primary consideration. The number of violations or the severity of the violations would be used by the evaluator in determining whether the system rates good, fair, or poor with regard to motorist expectancy.

Relatability

Relatability describes the general ease of determining the correct exit directions, exit destinations, and lane position from the associated cardinal directions, destinations, and lane use (assignment arrows). Inversion of the cardinal direction sequence on the overhead sign structure results in a "fair" rating. Multiple inversions result in a "poor" rating. Concurrently numbered routes splitting at a major interchange may yield extremely poor relatability scores. Signs located on horizontal curves of 1 or 3 degrees are rated "fair", whereas signs located on curves of more than 3 degrees are rated "poor."

Navigational Level of Service

To determine the level of service, a numerical score is determined for each of the four informational system factors previously described in the evaluation process. Table 1 contains these four factors along with their associated numerical score.

TABLE 1 Numerical Score for Each of the Four Navigational Factors

Factor	Good[a]	Fair[a]	Poor[a]
Sufficiency	1	*3*	10
Consistency	1	2	*5*
Expectancy	*1*	3	10
Relatability	1	*2*	5
Numerical score, T = *11*			

[a]The relative weight of each factor is based on the author's estimation of the relative consequences for violations of good signing principles.

The numerical score is then converted into a level of service grade for navigational requirements. This is accomplished by using the following level of service scale.

Navigational Level of Service

	A	B	C	D	"E"	F
0	4	6	8	10	12	Or more

Navigational Numerical Score, T

Motorists Work Load

A measure of the quality of service afforded freeway motorists by the design of the freeway guide signing system is determined by use of a concept known as the driver's work load. If the driver work load ratio is greater than one (1.0), the driver does not have sufficient time to read the signs and drive his or her vehicle. The driver can function effectively for short periods of time with work load ratios of between 1.00 and 1.15. A greater ratio would create a severe problem for the motorist. Correspondingly, work load ratios less than one (1.00) are desirable and indicate that the motorist has more time available to drive than that required to read the freeway guide signs. The work load ratio is defined as:

$$W = T_r/T_a \quad (1)$$

where

W = work load ratio,
T_r = time required to read sign (sec), and
T_a = time available to read sign (sec).

Time Available

The time motorists have available to read overhead freeway guide signs depends on many design, operational, and human factors. Some of the more important design factors include the type of sign lettering (alphabet), brightness and contrast of the lettering, familiarity of the message, sign density, competing sign messages, and location of the sign. Critical operational factors include operating speed and traffic density of surrounding vehicles. Principal human factors deal with the perception, comprehension, decision, and response of the drivers to the information provided on the sign.

Standard Conditions

Standard conditions for the design and evaluation of freeway guide signing follows. Standard conditions may be considered as describing the criteria and parameters for systems design and analysis. Basic criteria will be identified, parameters established, and the basis for each selection noted. System variables include legibility, visibility constraints, and operating speed, among others. The standard conditions affect the overall navigational time available to the driver. As the time available decreases, the level of service becomes worse, and as the navigational time available increases, the level of service becomes better.

Navigational Time Availability

Motorists driving along an urban freeway perform three basic driving tasks: control, guidance, and navigation (6). The control and guidance tasks include operating the vehicle, maintaining lane tracking, maintaining a safe speed and headway, and avoiding hazardous traffic situations. Motorists become more occupied with the control and guidance tasks as the complexity of alignment and traffic volumes increase. Motorists time-share among control, guidance, and navigational tasks as the need arises and as task demands permit. Safety considerations dictate, and driver behavior usually confirms that motorists must satisfy current control and guidance task demands before attending to navigational demands (6). As will be discussed later, motorists may require 25 to 50 percent of the total time available to perform the control and guidance tasks. Research also has indicated that at higher driving stress levels, the driver acts as a single channel processor and effectively performs only one task at a time (7).

Some research has been conducted to determine the percent of time required by drivers to maintain vehicle control while driving various horizontal alignment conditions. McDonald conducted an elaborate instrumented vehicle study (8) to determine the percent of time drivers needed (percent occupied) to drive the vehicle along tangent and curve sections of a highway. Subject motorists drove a test track at various speed levels. No other vehicles were present. On reaching the test section, subject drivers were not required to maintain the initial speed. McDonald found that drivers, traveling at 60 mph, are about 22 percent occupied when driving a tangent section and are 30 percent occupied when driving a 4.6-degree horizontal curve. It was also determined that drivers' work load and the percent of time drivers were occupied when driving a curve increased almost linearly with curvature for curves up to 15 degrees for a given speed.

Because the test data neither required speed control (to maintain a safe car-following headway) nor additional driver work load (to search for and possibly avoid vehicles in adjacent lanes), some additional increases in work load and percent of the time drivers are occupied while performing these additional urban freeway driving tasks are appropriate. Assuming that speed control and traffic surveillance are equal to the basic lane tracking task, then the net time drivers are occupied while performing control and guidance tasks on normal freeway tangent sections would be 44 percent (2 x 22 percent) and 60 percent (2 x 30 percent) occupied on a 4.6-degree horizontal curve.

Other research conducted along Ohio freeways, by Bhise and Rockwell (9), using the eye-marker camera system indicates the reasonableness of the adjusted time occupancy estimates for combining control and guidance tasks. In one case the Ohio researchers indicate that unfamiliar motorists driving in moderate to heavy freeway traffic began reading freeway guide signing as if they were occupied 45 to 50 percent of the time.

From the previous discussion, an estimate can be made of the percent of time, P, motorists have available to read urban freeway guide signing as a function of horizontal alignment conditions. The greater the horizontal curvature, the smaller the percent of time available to read signs. Using McDonald's driver work load study as a baseline, and the Ohio study to support the assumption that the total control and guidance task requirements is about twice (2.0 times) the baseline value, then the percent of time, P, available for reading guide signs (or other navigational information) would be:

$$P = 100 \text{ percent} - \text{control and guidance requirements, percent}$$

$$P = 100 \text{ percent} - 2.0\,(22 \text{ percent} + 1.74\,D), \text{ percent}$$

$$P = 56 \text{ percent} - 3.5\,D, \text{ percent} \qquad (2)$$

where D is the degree of horizontal curvature.

Available Reading Time

The amount of time (in seconds) motorists are estimated to have available to read overhead urban freeway guide signing under standard conditions is presented in the last line of Table 2. The estimated times are based on the standard conditions, namely, legibility distance for various letter series, horizontal and vertical alignment, speed of the vehicle, and the amount of time the motorist has to read the signs.

REQUIRED READING TIME

The time drivers require to read overhead freeway signs has been estimated based on considerable laboratory study data at the Texas Transportation Institute for high-quality simulated freeway guide signs under moderate display rates. This research is fully documented in a companion research report (10). Required reading times were determined for overhead freeway guide signs with various levels of total information load on the sign and by the number of sign panels used to display the information.

TABLE 2 Estimated Time Available for Reading Overhead Freeway Guide Signing Under Standard Conditions as Related to Horizontal Curvature

	Degree of Horizontal Curvature								
Analysis Step	0	1	2	3	4	5	6	7	8
Basic legibility, ft[a] 16/12-in. letters	800	800	800	800	800	800	800	800	800
Maximum legibility, ft 10 horizontal angle	–	–	–	920	750	650	570	520	480
Effective legibility, ft[b] 7.5 vertical angle	650	650	650	650	600	500	420	370	330
Speed, mph[c]	60	59	58	57	57	56	55	54	53
Maximum time, seconds 100 percent available	7.4	7.5	7.6	7.7	7.2	6.1	5.3	4.7	4.3
Percent of motorist time available, P	56	53	49	46	42	39	35	32	28
Reading time, seconds	4.1	3.9	3.7	3.5	3.0	2.4	1.9	1.5	1.2

[a]The basic legibility distance 20/24 visual acuity is therefore calculated from the legibility rate of 50 ft/in. multiplied by the letter height of the initial uppercase letter of the destination name. Most destination names on urban freeway signs are composed of 16-in. uppercase letters and 12-in. lowercase letters, which is the assumed standard. The resulting basic legibility distance is 800 ft (16 x 50).

[b]The effective legibility distance for overhead freeway guide signs under standard conditions is the basic legibility distance minus the lost legibility distance due to the maximum vertical cutoff angle of 7.5 degrees. Therefore, the effective legibility distance is 650 ft (800-150) for the 16/12 in. letter height standard.

[c]The standard operating speed for evaluating urban freeway signing systems under off-peak traffic conditions is estimated to be as follows:

$$S = 60 - 0.866\,D$$

Where S is the speed in mph and D is the degree of horizontal curvature.

The time required by the motorist to read overhead freeway guide sign information based on laboratory study data (10) is presented in Figure 1 as the family of four curves for two-, three-, four-, and five-panel overhead guide signs.

Level of Service Determination

The scale was subjectively specified based on the research available and the ramifications of W exceeding unity. It is known, for example, that motorists can perform at work load rates exceeding unity in a stressed condition for brief periods of time. However, driver errors would be expected to increase under these conditions. The example work load ratio of 1.0 would result in a work load level of service of D. A five-panel overhead freeway guide sign with a total of 20 units of information on the sign would require 4.1 seconds reading time. If it is assumed that the time available to read the sign is equal to 3.9 seconds, then the work load ratio (T_r/T_a) equals 1.05, which is a level of service of D. Any work load ratio greater than 1.0 is undesirable. Therefore, the following numerical scale was developed to determine the work load level of service.

Work Load Level of Service Grade

A	B	C	"D"	E	F	
0.0 – 0.5	0.5 – 0.8	0.8 – 1.0	1.0 – 1.2	1.2 – 1.5	1.5 – 1.7	1.7 Or more

Work Load Ratio, W

RESPONSE DISTANCE PROVIDED TO THE DRIVER

A scale, similar to the ones in the two preceding sections, is used to estimate the response level of service. The scale is based on the calculation of the ratio of the estimated travel distance needed by the driver to perform the driving tasks (which includes decision times) divided by the travel distance provided on the freeway by the placement of the sign relative to the exiting location. The response ratio, R, is defined as:

$$R = \text{Travel distance elements required/Physical distance provided} \qquad (3)$$

Driver Actions Evaluated

Advance guide signing should be placed far enough in advance of the exit point to permit a driver to perform the following actions:

1. Detect and read advance guide signs and exit direction signs,
2. Perform necessary lane changes,
3. Detect and read exit direction sign,
4. Perform exit preview, and
5. Exit.

The travel times and distances needed to perform each of these actions are presented in the following sections.

Detection and Reading of Advanced Guide Signs

In the normal routine of reading overhead freeway guide signs, motorists can see the signs a considerable distance before they can read them, and therefore there is very little detection time. Roadway design conditions do exist, however, whereby the view of an overhead (or ground-mounted) guide sign is routinely blocked or limited by an obstruction until the motorist is less than 1,000 ft from the sign structure. A 1.0- to 1.5-second detection time is considered to be satisfactory, based on existing literature sources (11,12). The following table presents the detection distance required for a 1.0-second and a 1.5-second detection time for various freeway speeds.

Speed of Vehicle (mph)	Detection Time (sec) 1.0	1.5
40	59	88
50	74	110
60	88	132

The time a motorist used while reading overhead freeway guide signs should account for the desired operating condition of providing a motorist sufficient space to maintain safe vehicle control and to avoid traffic hazards while routinely reading signs (6). The travel time (in seconds) a motorist would use while reading guide signs of a given information unit rate is estimated as:

$$T_s = T_r/(P/100) = T_r/0.56 - 0.0035\ D \quad (4)$$

where T_s is the travel time while reading signs. T_r was given in Figure 1 for various sign configurations and P was given in Table 2 for various horizontal curvatures. Resulting travel times of T_s as related to total information load on guide sign and degree of horizontal curvature may be read from the nomograph in Figure 2. As an example, an overhead guide sign structure containing a total of 15 bits of information on 4 panels located on a 2-degree horizontal curve would result in an estimated sign reading travel time of

$$T_s = 3.7/0.56 - (0.035) \times 2 = 7.5 \text{ seconds} \quad (5)$$

and can be determined from the nomograph in Figure 3.

The solution procedure follows. Trace vertically from 15 bits on the x-axis to the 4 panels curve. Next, trace horizontally to the turning line; then move vertically upward to the given degree of curvature (2 degrees). From this point, move horizontally left to the time scale on the y-axis, reading the travel time, T_s, of 7.5 seconds.

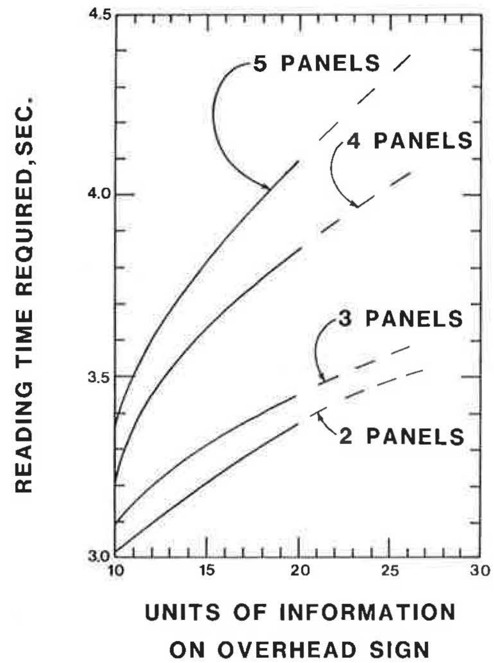

FIGURE 1 Reading time needed to acquire information as related to units of information on overhead guide sign.

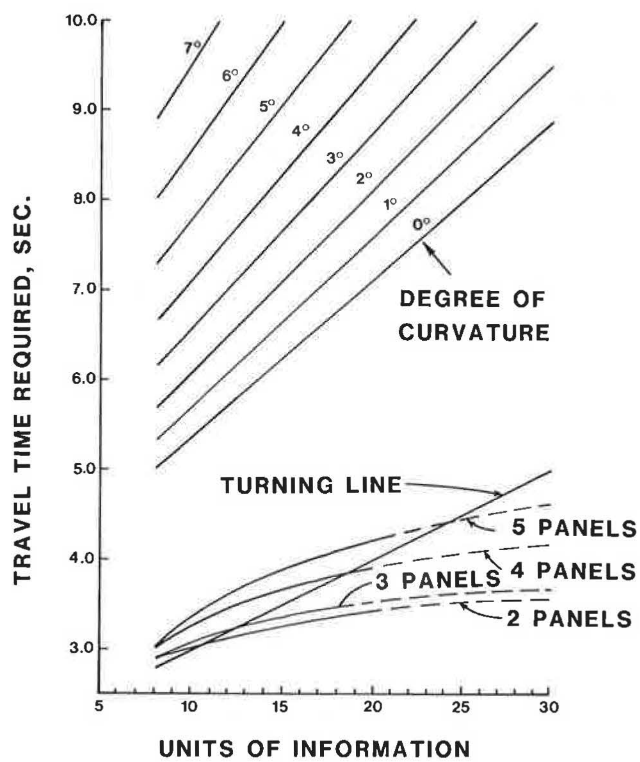

FIGURE 2 Nomograph for solving reading travel time.

The distance traveled during the sign reading travel time should be calculated next. This distance is determined from

$$D_r = 1.47 \cdot V \cdot T_s \quad (6)$$

A simplified procedure results in the satisfactory approximations for freeways not located on

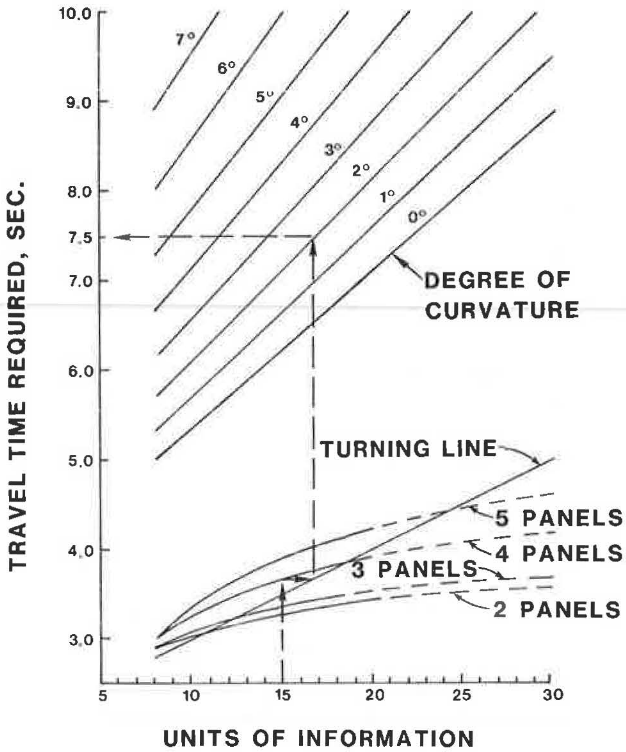

FIGURE 3 Example nomograph for solving reading travel time.

sharp horizontal curves (for example, less than 2 degrees) and typical freeway guide signs. The travel distance required for various speeds of the vehicle is given in Table 3. Under these conditions and assumptions, the average travel time ranges from about 6.5 to 7.5 seconds, with a midpoint of 7.0 seconds. The travel distances would result for a 7.0-second travel time (Table 3).

TABLE 3 Distance Traveled While Reading Signs at Different Operating Speeds

Speed (mph)	Approximate Travel Distance Dr (ft)
40	400
50	500
60	600

Lane Changing

Lane changing between freeway main lanes is frequently necessary to follow a route through an urban freeway system. One and probably more lane changes in succession may be required to follow the route as suggested by the overhead freeway guide signing. Although research has indicated that most motorists familiar with freeway guide signs probably do not follow the positioning of every overhead freeway guide sign (13), this same study demonstrated that a number of motorists (mostly motorists unfamiliar with freeway guide signs) were responding to the sign positioning over the freeway lanes.

The lane changing distance is the total distance traveled along the freeway while making lane changes of one or more lanes. McNees in 1976 used 13 male and 7 female subject drivers from the Houston area to conduct lane changing studies along the inbound freeway surveillance and control system of the six-lane Gulf Freeway (14,15). McNees' lane changing study resulted in the development of a total lane change distance. Because it is desirable to provide some margin of safety, the 85th percentile data may be used as a guide for estimating the average lane changing distance per lane change. It is recommended that a lane changing distance of 700 ft per lane change be used. Total lane change distances (in feet) for 4-, 6-, 8-, and 10-lane freeways are presented below:

Number of Freeway Lanes N	Total Lane-Change Distance (ft)
4-lane	700
6-lane	1,400
8-lane	2,100
10-lane	2,800

Detect and Read Exit Direction Sign

The detection distance required for the exit direction sign is the same as that required for the advanced guide sign. Therefore the detection distances presented in Table 3 are applicable for the exit direction sign.

One-half of the time required to read the sign, obtained from Table 3, should be used because the motorist is not time-sharing between navigation and control. Approximate travel distances for typical signing conditions with little or no horizontal curvature (for example, less than 2 degrees) give travel distances for various freeway speeds. Sign reading distances for typical exit direction signs are as follows:

Speed, V (mph)	Travel Distance, Dr (ft)
40	200
50	250
60	300

Preview Exit

On reaching the freeway exit, or an interchange split, the unfamiliar freeway motorist will require additional time and related travel distance to obtain a visual preview of the geometrics, identify the appropriate departure path, and determine a safe exit speed. This exit preview time has been assumed to be 3.0 seconds by AASHO for the design of intersections and freeway deceleration lanes (16). In an FHWA publication (17) on decision sight distance, a similar time variable for detection and recognition of potential geometric hazards is used. A minimum of 1.5 seconds was recommended in the FHWA publication for situations with moderate complexity and visual clutter, whereas 3.0 seconds was considered to be required for more complex situations or where the geometric feature is particularly difficult to detect, or where driver expectancies are violated.

An exit preview time of 1.5 seconds is recommended for use when all of the following conditions exist:

1. The exit is a nominal single lane, single exit ramp;
2. The exit is located on the right side of the freeway;
3. The adjacent through lane continues;
4. The ramp nose is readily visible to oncoming traffic; and
5. The freeway has a horizontal curvature of no greater than 2 D.

In all other situations a previous time of 3.0 seconds should be used. The distances required for various freeway speeds are summarized in the table below. An additional 1.0 seconds has been used for a typical response time (16).

Freeway Operating Speed (mph)	Exit Preview Time (sec) 1.5	3.0
40	150	240
50	180	290
60	220	350

Exit Maneuver

An exit maneuver is any traffic maneuver that departs from the main freeway route. To determine the departure location, a natural direct departure path from the freeway should be assumed. This location is about 100 ft upstream of the physical gore locations.

Developing the Response Level of Service

To be able to determine the response level of service, two assumptions have to be made: (a) the driver will require 975 ft to exit the freeway, and (b) the design of the freeway provides only 800 ft from the location where the sign becomes visible and the actual exit ramp. The response ratio would be

$$R = 975/800 = 1.22 \quad (7)$$

A response ratio of 1.22 results in a level of service E as depicted in the following level of service scale:

Response Level of Service Grade

	A		B		C		D		"E"		F	
0.0		0.5		0.8		1.0		1.2		1.5		Or more

Response Ratio, R

OVERALL LEVEL OF SERVICE

After the level of service has been determined for navigation, work load, and response for a particular sign, an overall level of service characterizing the sign must also be developed to classify a particular sign while taking into account the three levels of service previously determined. The overall level of service will be the worst (highest) level of service associated with each of the three previously described levels of service. Figure 4 shows the method used to determine the overall level of service. The overall level of service is E because both the navigational and response level of service is E.

The overall level of service E means that the particular sign or signing system is poor with regard to presenting pertinent route directions to the motorists in a timely manner, given the physical constraints under which they are operating. With a freeway signing level of service E, much could be done to improve the level of service. The consistency rating could be improved by relating the destination names, exit number, and route priority areawide. This would improve the navigational level of service. The response level of service could be improved by spreading the information over a larger distance before the exit. Both of these changes to the signing system would improve the level of service from an E to a D. To improve the overall level of service to a C or better, the work load level of service must be improved.

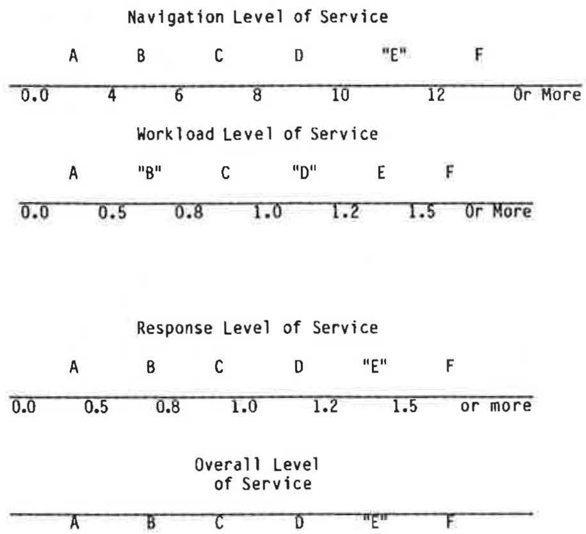

FIGURE 4 Determining the overall level of service.

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Abridgment

Evaluation of Vending Machine Operations in Rest Areas and Welcome Centers in Georgia

LAMAR CAYLOR

ABSTRACT

Section 153 of the Surface Transportation Assistance Act of 1978 authorized a demonstration program permitting the installation of vending machines in safety rest areas on the Interstate highway system. Georgia was one of the states selected by FHWA to participate in this demonstration program to evaluate the provision of vending machines in rest areas and welcome centers. Vending machines were installed in 13 rest areas and 5 welcome centers in Georgia for a 1-year evaluation period. About 92 percent of the 4,641 rest area and welcome center users interviewed indicated that providing vending machines in rest areas and welcome centers was a good idea. The provision of vending machines in the rest areas and welcome centers caused no serious security problems and only four incidents of vandalism occurred. All four of these break-ins occurred at welcome centers. The rest areas had no break-ins. Revenues from vending machines covered approximately 17 percent of the cost of operating a rest area. Revenues received during the 1-year evaluation period totaled $205,000 on gross sales of $639,000. Provision of vending machines in rest areas and welcome centers had no serious adverse effects on the operations of the rest areas and welcome centers during the evaluation period and it is recommended that they be made permanent.

Section 153 of the Surface Transportation Assistance Act of 1978 authorized a demonstration program permitting the installation of vending machines in safety rest areas on the Interstate highway system. According to the provisions of the Act the vending machines may dispense such food, drink, and other articles as the Secretary of the U.S. Department of Transportation determines necessary to ascertain the need for, and desirability of, this service to the traveling public. The Act also provided that the Secretary report to Congress by October 30, 1980, on the results of this demonstration project.

FHWA was empowered to select states to participate in this vending demonstration program. The states that were chosen to participate were required to evaluate the effects of vending machines on the operation of the rest areas. Georgia was one of the states selected to evaluate the provision of vending machines in rest areas and welcome centers.

The objective of this project was to evaluate the effects of vending machines on the operations of rest areas and welcome centers on Georgia Interstate highways by studying the effects on

- Maintenance of the rest area and welcome centers,
- Security,
- Vandalism,
- Litter on the highway downstream of the facility,
- Problems associated with increased stopping and length of stay, and
- Other problems or advantages of providing vending facilities.

Other objectives were to determine the reaction of the motoring public to the provision of vending machines in rest areas and welcome centers and to determine the economic benefits. See Figure 1 for a map of the locations of the 13 rest areas and 5 welcome centers where the evaluation was accomplished.

TYPICAL VENDING MACHINE INSTALLATIONS

The vending machines are housed in buildings that are separate from the main rest area and welcome center buildings. The buildings that house the rest area vending facilities were built by district maintenance crews and cost an average of about $4,000 to build, accounting only for material costs. The rest area vending buildings are of concrete block and plywood siding construction. The welcome center vending building is of concrete block construction. Each vending installation in a rest area or welcome center has six vending machines: two fountain soft drink machines, one coffee and hot chocolate machine, one pastries and chips machine, one candy and chewing gum machine, and one dollar bill changer. The vending machines are attractive and are supplied and serviced by the vending contractor, Servomation Corporation. The buildings and utilities are furnished by the Georgia Department of Transportation (GDOT) and the Georgia Department of Industry and Trade, which operate the welcome centers.

EFFECTS OF VENDING ON REST AREA USE

An important part of the evaluation of providing vending machines in rest areas and welcome centers is to determine the effects on rest area and welcome center use. Do vending operations cause more people to use the rest areas and welcome centers and do people stay longer because of the vending machines? These questions are addressed in the following sections.

Procedure Used to Determine Change in Percentage of Traffic Using Rest Area After Vending

To determine how rest area use would be affected by the presence of vending machines, before and after counts were made on the main line and off-ramps into the rest areas. For each of the before and after counts portable traffic recorders were set up on the highway immediately before the rest area off ramp and another portable traffic recorder was placed on the rest area off ramp. Data were recorded for 7 days for both the before and after traffic counts. The ratio of rest area off-ramp traffic to main line

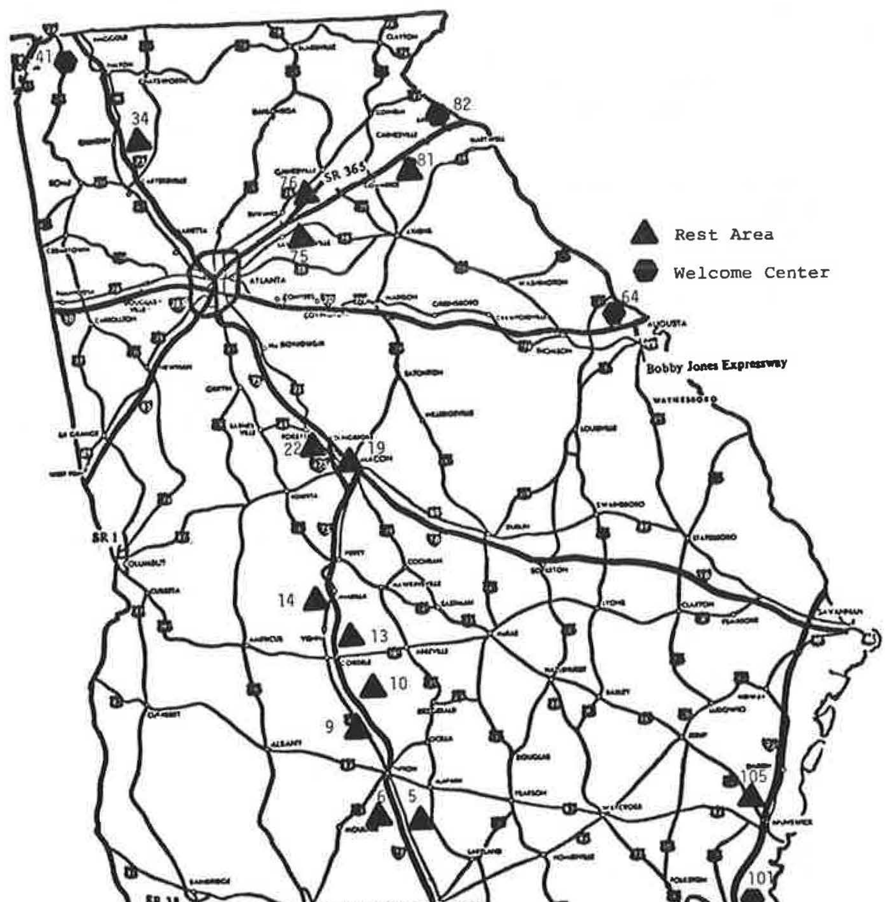

FIGURE 1 Georgia Department of Transportation rest areas and welcome center locations with vending facilities.

traffic gives the percentage of traffic using the rest area. The percentage from the before counts, which were done before the vending machines were installed, was compared with the percentage from the after counts, which were done after the vending machines were installed, to see how rest area use changed as a result of vending.

Analysis of Data from Before and After Rest Area Percent Use Counts

Table 1 gives the results from the before and after percentage of traffic use counts for the 13 rest areas in the test except for rest area 105, which was not counted. From this table note that the percent of change of main line traffic using the rest area increased from a low of 0 percent for rest area 5 to a high of 42.5 percent for rest area 10 when comparing the after percent use with the before percent use. Four locations, rest areas 13, 34, 76, and 81 suffered a reduction in percent use of 32.8, 9.7, 4.5, and 47.8 percent after the vending machines were installed. The difference between the before and after mean percent use figures at 10.05, and 10.54 percent was only 5 percent.

Although there were sizable increases in percent use after the vending machines were installed at seven test locations and sizable decreases at four others, using the t-test at the 95 percent confidence level, it could not be determined whether there was a statistically significant difference in the mean percent use figures for the before and after situations. If signs had been used to advise motorists that the rest areas have vending machines the results would possibly have been different with more people using the rest areas to get refreshments.

TABLE 1 Change in Percentage of Traffic Using Rest Areas After Vending Operations

Rest Area	Before Percent Use	After Percent Use	Change in Percent Use
5	11.8	11.8	0
6	10.0	10.8	+8.0
9	9.2	12.1	+31.5
10	8.0	11.4	+42.5
13	6.4	4.3	-32.8
14	10.0	12.8	+28.0
19	10.9	13.3	+22.0
22	8.4	11.1	+32.1
34	10.3	9.3	-9.7
75	6.6	7.8	+18.2
76	15.4	14.7	-4.5
81	13.6	7.1	-47.8
$\bar{X}$	10.05	10.54	
S^2	7.086	8.635	

Permanent Traffic Recorders

In addition to the traffic counts mentioned earlier, permanent traffic recorders were installed in the rest area off ramps and will be operated as continuous counting stations. Also, quarterly occupancy counts were conducted at each rest area. Using the occupancy rates from the quarterly occupancy counts and the traffic from the rest area continuous-count stations, the number of persons using the rest areas can be determined.

HOW LITTER IS AFFECTED BY VENDING OPERATIONS

Two areas might be affected by the accumulation of litter from vending machines in rest areas and welcome centers: the grounds surrounding the rest area and welcome center and the main line downstream of the rest area or welcome center.

Before and After Litter Surveys on Main Line

To determine the effect of vending machines in rest areas on litter downstream of the rest area, before and after litter surveys were conducted on the main line for 1 mile downstream of the rest area. In this 1-mile segment litter was collected on the side of the road where the rest area is located beginning at the end of the entrance ramp from the rest area onto the main line and for 1 mile downstream of the rest area. The litter was classified into one of 11 types of litter, and the loose volume of the litter was measured.

Similar before and after litter surveys were used. The days of accumulation of litter for the before and after surveys in the study varied between 8 and 95 days with a mean value of 62 for the days of accumulation.

Analysis of the Before and After Litter Survey Data

From the analysis of the data it could be determined that there was a statistically significant difference in the litter downstream of the rest area as a result of the vending operations. The data in Table 2 indicate that the total volume of litter increased only 3.5 percent after the vending machines were installed. As expected, the number of paper cups increased and the number of soft drink bottles and cans and beer bottles and cans decreased because the vending machines at the rest areas dispense soft drinks in paper cups whereas bottles and cans are usually dispensed at other off-road vending areas such as service stations.

TABLE 2 Before and After Litter Survey Tabulation

	Before Litter Survey		After Litter Survey				
Litter Items	Mean	Standard Deviation	Mean	Standard Deviation	Percent Change in Litter	t-Statistic	Significant at 95 Percent Confidence
Cigarette packs	145.00	140.50	151.50	91.89	4.5	0.102	No
Paper cups	64.38	43.28	117.50	68.09	82.5	1.742	No
Candy and chewing gum wrappers	87.50	59.67	76.13	67.33	-13.0	0.334	No
Potato chip and cracker wrappers	44.38	34.78	23.88	19.36	-46.2	1.363	No
Pastry wrappers	24.75	23.21	15.75	20.82	-36.4	0.764	No
Pain reliever containers	3.88	5.79	3.88	3.91	0	0	No
Soft drink bottles and cans	134.75	94.85	87.38	26.55	-35.2	1.272	No
Beer bottles and cans	173.50	134.59	110.25	53.51	-36.5	1.149	No
Food bottles and cans	13.63	9.77	14.00	19.32	2.7	0.045	No
Other food containers	47.13	47.62	18.50	15.78	-60.7	1.510	No
Other nonfood and miscellaneous	223.13	243.70	221.13	178.53	-0.9	0.018	No
Total litter volume (ft^3)	28.3	20.59	29.3	28.23	3.5	0.076	No

All 11 classes of litter showed a decrease in the number of items collected in the after litter survey except for paper cups, as mentioned earlier, and small increases in the number of cigarette packs and food bottles and cans. The litter categories for those items vended in the rest areas--paper cups, candy and chewing gum wrappers, potato chip and cracker wrappers, and pastry wrappers--all showed a decrease in number of litter items in the after survey except for paper cups. This could perhaps be explained by the fact that convenient trash receptacles are provided near the vending machines and that the food items purchased in the vending machines are consumed at the rest area rather than on the highway.

Table 2 includes data from rest areas 5, 6, 9, 10, 13, 14, 81, and 105 but does not include data from litter surveys at rest areas 19, 22, 34, 75, and 76. Data from rest area 19 were not included because of scheduling problems and data from rest area 34 were not included because data from the after survey were not available before the project was completed. The data for rest areas 22, 75, and 76 were rejected because of apparent discrepancies; these data indicated unusually large reductions in litter in the after survey as compared with the before litter survey.

A t-test was run on the data items in Table 2. Although some items showed large percent changes in litter in the after survey as compared with the before litter survey at the 95-percent confidence level, there was no statistically significant difference for any of the 11 categories of litter or the volume of litter because of the large standard deviations for all the items.

The Effects of Vending on Litter on the Grounds of the Rest Areas and Welcome Centers

To determine the effects of vending operations on the litter on the grounds of the rest areas and welcome centers, questionnaires were sent to the district engineers requesting that the maintenance area managers conduct interviews with the rest area caretakers. Questionnaires were also sent to the welcome center managers to ascertain their observations of the effects of vending on rest area and welcome center litter and other effects of the vending machines on the operation of the rest areas and welcome centers. The results of this questionnaire are discussed later.

REST AREA AND WELCOME CENTER USERS SURVEYS

To determine the reaction of the public to the provision of vending machines in rest areas and welcome centers, interviews were conducted in the rest areas and welcome centers using an appropriate questionnaire. These interviews were conducted for 8 hours at each location from 10 a.m. to 6 p.m. The interviewers were instructed to select interviewees from among all rest area users and not just those people using the vending machines.

Two interviews were conducted at rest areas 9, 10, 22, 34, 75, and 76 and at the five welcome centers; only one interview (during February 1980) was conducted at rest areas 5, 6, 13, 14, 19, 81, and 105. The first interviews were completed during the peak travel season in August 1979 and the second interviews, which were conducted at all 13 rest areas and 5 welcome centers, were completed during the off-peak travel period during February 1980. During the first set of interviews (August 1979) 2,346 people were interviewed and an additional 2,295 were interviewed during February 1980. A total of 2,403 rest area users and 2,238 welcome center users were interviewed.

Peak-Travel Season Interview Results (August 1979)

The August 1979 interviews were conducted at rest areas 9, 10, 22, 34, 75, and 76 and the five welcome centers. From this survey about 73 percent of the people interviewed were males and about 86 percent were white, 11 percent black, and 3 other races. The respondents were fairly uniformly distributed across the four age ranges that were used with a low of 20 percent for 16 to 29 year olds to a high of 29 percent for 30 to 39 years olds. Almost 40 percent of the people interviewed were either professionals or students.

Almost one-half of those stopping at the rest areas stopped to use the rest rooms followed by almost 18 percent who stopped to stretch. Almost 12 percent stopped to use the vending facilities, although there are no signs on the Interstates announcing that the rest areas have vending machines. About 89 percent of the people interviewed believed that having vending machines in rest areas was a good idea whereas only about 6 percent believed that it was not a good idea and 6 percent had no opinion.

The 68 percent interviewed who said that they had used the vending machines were asked additional questions about the vending operations. The personal characteristics of the group that used the vending machines are about the same as those for the rest area user group as a whole, except truck drivers were overrepresented in this group.

About one-half of the vending users interviewed indicated that their stay at the rest area was prolonged by less than 4 minutes and another one-third stated that their stay was prolonged by between 5 and 9 minutes because of their use of the vending machines. The average stay was prolonged by less than 6 minutes because of use of the vending machines.

Off-Peak Travel Season Interview Results (February 1980)

In February 1980 interviews were conducted at 13 rest areas and 5 welcome centers. Almost 78 percent of those interviewed were males, about 91 percent were white, about 8 percent were black, and about 1 percent were other races. The 16 to 29 age group had the smallest representation at about 16 percent whereas the 30 to 39 age group had the largest representation at more than 31 percent. The age 40 to 49 and over 50 age groups had representations of about 28 and 25 percent.

About 77 percent of those interviewed stated that they stopped to use the rest rooms whereas another 10 percent stopped to stretch, and only about 3 percent stopped to use the vending machines. About 95 percent of the respondents agreed that having vending machines in the rest areas and welcome centers was a good idea whereas only about 1 percent disagreed and about 4 percent had no opinion.

As in the August 1979 interviews, the 68 percent of those interviewed who stated that they had used the vending machines were asked additional questions about their opinion of the vending operations. The personal characteristics of this group are similar to the characteristics of the group of all persons interviewed except, as in the peak-period surveys, again, truck drivers were overrepresented in the vending machine users group.

Almost one-third of the vending machine users prolonged their stay less than 4 minutes and almost one-half prolonged their stay at the rest area from 5 to 9 minutes by using the vending machines. On the average the length of stay at the rest area or welcome center for a vending machine user was prolonged by less than 7 minutes.

Comparison of August 1979 and February 1980 Interview Results

After combining the results from the August 1979 and February 1980 interviews, almost 92 percent of all people interviewed agreed that vending machines in the rest areas and welcome centers was a good idea whereas only about 4 percent disagreed and 4 percent had no opinion. The average stay for a vending machine user was prolonged by between 6 and 7 minutes because of the use of vending machines.

The results from the August 1979 and February 1980 surveys are comparable except in the February 1980 interviews a much larger percentage of people (77 percent) listed using the rest rooms as their purpose for stopping as compared with 48 percent for the August 1979 surveys. No explanation can be found as to why, in the February 1980 interviews, only 3 percent indicated that they stopped to use the vending machines as compared with almost 12 percent in the August 1979 interviews who stopped to use the machines. It would be expected that the February 1980 percentages for stopping to use the vending machines would be larger than those for August 1979 because more people would be expected to know the machines are installed as time passes and hence more people would stop to use them. However, the warmer weather during the August interviews may have contributed to the larger numbers of people stopping to use the vending machines during that survey.

RESULTS OF SURVEY OF REST AREA CARETAKERS AND WELCOME CENTER ATTENDANTS

The rest areas are attended from 7 a.m. to 12 a.m., 7 days a week and on holidays; the welcome centers are attended from 8:30 a.m. to 5:30 p.m. A typical rest area has four caretakers to share the two 8-hour shifts. To determine the type of experiences the people most affected by having vending machines in rest areas and welcome centers have had with vending, interviews were conducted with rest area caretakers and questionnaires were sent to welcome center attendants.

The interviews with the rest area caretakers were conducted by the maintenance area managers. The questionnaires returned from the welcome center attendants were completed by the welcome center managers. Forty-seven responses were received--42 from rest area caretakers and 5 from welcome center attendants. The responses from Rest Area 105 were received too late to be included in the survey results. The results of the survey of rest area caretakers and welcome center attendants are given in Table 3.

Effects on Litter on the Grounds

The data in Table 3 indicate that 57 percent of the respondents noticed an increase in litter on the grounds of the rest areas and welcome centers after the vending machines were installed. None of the respondents indicated a decrease in litter after vending machines were installed and 43 percent noticed no change in litter as a result of the vending machines.

Vandalism and Security Problems

Only 13 percent of the respondents believed that there were vandalism or security problems at the rest areas and welcome centers because of vending machines, and about one-half of this 13 percent considered minor vandalism problems such acts as banging on the machines when they kept a user's money. There were only four serious incidents of vandalism--all occurred at welcome centers. The machines at the welcome center in Lavonia and Augusta were broken into once and the machines at the welcome center in Valdosta were broken into twice. No serious incidents of vandalism occurred at the rest areas. This could perhaps be explained by the use of heavy metal doors to close the vending buildings at 12 a.m. when the rest areas are unattended whereas only metal mesh gates are used to close the vending buildings at the welcome centers. The heavy metal doors hide the vending machines and appear less prone to vandalism than the metal mesh gates.

Change in Welcome Center and Rest Area Use

Only 47 percent of the respondents noticed a change in welcome center and rest area use after vending machines were installed and 53 percent noticed no change. The changes in rest area use most often mentioned were that more trucks and buses were using the rest areas. Also it was noticed that a number of people were stopping at the rest areas just to

TABLE 3 Rest Area Caretaker and Welcome Center Attendants Survey Results

Question	Respondents Who Indicated	Percent Responding	Comments
1. Change in litter on the grounds	Increase	57	
	Decrease	0	
	No change	43	
2. Vandalism or security problems	Yes	13	The welcome centers at Lavonia and Augusta experienced one machine break-in each; two such incidents occurred at the welcome center at Valdosta. No break-ins occurred at rest area vending machines.
	No	83	
	No response	4	
3. Have you noticed any change in rest area or welcome center use?	Yes	47	More buses and trucks use rest areas. More people now use rest areas than before vending machines were installed.
	No	53	
4. Has work load changed as a result of vending?	Yes	68	Small increase in work load due to clean-up work in vending area and issuance of refunds. More trash to empty from litter receptacles.
	No	30	
	No response	2	
5. Additional remarks about vending			The caretakers like the vending machines and believe that the public does too.

use the vending machines. A few respondents remarked that more people were using the rest areas and were staying longer because of the vending machines.

Change in Work Load Due to Vending

Even though 68 percent of the respondents found that their work loads increased because of vending, the majority believed that the increase was small. Most of the increased work load was the result of cleaning the immediate area around the vending machines. There also was more trash to empty from the litter receptacles around the vending machine buildings. Some extra work was also involved in providing refunds to customers and maintaining the petty cash fund to cover these refunds.

Additional Remarks about Vending

Of the 20 respondents who had additional remarks about vending, more than 80 percent were favorable toward having vending machines in the rest areas. Most liked having the vending machines in their rest areas or welcome centers and they believed that the public like them too.

ECONOMIC EVALUATION

The provision of vending machines in rest areas and welcome centers is a welcome service to the traveling public, which has expressed support for having vending machines in rest areas and welcome centers. The machines benefit not only the traveling public but also the Georgia Department of Transportation and Department of Industry and Trade, both of which receive commissions from the firm that contracted to operate the vending machines. The welcome centers are operated by the Department of Industry and Trade and the rest areas are operated by GDOT. It was hoped that the income from the vending operations would make a significant contribution toward paying for the operation of the rest areas and welcome centers.

The Vending Contract and Contractor

GDOT and the Department of Industry and Trade (I&T) contracted with Servomation Corporation to operate the vending machines. GDOT and I&T provided the utilities and buildings to house the machines. The contract provided for 30 percent commission on total sales, which increases on a sliding scale as sales volume increases. The contractor has serviced the machines in a satisfactory manner and, in general, is fulfilling his contractual duties adequately. To assure that the vending contractor is reporting sales receipts and commissions fairly and accurately the contract requires that the contract post a $100,000 bond and provides that the state can audit the records of the contractor.

Total Sales and Commissions

GDOT expected to receive $205,100 on total sales of $639,200 in its 13 rest areas during the first full year of operation. Data on sales for a full year were not available when the final report was written so data for a partial year were factored up to obtain the preceding numbers. Seven of the rest areas had data available for only 7 months and six had data available for 12 months, but all were factored up to obtain 12 months of receipts for all 13 rest areas. More recent follow-up data on revenues are included later in this paper.

The data in Table 4 indicate gross sales by month and the data in Table 5 indicate net revenue by

TABLE 4 Gross Vending Sales by Month

REST AREA	JUNE 80	MAY 80	APR. 80	MAR. 80	FEB. 80	JAN. 80	DEC. 79	NOV. 79	OCT. 79	SEPT. 79	AUG. 79	JULY 79
No. 19 Bibb County	8,356	4,621	5,554	5,247	2,638	2,857	2,282	-	-	-	-	-
Nos. 5 & 6 Cook County	8,486	6,732	9,979	8,045	4,573	5,031	3,324	-	-	-	-	-
Nos. 13 & 14 Dooly County	8,409	5,040	7,908	7,220	4,292	7,024	6,109	-	-	-	-	-
No. 81 Franklin County	3,491	1,983	1,869	2,112	1,238	1,817	2,487	-	-	-	-	-
No. 105 Glynn County	3,931	3,017	4,149	4,264	3,074	3,976	2,884	2,085	-	-	-	-
No. 34 Gordon County	6,963	6,471	8,808	6,688	2,809	5,573	4,174	3,669	3,586	7,077	7,725	5,361
Nos. 75 & 76 Gwinnett County	10,898	7,957	7,820	7,390	4,417	5,724	9,413	6,709	7,094	10,852	9,955	2,729
No. 22 Monroe County	6,664	5,741	8,001	8,018	5,133	7,768	7,558	7,268	5,751	7,657	8,086	4,236
Nos. 9 & 10 Turner County	10,160	7,940	10,278	8,351	5,380	8,246	7,168	4,416	5,951	8,738	9,516	2,991

TABLE 5 Net Revenues from Vending by Month

REST AREA	JUNE 80	MAY 80	APR. 80	MAR. 80	FEB. 80	JAN. 80	DEC. 79	NOV. 79	OCT. 79	SEPT. 79	AUG. 79	JULY 79
No. 19 Bibb County	2,507	1,386	1,666	1,574	791	857	685	-	-	-	-	-
Nos. 5 & 6 Cook County	2,546	2,020	2,994	2,414	1,372	1,509	997	-	-	-	-	-
Nos. 13 & 14 Dooly County	2,523	1,512	2,372	2,166	1,288	2,107	1,833	-	-	-	-	-
No. 81 Franklin County	1,047	595	561	634	371	545	746	-	-	-	-	-
No. 105 Glynn County	1,179	905	1,245	1,279	922	1,193	865	834	-	-	-	-
No. 34 Gordon County	2,089	1,941	2,642	2,006	843	1,672	1,252	1,467	1,434	2,831	3,090	2,144
Nos. 75 & 76 Gwinnett County	3,269	2,387	2,346	2,217	1,325	1,717	2,824	2,684	2,838	4,341	3,982	1,092
No. 22 Monroe County	1,999	1,722	2,400	2,405	1,540	2,330	2,267	2,907	2,300	3,063	3,234	1,694
Nos. 9 & 10 Turner County	3,048	2,382	3,083	2,505	1,614	2,474	2,150	1,766	2,380	3,495	3,806	1,196

month from the vending operations in the 13 rest areas. From Table 4 it can be seen that gross sales during the peak-travel months of the summer are from two to three times larger than the sales during the winter months as would be expected because more traffic means more potential customers and more sales.

Comparison of Vending Income and Rest Area Operating Costs

The data in Table 6 indicate the estimated annual operating cost, annual income from vending (factored), and percent of operating cost covered by the income from vending. From this table it can be observed that the cost of operating a rest area varied from $68,524 to $122,039 in 1979--the average for the 13 rest locations was $92,875. The commissions from vending operations in rest areas for the 1-year test period varied from $8,350 to $27,864 with an average of $15,775. The percent of operating expense covered by the income from vending varied from a low of 10.0 percent to a high of 22.83 percent with an average of 17.0 percent. This 17.0 percent was sufficient to pay for the material and supplies used at the rest areas.

Although the income from vending will not pay the total cost of operating a rest area, it does provide a significant sum of money that the department will not have to take from its regular budget to spend on rest areas, instead the money can be spent on other maintenance items. The vending income that is generated does not place an added tax burden on the public, but it results from the provision of convenient snack foods and drink to the traveling public without requiring capital expenditure by GDOT.

Welcome Center Vending Income

The Department of Industry and Trade received $58,792 in commissions from vending operations in the five welcome centers during the first full year of operation. The vending machines were open only from 8:30 a.m. to 5:30 p.m. whereas the vending machines at the rest areas were open from 7:00 a.m. to 12:00 a.m. This explains why the average vending income for a welcome center was $11,758 whereas the average income for a rest area ws $15,775 during the first year of operation.

RECOMMENDATIONS AND CONCLUSIONS

The provision of vending machines in 13 rest areas and 5 welcome centers in Georgia had no observed adverse effects on their operations during the 1-year evaluation period. Litter downstream of the rest area was not affected and litter on the grounds of the rest areas and welcome centers increased only slightly. The work load of the attendants increased only slightly due to vending.

Having vending facilities in the rest areas and welcome centers did increase the length of stay of those people who stopped at the rest areas and welcome centers by between 6 and 7 minutes. Also the rest area caretakers and welcome center attendants expressed the belief that more people used the rest areas as a result of vending. They also noted that more buses and trucks were using the rest areas and welcome centers after vending machines were installed.

Vandalism is not a problem even though there were four incidents of breaking into the vending machines at the welcome centers. This could possibly have

TABLE 6 Rest Area Operating Costs and Vending Income Comparison

REST AREA NO.	LOCATION	ANNUAL USAGE PERSONS	1979 ESTIMATED ANNUAL OPERATING COST	ESTIMATED ANNUAL INCOME FROM VENDING	RATIO OF VENDING INCOME TO OPERATING COST (%)
5	Cook County	450,100	68,524	12,854	18.76
6	Cook County	762,900	68,524	12,854	18.76
9	Turner County	640,800	114,011	14,951	13.11
10	Turner County	619,300	114,011	14,951	13.11
13	Dooly County	457,000	83,612	12,807	15.32
14	Dooly County	686,000	83,612	12,807	15.32
19	Bibb County	635,800	106,451	17,570	16.51
22	Monroe County	758,800	122,039	27,864	22.83
34	Gordon County	804,800	111,355	23,413	21.03
75	Gwinnett County	366,700	87,883	15,511	17.65
76	Gwinnett County	581,000	87,883	15,511	17.65
81	Franklin County	357,000	83,525	8,350	10.00
105	Glynn County	512,500	75,946	15,632	20.58
		TOTALS	$1,207,376	$205,073	
			$ 92,875 Avg.	$ 15,775 Avg.	17.00% Avg.

been prevented if the welcome center vending buildings had been set up like those at the rest areas. There were no break-ins at the rest areas.

The income from the vending operations at the 13 rest areas, which amounts to a commission of 30 percent of gross sales, was adequate to cover about 17 percent of the cost of operating a rest area on the average. Even though this income is not adequate to cover all the operating expenses of a rest area it does provide a significant amount of money that would ordinarily have come from taxes. This additional money requires no capital investment by GDOT in order to collect it but comes from the provision of a convenient snack food service to the traveling public.

From interviews with 4,641 rest area and welcome center users, it was determined that almost 92 percent of the traveling public favor having vending machines in rest areas and welcome centers. Also the majority of vending machine users were satisfied with the vending operations. In particular they agreed that the quality and selection of merchandise were good and the prices were reasonable.

It is recommended that Congress change the law to allow the permanent provision of vending machines in rest areas and welcome centers and that the law be extended to all present and future rest areas with facilities and to all welcome centers. The benefits of having vending facilities in rest areas and welcome centers outweigh the disadvantages.

It is further recommended that consideration be given to allowing signs to be erected on the highway in advance of the rest areas advising motorists that refreshments are available at the rest stop.

FOLLOW-UP EVALUATION

Four new rest areas have been constructed since this study was completed, and vending facilities were added to them. Georgia now has vending facilities in all 17 rest areas that have bathroom facilities. Vending facilities have also been added to all 8 welcome center locations. Since the evaluation was completed in 1980, no significant problems have occurred in connection with the provision of vending facilities in rest areas or welcome centers.

The state now receives 32.5 percent commission on gross sales. The revenue received for the next year after the evaluation period, fiscal year (FY) 1981, was $193,218 on gross sales of $644,059. Receipts for FY 1982 were $218,798 on gross sales of $729,327. The four new rest areas were added during FY 1982. Data for the most recent year, FY 1983, indicate commissions of $250,525 on gross sales of $824,990 for the 17 rest areas.

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Abridgment

Airborne Traffic Advisories: Their Impact and Value

JON D. FRICKER and HUEL-SHENG TSAY

ABSTRACT

Airborne traffic advisories provided by radio stations have long been considered primarily a promotional activity, but at least part of the expense of this service might be justified in terms of improved traffic flow. An assessment of airborne traffic advisories in a major U.S. city is described along with a technique for quantifying and comparing the response of drivers to a sudden incident that blocks traffic flow--with and without traffic surveillance. The study is presented in the context of an Interstate highway corridor leading into a downtown area during the morning peak period. A graphical queueing analysis technique is used to calculate the delay on the link where the incident takes place. An equilibrium traffic assignment model is adapted to determine the changes in travel times elsewhere in the corridor as traffic avoids the partially blocked link. Preliminary results indicate a surprising magnitude of benefits to justify the traffic advisory service in terms of the extra delay that can be avoided. In the case study, the queue on the Interstate highway cleared 21 percent sooner, and 33 percent of the delay in the queue was prevented when just 20 percent of the approaching drivers heard (and heeded) the traffic advisory.

Almost every U.S. city with any degree of traffic congestion has one or more radio stations that offer some form of traffic advisory service. Of particular interest to this study is the sudden incident that blocks one or more lanes of traffic (traffic accident, vehicle breakdown, traffic signal malfunction, emergency equipment in use, etc.) and that could lead to severe deterioration of network service, if untreated. Private radio stations that provide a response to this condition usually do so as a public service that also serves as a promotional tool. It would be of interest to determine just how much impact their traffic advisory service has on alleviating congestion and reducing travel times. A preliminary analysis is demonstrated of the role of airborne traffic advisories in helping traffic adjust to a sudden loss of capacity in one part of a road network.

CASE STUDY: INDIANAPOLIS

Indiana's capital city has several commuter corridors that are vulnerable to disruption by the lane-blocking incidents described earlier. One of these corridors lies along I-70 East, and is shown as running through nodes 17, 19, 21, 33, 35, and 40 in Figure 1. Between 7 and 9 a.m., westbound volumes on this Interstate highway segment approximate saturation flow. This segment leads to the infamous Spaghetti Bowl (node 40), named for the impression given an airborne observer by the many ramps at the

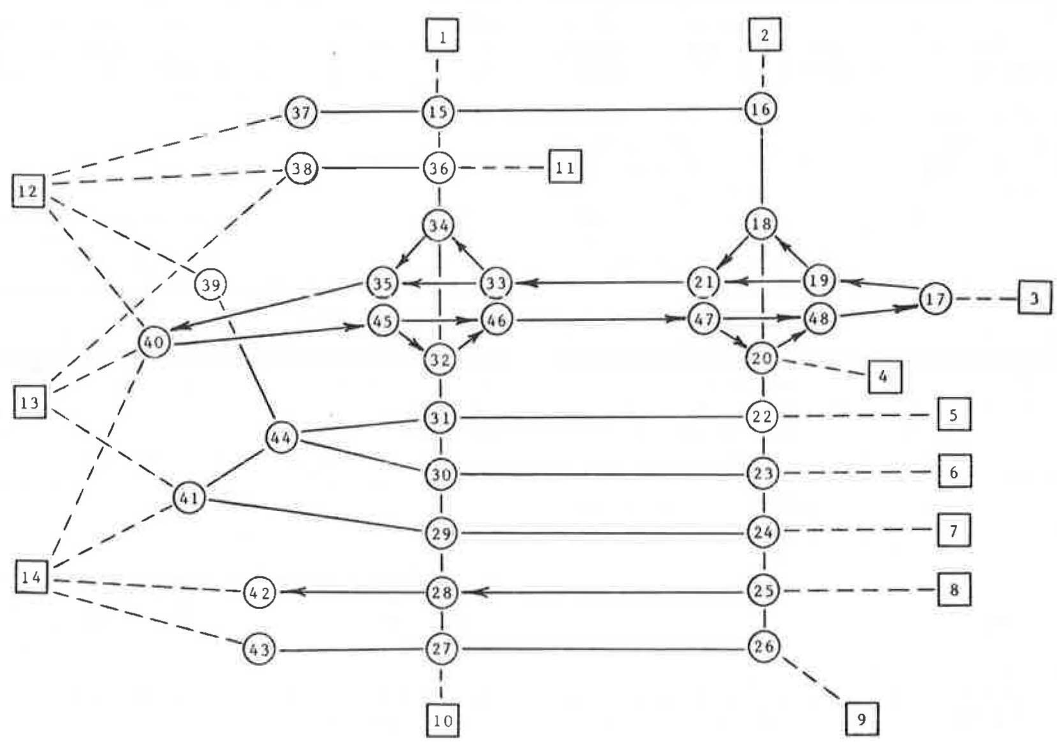


FIGURE 1 Link-node diagram of the I-70 corridor.

I-70 interchange with I-65. There are several major arterials that either parallel I-70 or cross this corridor. They connect with I-70 via ramps at Keystone Avenue-Rural Street (nodes 32 through 35) and Emerson Avenue (nodes 18 through 21). In the event that an incident occurs in, or on the approach to, the Spaghetti Bowl, these ramps and arterial routes can become vitally important. These facts, together with the availability of useful data, make this corridor a good one for a sample analysis.

STEPS IN ANALYSIS

1. Calibrate a traffic assignment (T/A) model to duplicate existing link volumes.
2. Run the T/A model to determine driver and network response to a lane blockage without a traffic advisory service.
3. Repeat step 2, but allow for some form of traffic advisory service.
4. Examine differences in driver route choices and network response for the cases tested.
5. Compare travel time saved (if any) with the cost of providing the traffic advisory service.

NORMAL CONDITIONS (Case 0)

In the link-node diagram of Figure 1, nodes 1 through 14 are actually centroids representing the origins and destinations (O-D) of trips through the corridor. It was necessary to develop an O-D trip interchange table for these centroids that, when loaded onto the corridor's links, led to link volumes similar to the counts provided by the Indianapolis Department of Transportation (DOT). Using the city DOT data on speed limits, lane counts, travel times, and volumes by time of day for each link (ℓ) in the corridor, reasonable initial values for $\underline{a}$ and $\underline{b}$ in the FHWA link congestion function (1)

$$T = T_0 \, [a + b(V/C)^4] \tag{1}$$

were derived. In Equation 1, T = travel time on link, T_o = free-flow link travel time, V = link volume, and C = link capacity. Because the focus is on driver behavior, especially with respect to route choice, the equilibrium T/A model was selected (2-4). After a few applications of the equilibrium T/A model--followed each time by adjustments of values for a_ℓ, b_ℓ, and the trip interchanges--the observed link volumes were reproduced within reasonable limits.

AN INCIDENT WITHOUT INFORMATION (Case 1)

A vehicle breakdown or accident on the approach to the Spaghetti Bowl (node 40 in Figure 1) is fairly common. Weaving maneuvers and limited sight distances not only increase the frequency of such incidents there, but also add to the problems of succeeding drivers who must decide how to respond to the suddenly blocked lanes.

For this case study, the assumption is that an incident occurs just east of node 40 (about 1.1 miles west of node 35) at 7:20 a.m. such that one westbound lane of I-70 is blocked. Given normal traffic volumes for this time of day, the question becomes: How will drivers respond to this incident if no information about it is provided to them before they seek to enter link (35,40)? The problem must be addressed in two parts. First, the geometry of link (35,40) is such that a traffic backup is not visible to drivers approaching on link (21,33) until the tail end of the backup is close to the Keystone-Rural interchange, represented by nodes 32 through 35. This means that, in the absence of outside information, approaching drivers have no warning until the queue of vehicles on (35,40) nearly reaches node 35. Second, when the approaching drivers observe this situation, what is their response and how does the street network perform?

Queueing on Link (35,40)

Under normal conditions, the capacity of each freeway lane is 1,600 vehicles per hour (vph) (5). If one of three lanes on a freeway is closed to traffic, the combined capacity of the remaining two lanes is approximately 3,000 vph (6). The westbound traffic flow passing node 35 at 7:20 a.m. is about 4,375 vph and increasing. To measure the increase of the vehicular queue where the arrival rate (traffic volume) exceeds the service rate (road capacity) and is nonuniform, the analysis period is divided into 5-minute intervals. The data in Table 1 indicate the increase in the arrival rate and the queue during a one-half-hour period beginning at 7:20 a.m., if no drivers change route choice and if the lane obstruction is not removed.

TABLE 1 I-70 Queue With One Westbound Lane Blocked

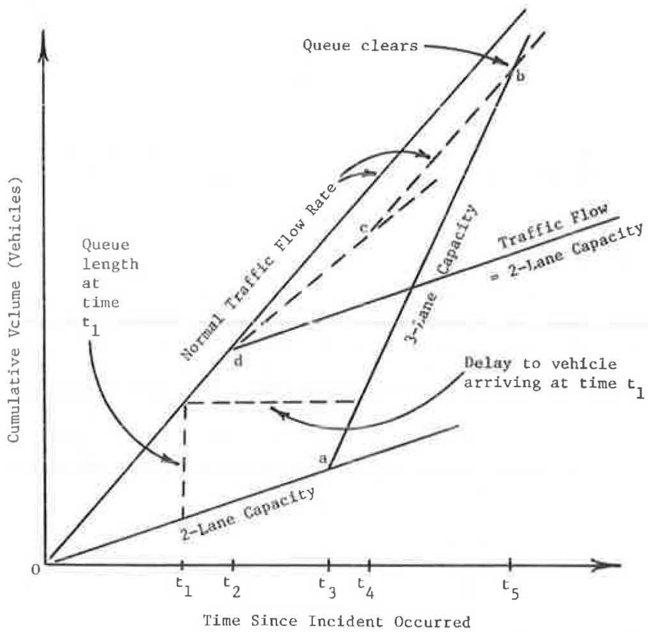
Time Period (Peak a.m.)	Three-Lane Volume (No. of Vehicles)	Two-Lane Capacity (No. of Vehicles)	End-of-Period (No. of Vehicles)	Queue Length (Miles)
7:21-7:25	368	250	118	0.34
7:26-7:30	374	250	242	0.69
7:31-7:35	375	250	367	1.04
7:36-7:40	378	250	495	1.41
7:41-7:45	382	250	627	1.78
7:46-7:50	384	250	761	2.16

The far right column in Table 1 indicates that by 7.35 a.m., the queue has nearly reached the 1.1 miles to node 35 and is visible to approaching motorists. An easy graphical technique to verify the entries in Table 1 and to calculate delay in the queue is shown in Figure 2. The broken line (d,c) indicates the reduced traffic flow into link (35, 40), as some of the traffic volume in excess of 3,000 vph diverts from I-70 at the Keystone-Rural ramps. Diversion can take the form of choosing not to enter I-70, as well as exiting from I-70. This diversion rate will be estimated in the next section. Line (c,b) in Figure 2 represents a return to normal flow rates after the queue is no longer visible. The area of the polygon 0, a, b, c, d represents the total delay in the queue.

Delays to Diverted Traffic

When approaching drivers become aware of a queue ahead, each must make a new decision as to the best route to follow. To portray driver reactions using a T/A model that gives only final equilibrium link loadings requires some care. The following steps were devised to calculate delays for Case 1.

1. Establish Case 0 equilibrium link loadings for the entire corridor.
2. At t = 0, the incident on link (35,40) occurs. The area of triangle 0, e, d in Figure 2 is the queueing delay on link (35,40) through time...



t_1 = time at which a vehicle enters the queue
t_2 = time at which the queue becomes visible to approaching traffic
t_3 = time at which the lane blockage is removed
t_4 = time at which the queue becomes invisible again
t_5 = time at which the queue clears (becomes empty).

FIGURE 2 Graphical analysis of a transient queue.

3. At t_2 = 15 minutes, the queue has backed up within sight of the Keystone-Rural interchange. Modify the travel times in Step 1 by running a separate T/A loading for Corridor West with capacity of links (33,35) and (35,40) at 3,000 vph. Corridor West is that part of the network in Figure 1 that is affected by westbound traffic avoiding link (35,40). It consists of all links on Keystone Avenue and Rural Street (between nodes 1 and 10) and those west of this line. Apply the graphical queueing analysis to link (35,40) for this time period, until...

4. At t_3 = 25 minutes, the blockage is removed. Case 0 travel times will once again apply throughout the corridor except on link (35,40). On this link use Figure 2 to monitor the clearing of the queue, until...

5. At t_4 = 35 minutes, the traffic backup becomes invisible to approaching traffic as queue-clearing proceeds; diversion from I-70 ceases.

6. The queue clears (point b in Figure 2) at t_5 = 212 minutes.

The seemingly long time until the queue clears is due to the strict definition of a queue that is inherent in using Figure 2: as long as any entering vehicle is delayed (a horizontal line can be drawn from left to right between the traffic and service lines), a queue exists. The later stages of queue clearing are actually periods of congestion delay, much like the saturated flow conditions on link (35, 40) just before the incident, rather than a prolonged series of starts and stops. Ending the study period at t = 60 minutes provides a reasonable common basis for comparing Cases 0, 1, and 2.

The sequence of the preceding steps recognizes the changing observations of drivers in the corridor during the study period. It neglects detailed calculations of travel times during transition periods between the equilibria generated by the T/A model as being insignificant. The contributions of each step to the total travel time values in Case 1 are weighted by the duration of each step. Delay in the queue is calculated as the difference between the saturated flow travel time in Case 0 and the values calculated using Figure 2. The results in Case 1 were: 355 vehicle-hours of queueing delay on link (35,40) and 13 extra vehicle-hours in the rest of the network for a total of 368 extra vehicle-hours of travel time caused by the lane blockage. Flow entering link (35,40) was reduced by 757 vph (16 percent) while the queue was visible, according to the T/A output for Corridor West. This reduction is 55 percent of the volume in excess of that link's 3,000 vph capacity during the partial blockage.

AN INCIDENT WITH TRAFFIC ADVISORIES (Case 2)

This is the same lane-blocking incident as in Case 1. The only difference is that an airborne traffic advisory service is now in operation. Often, the Indianapolis traffic observer hears about incidents from ground observers before they are seen from the air. The region is too large to allow constant air surveillance of every trouble spot. Therefore, the Case 2 analysis is begun by assuming that 5 minutes will elapse between the time of the incident and the time it is reported on the radio.

As in Case 1, when a driver becomes aware of severely reduced capacity on a link ahead in the intended path, his or her route selection process must start anew. In this case, however, listeners to traffic advisories can plan ahead to avoid link (35,40). For example, westbound drivers, who have not reached node 19 have the additional option of exiting at Emerson. Again, the responses to these options were analyzed in two parts.

Delay in the Queue on Link (35,40)

Figure 2 can be applied again to calculate total delay, with the traffic flow line undergoing modifications as traffic conditions change. For Case 2, t_2 = 5 minutes, but it is defined here as the time at which the traffic advisory is broadcast on the radio. The time at which the removal of the blockage is announced--the Case 2 definition of t_4--is assumed to be 5 minutes after t_3, or at t_4 = 30 minutes. After the diversion rate is estimated using the T/A model, t_5 and the area of the delay polygon can be calculated.

Delays Due to Diverted Traffic

In Case 1 the new route selection process was initiated on the basis of a driver's direct observation of the queue. In Case 2 this direct observation is replaced in large part by a perception conveyed by the traffic observer's broadcast. What complicates the problem is that only A percent of the drivers have heard the advisory. The other 1-A percent will operate unaware, as in Case 1. A reduction in volume between nodes 19 and 33 of something less than A percent would be expected, to allow for those drivers who choose to postpone their diversion until node 33. An upper bound for this diversion rate can be found by setting the capacity of links (19, 21) and (21, 33) to 3,000 vph, as if all drivers perceived these approach links to link (35,40) being as congested as (35,40) itself. The T/A model produces a diversion rate of 22 percent for this situation. If 75 percent of the approach links' capacity is restored to recognize that only about 25 percent of the drivers may be listening to the traffic advisories, the T/A model result is a 10 percent diversion rate at the Emerson ramps (nodes 18 through 21).

with this early diversion possible, the total drop in traffic entering link (35,40) becomes 20 percent. Among listeners, the diversion rate is estimated to be 40 percent at Emerson and 79 percent at or before Keystone-Rural.

In calculating total delay, the lower actual travel times between nodes 19 and 33 in the T/A output were restored, because capacity there would not actually be reduced. Table 2 summarizes the calculations for all three cases. The obvious reason for reduced delay in Case 2 is that the traffic advisory prevented the queue from filling too rapidly and, therefore, also had less of a queue to clear. The strict definition of a queue (Figure 2) was maintained until t = 166 minutes, but the Table 2 entries are only for the 60-minute study period. There was a 33 percent reduction (239 vehicle-hours versus 355) in queueing delay on link (35, 40). An examination of the link loadings elsewhere indicates a more subtle, but wider spread result of the traffic advisory. By allowing some drivers to divert away from an invisible queue, an increase of 7 vehicle-hours of travel time in the network exclusive of link (35,40) occurs. However, this diversion makes possible the 116 vehicle-hour saving in queueing delay on link (35,40) during the study period.

TABLE 2 Travel Time and Delay in the I-70 Corridor

	Total Travel Time (Vehicle-Hours)	Delay Due to Incident (Vehicle-Hours)	Δ Delay Case 2, Case 1 (Vehicle-Hours)
Case 0	1,125	–	–
Case 1	1,493	368	–
(In 35, 40 queue)	–	355	–
(In rest of network)	–	13	–
Case 2	1,384	259	109
(In 35, 40 queue)	–	239	116
(In rest of network)	–	20	-7

EVALUATION OF BENEFITS

Cities experience capacity-reducing incidents that will differ in frequency and severity. However, every conceivable incident need not be modeled in detail at every possible location to provide a reasonable basis for evaluating the impact of traffic advisory services. It should be sufficient to model the typical incidents that frequently plague a system during rush hour, then categorize each actual incident occurring over an evaluation period in terms of these standard incidents.

In the I-70 example, 109 hours of delay time were saved by the traffic advisory service. If an incident of this severity occurs within the observation area about four times a week, if delay time saved has the average value of $3.50 per hour per person, and if average vehicle occupancy is 1.5 persons, then $120,000 in travel time is saved each year by this service (see Figure 3). This annual value can be based on a series of analyses for typical incidents of different severity (fraction of lanes blocked, sight distance) and venue (expressway, arterial). The results can be combined by means of a weighted sum:

$$D = \sum_i f_i \cdot E[d_i] \tag{2}$$

where

D = total travel time saved,
f_i = frequency of incident type i, and
$E[d_i]$ = expected value of delay saved after incident i.

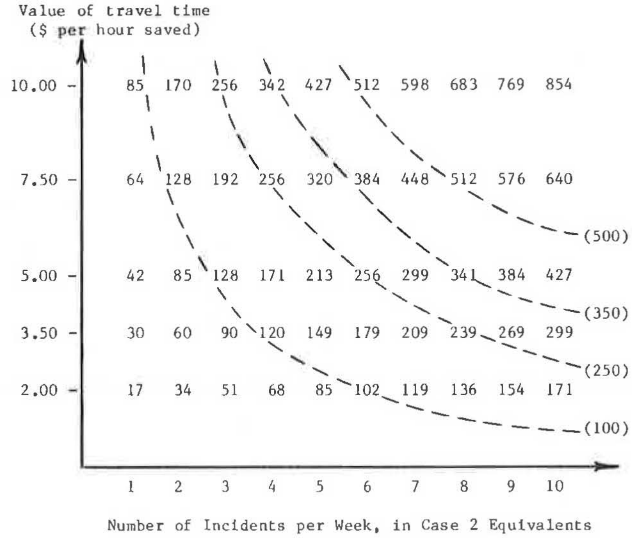


FIGURE 3 Annual benefits of travel delay saved by traffic advisory service (Case 2) for various combinations of incident frequency and value of time. An average of 1.5 occupants per vehicle. Dollar value entries are in thousands.

Using Figure 3, a current or potential provider of a traffic advisory service can compare the annual benefits and annual costs of the service. Figure 3 is drawn for the incident described in this paper. If the equivalent of only three such incidents occur in the Indianapolis surveillance area each week, if the value of travel time saved is $5.00, and if the annual cost to provide the service is $350,000, then 37 percent ($128,000) of that cost can be justified by the annual benefits of reduced delay.

CONCLUSIONS

A framework is introduced for analyzing the role of airborne traffic advisories in alleviating congestion caused by incidents that cause sudden reductions in roadway capacity. Although refinements in the procedures presented are possible, the techniques used produce believable results without excessive efforts.

The results of Case 2--an incident with informed drivers--revealed a large improvement in traffic flow over the uninformed drives in Case 1. Graphical queueing analysis estimated a 33 percent reduction in delay on link (35,40) when only 10 percent of the traffic headed for the trouble spot was able to leave I-70 just one exit ramp earlier, having heard the traffic advisory. Another 10 percent will exit at Keystone-Rural, responding to the radio report.

A means of combining a variety of incidents for evaluating economic benefits was suggested. The isoquant display of Figure 3 allows the analyst flexibility in dealing with such hard-to-specify values. The sample calculations demonstrated sizable economic benefits (saved travel time) that the advisory service provider could cite to justify its costs. Such findings could be used by a radio station to quantitatively support the public service element of the advisory's mission. This part of the station's programming may therefore become more attractive to both listeners and advertisers. The benefits accruing to the traveling public are, of course, otherwise unrecoverable. In the absence of airborne advisories provided by radio stations, local governments could use the methods described in this paper to evaluate the idea of providing or subsidizing such a service. It should be pointed out that surface-based advisories are nearly as effective as airborne services, are much less expensive, and can be analyzed with the methods described in this paper.

ACKNOWLEDGMENTS

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Road Surface Reflectance Measurements in Ontario

W. JUNG, A. KAZAKOV, and A. I. TITISHOV

ABSTRACT

A photometer for measuring surface reflectance matrices of dry pavements was developed at the University of Toronto and has been used to measure the light reflectance properties of many pavements in Ontario. The laboratory measurements were carried out on 6-in. diameter samples, and statistical deviations were carefully studied to determine the feasibility of classifying the pavements in accordance with Committee International de l'Eclairage (CIE) and (more recent) Illumination Engineering Society (IES) practice and to establish a reliable sampling procedure. The measured pavement types were classified on the basis of their reflectance parameters (Q0, S1, S2) established as average values from reflectance matrices of at least three samples. These parameters were found to be dependent on aggregate polishing and stone brightness, and on accumulated traffic load. The influence on the light reflectance of the viewing angle α being different from the standard 1 degree was also studied, and it was found that all parameters tend to decrease with increasing angle α. The findings indicate that reflection properties can be measured with fair accuracy and confidence, but that significant fluctuations of the reflectance properties can occur on a given pavement. The IES or CIE proposal for four specularity classifications under dry conditions can be recommended; however, the brightness parameter, Q0, was found to be of greater significance in lighting than originally anticipated and more accurate values should be established as discussed in this paper.

The Illumination Engineering Society (IES) has recommended the luminance method of lighting design for expressways and freeways (1), and there are some efforts to introduce even more advanced design methods based on luminance contrast or visibility index. All of these computer-based design methods require information on light reflectance properties of pavement surfaces for computational input of data.

Sponsored by the governments of Ontario and Canada, the University of Toronto has built a photometer for the measurement of road surface reflectance matrices based on the concepts originally developed by the Committee International de l'Eclairage (CIE) (2). The Ontario system features an automated control of positioning, reading and recording, and a conveniently small sample size [6 to 8 in., (150 to 200 mm)] obtained from normal pavement cores, although at least three samples are needed to classify a pavement type (3).

The pavement reflectance photometer has been used to measure the reflectance coefficients of more than 400 samples from different experimental pavements in Ontario. The results have been processed to determine pavement types by composition and age and to classify them with regard to brightness and specularity classes. The parameters of light reflectance were studied with regard to their statistical variations from:

- The measurement procedures,
- Close-range or local changes of surface features, and
- Long-range fluctuations along or across lanes.

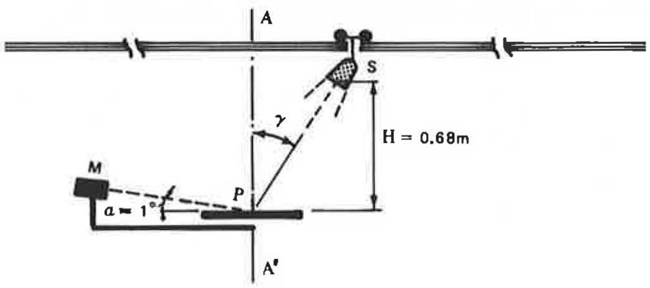

FIGURE 1 Principle of experimental setup for measurement of the reflection properties of road surfaces.

The influence of changes in viewing angle (usually set to 1 degree) on the reflectance matrix was also investigated.

REFLECTANCE MATRIX

The basic principle of road surface reflectance measurement in a laboratory is shown in Figure 1. The sample is placed horizontally on a rotating table, centered at P, and is illuminated from various positions determined by the angle γ. A photometer, M, measures the reflected light or luminance from a constant angle $\alpha = 1$ degree. The table with the rigidly fixed photometer and the sample rotate around the axis A-A', so that the projections of the light beam axis S-P and the viewing axis M-P form successive increments of a rotating angle β varying from zero to 180 degrees. The luminous intensity, I, of the lamp, S, pointing toward P is kept constant by tight voltage control. The lamp moves along a rail with constant height, H, above the sample (P).

The corresponding road geometry is shown in Figure 2. In the CIE system, the influence of the angle δ is neglected ($\delta = 0$), and the angle α is fixed to 1 degree. The laboratory measurements are automatically processed, and a matrix of reduced luminance coefficients, $R(\beta, \tan \gamma)$, is calcu-

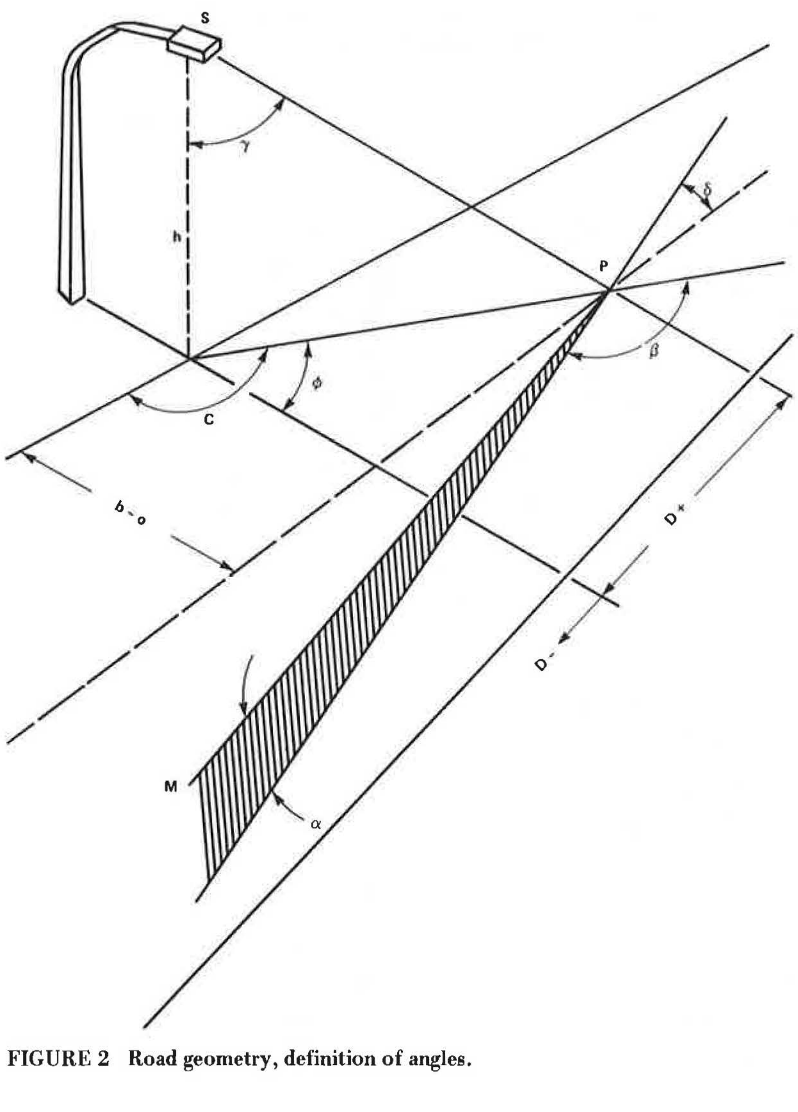

FIGURE 2 Road geometry, definition of angles.

lated and printed. Each coefficient, R, is calculated by the following relationship:

$$R = L\,H^2/I \qquad (1)$$

where

- L = luminance measured at P, in cd/m^2,
- H = height of lamp above the sample surface, in meters (0.68 m), and
- I = luminous intensity of the lamp, in lumens.

Note that the coefficient R, is reduced by a factor $\cos^3\gamma$ applied to the normal luminance coefficient, q, ranging from zero to $1/\pi$ for a perfect white diffusor. Figure 3 shows an example of a matrix printout with R-values multiplied by 100,000. The CIE reflectance parameters, Q0, S1, and S2 for this particular matrix are also given in the figure. The modified values are calculated from improved values of R(0,0) and R(0,2) based on averages or nonlinear regression analysis. The parameters are defined as follows:

- S1 = R(0,2)/R(0,0),
- S2 = Q0/R(0,0), and
- Q0 = average luminance coefficient as defined by the IES Roadway Lighting Committee (2).

All normal measurements of the matrix values, R, were carried out on samples of 150 mm (6 in.) diameter, on a centrally located field-of-view of 65 mm by 115 mm.

CLASSIFICATION CRITERIA

The aforementioned quantities (Q0, S1, and S2) are generally recognized as a set of parameters that essentially describe the reflection characteristics of a pavement for roadway lighting design. Here, Q0 is a measure for the overall brightness of the pavement as it appears to the viewer, whereas S1 and S2 describe the degree of specularity. Traditionally, these parameters are used to classify pavements for the purpose of lighting design. Specularity classes are defined by selected standard values and boundaries of S1 or S2. Systems of four or eight standard reflectance tables (i.e., matrices as shown in Figure 3) have been proposed for dry pavements (1-6). Table 1 contains the parameter values for the R-series (4) and N-series (5) of standard reflectance tables. The R-series of standard tables has been approved by IES (1).

Although Table 1 contains standard values for Q0, the overall brightness can change independently from the degree of specularity. If Q0 differs from the standard value, all reflectance coefficients, R, in

DATE: MARCH 15/83
IDENTIFICATION: 14-1

Open Grade Mix, 67% coarse aggregate, 5.8% asphalt, Hwy. 401 test section

tan GAMMA BETA ——>

tan GAMMA	0	2	5	10	15	20	25	30	35	40	45	60	75	90	105	120	135	150	165	180
0	2713	2760	2760	2742	2713	2760	2713	2754	2807	2760	2760	2848	2654	2789	2836	2807	2754	2760	2742	2742
.25	3319	3325	3325	3307	3325	3272	3230	3230	3183	3125	3072	2901	2795	2648	2471	2383	2318	2224	2177	2242
.5	3789	3731	3778	3731	3601	3495	3325	3183	3089	2895	2795	2430	2148	1954	1820	1759	1711	1679	1650	1620
.75	4078	4160	4078	3842	3654	3401	3025	2801	2477	2277	2163	1783	1556	1398	1298	1217	1208	1208	1209	1203
1	4219	4248	4160	3748	3325	2836	2430	2130	1834	1647	1481	1195	1005	944	861	845	837	859	650	856
1.25	4295	4213	4078	3554	2807	2242	1787	1506	1302	1137	996	795	719	677	653	625	653	621	645	645
1.5	4342	4219	3936	3089	2283	1675	1289	1039	885	785	713	577	525	484	475	478	456	494	484	503
1.75	4172	4060	3554	2560	1754	1256	913	757	653	560	522	422	393	367	360	374	376	380	386	398
2	4031	3842	3213	2130	1292	878	673	540	461	419	375	324	310	291	287	301	301	304	319	324
2.5	3654	3460	2471	1278	732	492	384	315	263	254	235	221	208	210	210	212	202	224	216	230
3	3325	2989	1830	790	437	277	243	219	194	183	189	165	148	158	149	163	164	174	183	189
3.5	3042	2424	1275	489	266	214	170	151	154	151	135	120	112	119	119	112	131	132	141	139
4	2789	2101	927	329	203	148	124	144	129	118	106	99.1	86.8	90.3	94.9	98.3	99.1	119	127	121
4.5	2477	1705	672	249	153	112	99.1	86.5	106	100	84.6	76.6	78.8	80.2	84.6	89.6	93.4	94.1	100	106
5	2336	1463	492	194	114	90.1	84.6	75.1	76.5	84.0	76.6	68.2	63.3	72.9	70.8	78.8	79.3	89.4	91.6	95.3
5.5	2164	1162	376	154	95.0	80.8	73.2	64.4	64.4	63.0										
6	2024	950	282	117	83.5	67.7	68.5	63.0	58.6											
6.5	1904	751	236	97.4	74.6	60.6	58.8	57.8												
7	1745	629	204	84.6	65.3	60.9	55.0	54.5												
7.5	1725	550	175	73.2	64.2	56.2	47.9													
8	1557	456	141	65.3	55.5	52.5	50.2													
8.5	1517	386	126	66.1	53.9	45.4	48.3													
9	1401	351	102	61.9	48.8	45.5														
9.5	1330	309	101	56.2	48.7	47.9														
10	1231	263	85.6	59.2	47.3	44.6														
10.5	1241	253	78.0	56.3	47.8	47.8														
11	1136	201	75.8	50.1	44.6	42.3														
11.5	1090	197	73.3	52.6	43.7															
12	1076	194	65.8	49.3	42.8															

X 100000

Qo= .0737384
S/1= 1.48577
S/2= 2.7178
R(0,0)= .0271316
R(0,2)= .0403113

MODIFIED VALUES:

S/1= 1.46331
S/2= 2.66294
R(0,0)= .0276906
R(0,2)= .0405197

FIGURE 3 Typical printout of a light reflectance matrix.

TABLE 1 Parameter Values of Standard Surfaces, "R" and "N" Classification

	R Series				N Series			
Parameter	R1	R2	R3	R4	N1	N2	N3	N4
Q0	0.10	0.07	0.07	0.08	0.10	0.07	0.07	0.08
S1	0.25	0.58	1.11	1.55	0.18	0.41	0.88	1.61
S2	1.53	1.80	2.38	3.03	1.30	1.48	1.98	2.84

the standard reflectance table must be increased or decreased proportionally.

The Erbay Atlas (5) contains 240 measured or calculated matrices, carefully numbered and identified, with an even spread of plotted points of log(S1) versus log(S2), which can be used for refined classification work. The pavement types measured in Ontario have been classified in terms of R and N classes, and also with regard to the closest matrix table contained in the Erbay Atlas.

The classification of dry pavement surfaces into a system of four specularity classes, R or N, leads to root-mean-square (rms) errors of no greater than about 5 percent in luminance and 9 percent in uniformity, when design calculations are compared (6) using an accurate matrix versus the closest standard table.

STATISTICAL VARIATIONS IN MEASURING PAVEMENT REFLECTANCE

Several studies were undertaken to determine statistical variations and confidence limits (95 percent) for matrix and parameter measurements on a pavement type. The first study was on the repeatability of measurements on the same core sample to establish the accuracy of the instrument in conjunction with the procedure of placing and leveling the sample. In this instance, it was found that the standard deviation of all parameters being measured was no greater than 2 percent.

More important were the efforts to determine the influence of texture randomness and small sample size. It was found that sufficient confidence could be established for all reflectance parameters if averages are formed from three samples taken from the same pavement. The term "same pavement" has to be understood in three ways:

1. Including only very local variations of the surface on the same sample and its immediate vicinity,
2. Including not only local variations but also variations farther along the same wheelpath, and
3. Including variations over the whole surface of a test section.

If variations of the first kind (Item 1) are much smaller than variations of the second kind (Item 2), then the small sample size in conjunction with texture randomness is generally acceptable.

The first study was carried out on a worn HL-1 type pavement of relatively uniform quality. The 95 percent confidence limit for averages from three samples were found to be as follows:

	Local (%)	Whole Wheelpath (%)
Q0	3	11
S1	4.5	11.5
S2	3.5	5

For the specularity parameters, S1 and S2, the results have been plotted in Figures 4 and 5 in the form of ellipses among the plotted points from the Erbay Atlas (4). The 95 percent confidence limit for local variation (Figure 4) is consistent with the density of the Atlas points, whereas the 95 percent confidence limit for the whole wheelpath variation (Figure 5) extends beyond several plotted Atlas points. Similar studies were carried out for other types of pavements, always showing localized variations as being substantially smaller than overall variations. The results of these studies are summarized in Table 2.

Thus, averages from three core samples of 150 mm (6 in.) diameter can be regarded as equivalent to measurements of one larger sample used by earlier CIE research. However, overall variations over long stretches of pavements may be much larger than indicated by the 95 percent confidence level for the whole wheelpath quoted previously. Such surface texture variations may be caused by

1. Inconsistencies in mixture compaction;
2. Differences in pavement wear, polishing, and aggregate loss; and
3. Contamination of pavement surface.

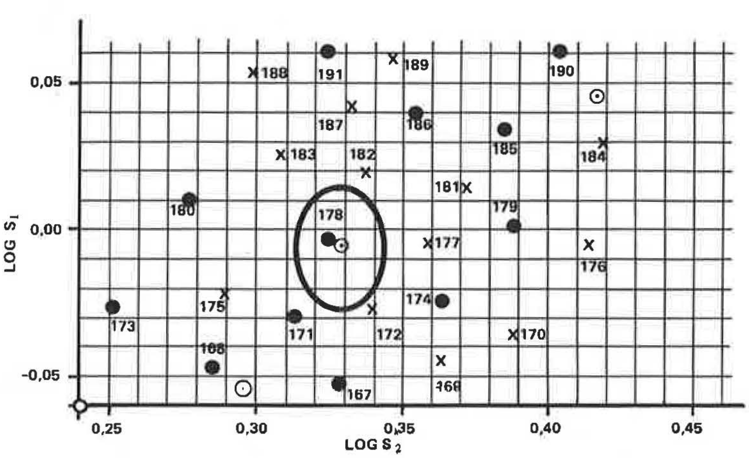

FIGURE 4 95 percent confidence limit of specularity parameters S1 and S2 for local variation.

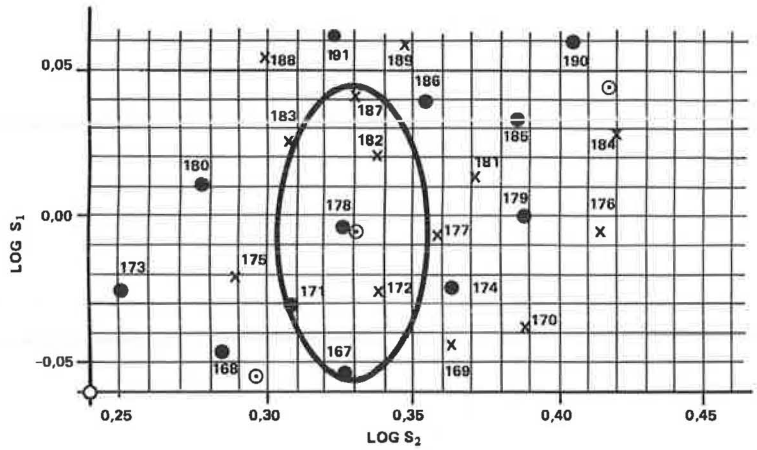

FIGURE 5 95 percent confidence limit of parameters S1 and S2 for variations along the wheelpath of a longer stretch pavement.

TABLE 2 Statistical Variations for Various Levels of Measurements

Level of Measurement	Parameter	Std. Deviation
Measurements repeated on the same sample with repositioning	Q0 S1 S2	2 2 2
Measurements on adjacent viewing areas close to same sample	Q0 S1 S2	2 9 4
Measurements along the wheelpath of the same test section	Q0 S1 S2	6% 20% 6%
Measurements over the whole road section including outside the wheelpath	Q0 S1 S2	2-15%) 4-56%)[a] 3-23%)

Note: The values in Table 2 have been established based on selected sections of Hwy 401, Toronto Bypass.

[a]Total Range Encountered.

LIGHT REFLECTANCE MEASUREMENTS OF ONTARIO PAVEMENTS

The reflectance matrix photometer used for the measurements on 36 different types of pavements in Ontario is shown in Figure 6. More than 400 core samples were processed, including those for a more rigorous statistical investigation of test sections, and the matrices of all samples were printed as shown in Figure 3 and are kept on file. The reflectance parameters of each pavement type (averages of at least three measurements) are given in Tables 3-5, indicating location, pavement types, and composition. The last three columns of these tables contain the specularity classes in terms of R and N number, and the number of the nearest Erbay Atlas matrix table (5).

Several figures have been prepared to assist interpretation of the results presented in Tables 3-5. Figures 7 and 8 represent plots of log(S1) versus log(S2), similar to plots found in Calculation and Measurement of Luminance and Illuminance in Road Lighting (2) and the Erbay Atlas (5). The figures show that for all Ontario test sections the plots are within the boundaries of the Erbay Atlas "cloud" indicated by the dashed lines. Further, Figure 7 reveals a relationship between S1 and the type of coarse aggregate, ranging from hard traprock and igneous stone to limestone. S1 values are grouped as follows:

Coarse Aggregate	Range of Log(S1)
Igneous or traprock	-0.29 to -0.17
Limestone	-0.10 to -0.06
Blend of the two	-0.23 to -0.08

This grouping can be explained by different resistance to polishing under traffic load.

Figures 9 and 10 represent plots of log(S1) versus Q0, similar to Table 9c in Theoretical Basis of Road Lighting Design (4). In general, these diagrams show a wide scattering of Q0 values, indicating large variations in brightness. More specifically,

FIGURE 6 Photograph of equipment—the reflectance matrix photometer.

TABLE 3 Reflectance Parameters of Highway 7 Test Sections (Lindsay)

LOCATION, TYPE AND MIX COMPOSITION (%)					REFLECTANCE PARAMETERS			SPECULARITY CLASSES		NEAREST ERBAY TABLE
SECT. No.	PAVEMENT	AGE (YRS)	COARSE AGGREG.	FINE AGGREG.	Q0	S1	S2	R	N	Number
1	O.G.	5	58 MR	35 MRS*	0.0744	0.6623	1.9457	2	3(2)	149
2	O.G.	5	61 LS	35 MRS*	0.1019	0.8814	2.0171	2	3	164
3	O.G.	5	61 MR&LS	35 MRS*	0.0887	0.7805	1.9875	2	3	157
4	O.G.	5	58 MR&LS	35 MRS*	0.0947	0.8289	2.0058	2	3	165
5	O.G.	5	58 LS	35 LSS	0.1125	0.8586	1.9732	2	3	164
6	O.G.	5	59 LS	35 LSS	0.0821	0.5911	1.8119	2	2(3)	136
7	D.F.C.	5	54 LS	45 S	0.1135	0.9804	2.0997	2	3	178
8	D.F.C.	5	52 LS	45 BLEND	0.1146	0.8899	2.0375	2	3	167
9	D.F.C.	5	52 LS	45 BLEND	0.1066	0.9608	2.1192	2(3)	3	178
10	D.F.C.	5	55 LS	45 MRS	0.1061	0.7990	2.0396	2	3	157
11	D.F.C	5	52 MR	45 MRS	0.0791	0.6376	1.9476	2	3(2)	143
12	D.F.C.	5	53 MR	45 LSS	0.0858	0.5425	1.8352	2	2	127
13	D.F.C.	5	52 BLEND	45 LSS	0.0969	0.6877	1.9254	2	3	149
14	D.F.C.	5	52 LS	45 LSS	0.1155	0.9660	2.0436	2	3	171
15	HL-3	5	45 LS	55 S	0.1242	1.1288	2.3490	3	3	186
16	DELUGRIP	5	60 BLEND	35 MRS	0.0858	0.5988	1.9103	2	2(3)	140
17	HL-1	5	45 TR	55 S	0.0879	0.5193	1.8068	2	2	125

LEGEND:
* -- WASHED
O.G. -- OPEN GRADE
D.F.C. -- DENSE FRICTION COARSE
MR -- MAPLE RIDGE
LS -- BEAMISH (BROWN) LIMESTONE
TR -- HAVELOCK TRAPROCK
MRS -- MAPLE RIDGE SCREENINGS
LSS -- LIMESTONE SCREENINGS
S -- ORMELL SAND

TABLE 4 Reflectance Parameters of Highway 401 Test Sections

LOCATION, TYPE AND MIX COMPOSITION (%)						REFLECTANCE PARAMETERS			SPECULARITY CLASSES		NEAREST ERBAY TABLE
SECT. No.	PAV'T	AGE (YRS)	COARSE AGGREG.	FINE AGGREG.	LANE	Q0	S1	S2	R	N	Number
1	HL-1	8	45 TR	41 NS	DRIVING	0.0875	1.0553	2.4168	3	3	197
				14 LS	CENTRE	0.0845	1.1937	2.5148	3	3	195
					PASSING	0.0919	0.7013	2.1601	3(2)	3	148
2	HL-1	8	45 TR	41 NS	DRIVING	0.0962	1.3114	2.7968	4(3)	4(3)	200
				14 TRS	CENTRE	0.0888	1.3491	2.6985	3	4(3)	202
					PASSING	0.0818	0.6646	2.0795	2	3(2)	145
3	HL-1	8	45 TR	55 TRS	DRIVING	0.0696	0.7532	2.2817	3	3	156
					CENTRE	0.0714	0.7863	2.2428	3	3	160
					PASSING	0.0708	0.5868	2.0908	2	2(3)	137
4	HL-1	8	55 TR	34 NS	DRIVING	0.0775	1.1883	2.6601	3	3	195
				11 LS	CENTRE	0.0738	0.9037	2.2916	3	3	169
					PASSING	0.0748	0.6203	2.0066	2	3(2)	145
5	HL-1	8	60 TR	28 NS	DRIVING	0.0774	0.8829	2.2766	3	3	169
				10 LS	CENTRE	0.0749	0.9179	2.3568	3	3	174
					PASSING	0.0745	0.4992	1.9236	2	2	118
6	HL-1	8	60 TR	38 TRS	DRIVING	0.0656	1.4944	2.7842	4(3)	4	207
					CENTRE	0.0632	1.5247	2.6752	3	4	212
					PASSING	0.0620	1.2059	2.4144	3	3	194
7	MODI-	8	45 LS	55 SLS	DRIVING	0.0774	0.6631	2.4803	3	3(2)	151
	FIED				CENTRE	0.0704	0.7604	2.4639	3	3	160
	HL-1				PASSING	0.0654	0.5974	2.1512	3(2)	2(3)	137
8	MODI-	8	50 SL	38 NS	DRIVING	0.0841	1.9061	3.3580	4	4	225
	FIED			12 LS	CENTRE	0.0798	1.6745	3.0357	4	4	215
	HL-1				PASSING	0.0736	1.6503	2.8793	4	4	216
9	MODI-	8	45 BF	55 BFS	DRIVING	0.0812	0.6129	2.0031	2	3(2)	140
	FIED				CENTRE	0.0737	0.5665	1.9588	2	2	134
	HL-1				PASSING	0.0788	0.5290	1.8862	2	2	125
10	MODI-	8	40 BF	45 NS	DRIVING	0.0934	1.1418	2.7321	3	3	192
	FIED			15 LS	CENTRE	0.0866	1.0565	2.5498	3	3	184
	HL-1				PASSING	0.0866	0.7446	2.1071	2	3	153
11	SAND	8	14 TR	84 TRS	DRIVING	0.0673	1.4215	2.9109	4	4	204
	MIX				CENTRE	0.0655	1.2502	2.7779	4	3	198
					PASSING	0.0614	1.1994	2.7355	3(4)	3	192
12	SAND	8	9 TR	89 TRS	DRIVING	0.0691	1.2944	2.8359	4	3(4)	200
					CENTRE	0.0659	1.1170	2.6160	3	3	190
					PASSING	0.0646	0.9624	2.3550	3	3	174
13	OPEN	8	67 TR	33 TRS	DRIVING	0.0674	1.4182	2.8718	4	4	204
	GRADE				CENTRE	0.0669	1.5510	2.9015	4	4	211
					PASSING	0.0656	1.1693	2.5874	3	3	190
14	OPEN	8	67 TR	31 TRS	DRIVING	0.0646	1.4286	2.7329	3(4)	4	207
	GRADE				CENTRE	0.0624	1.3612	2.6552	3	4(3)	202
					PASSING	0.0611	1.0972	2.3352	3	3	186
15	OPEN	8	30 TR	70 TRS	DRIVING	0.0750	1.7050	3.2041	4	4	215
	GRADE				CENTRE	0.0686	1.5377	2.8701	4	4	211
					PASSING	0.0655	1.0616	2.5457	3	3	184
16	OPEN	8	30 TR	68 TRS	DRIVING	0.0673	1.2342	2.8069	4(3)	3	198
	GRADE				CENTRE	0.0613	1.1412	2.5527	3	3	190
					PASSING	0.0611	0.9703	2.4239	3	3	179
17	MASTIC	8	70 LS	19 SLS	DRIVING	0.0629	1.4552	2.6720	3	4	207
					CENTRE	0.0647	1.3673	2.5713	3	4(3)	202
					PASSING	0.0658	1.2484	2.5214	3	3	197
1 & 19	HL-1	8	45 TR	41 NS	DRIVING	0.0817	1.8504	3.1227	4	4	224
					CENTRE	0.0752	1.6846	2.9403	4	4	216
					PASSING	0.0743	1.3179	2.6698	3	4(3)	202
18	OPEN	1/2			DRIVING	0.0610	0.6555	2.0780	2	3	145
	GRADE				CENTRE	0.0613	0.6234	2.0760	2	3(2)	145
					PASSING	0.0618	0.7702	2.0627	2	3	157

LEGEND: COARSE AGGREGATE
- TR -- TRAPROCK
- SL -- STEEL SLAG
- BF -- BLAST FURNACE SLAG

FINE AGGREGATE
- SLS -- STEEL SLAG SCREENINGS
- LS -- LIMESTONE SCREENINGS
- TRS -- TRAPROCK SCREENINGS
- BFS -- BLAST FURNACE SCREENINGS
- NS -- NATURAL SAND

TABLE 5 Reflectance Parameters of Concrete Samples from Highway 401

LOCATION, TYPE AND MIX COMPOSITION (%)					REFLECTANCE PARAMETERS			SPECULARITY CLASSES		NEAREST ERBAY TABLE
SECT. No.	PAVEMENT	AGE (YRS)	COARSE AGGREG.	FINE AGGREG.	Q0	S1	S2	R	N	Number
1, #2	even concrete polished	~15	LS	PRS	0.1235	1.294	2.676	3	3(4)	200
1, #3	even concrete gritty	~15	LS	PRS	0.1093	0.473	1.875	2	2	114
2, #2	concrete longitud'l grooving	~15	LS	PRS	0.1291	1.318	2.267	3	4(3)	201
2, #2	concrete lateral grooving	~15	LS	PRS	0.0945	0.906	2.249	3	3	169
3, #2	concrete longitud'l grooving	~15	LS	PRS	0.1244	1.333	2.461	3	4	202
3, #2	concrete lateral grooving	~15			0.0958	1.324	2.625	3	4	202
4, #2	concrete rough bridge deck	~15	LS	PRS	0.1206	1.264	2.092	2(3)	3(4)	196

LEGEND: LS -- LIMESTONE
PRS -- PIT-RUN SAND

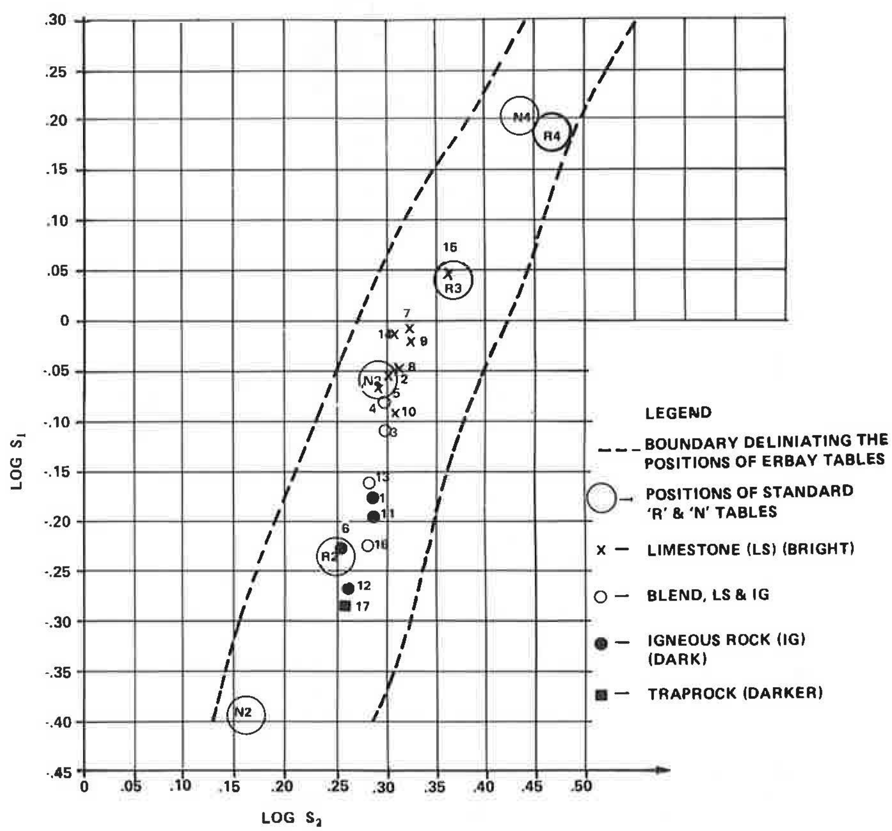

FIGURE 7 Specularity plot of Lindsay section.

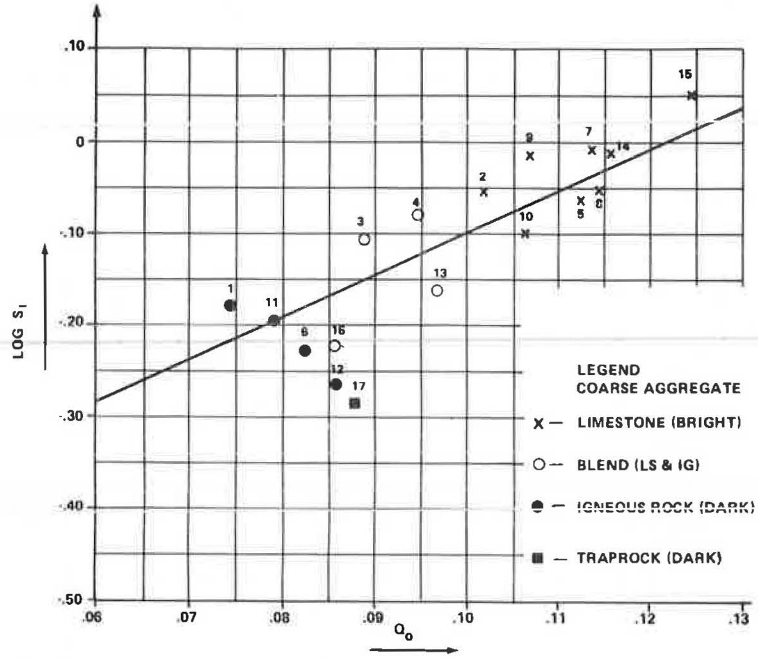

FIGURE 8 Specularity plot of Highway 40 test sections.

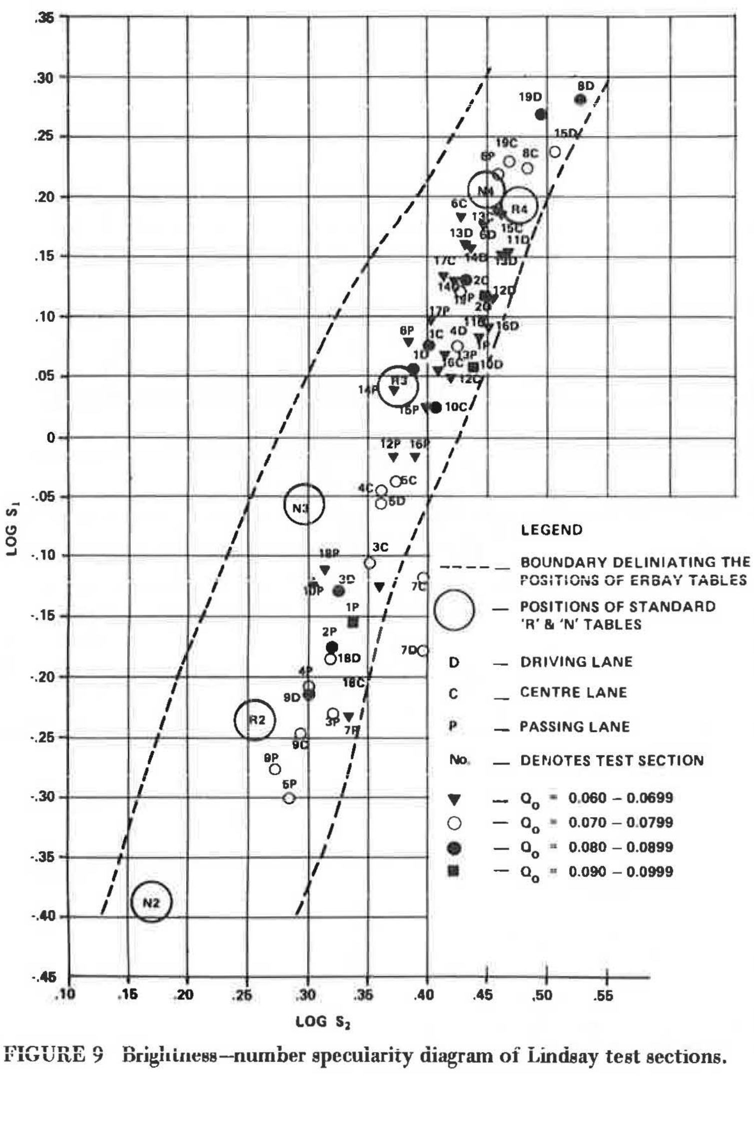

FIGURE 9 Brightness—number specularity diagram of Lindsay test sections.

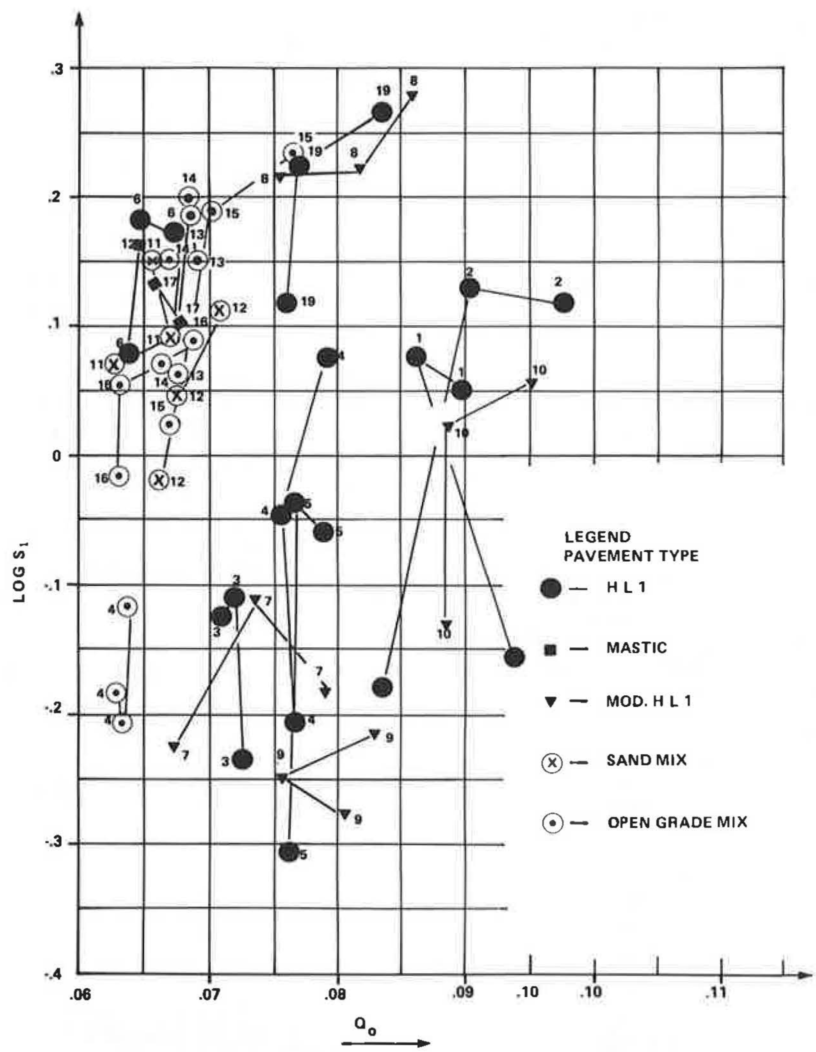

FIGURE 10 Brightness—specularity diagram for Highway 401 test sections.

Figure 9 (the Lindsay test site) reveals the following grouping in terms of coarse aggregates:

Coarse Aggregate	Range of Q0
Dark traprock	0.074 to 0.088
Blend of the two	0.086 to 0.097
Bright limestone	0.102 to 0.124

This grouping depends obviously on the brightness of aggregates and perhaps partly on a concurrent specularity increase.

Finally, with regard to Figure 10, representing the Highway 401 test site, there appears to be a narrow grouping of the open-grade mixes with traprock aggregate (and sand mixes combined). For these pavement types a narrow band of Q0 values exists between 0.061 to 0.069. Otherwise, the Highway 401 test sections show very large variations in brightness (Q0 = 0.06 to 0.10).

Some results from concrete samples that have been measured are given in Table 5. Much depending on the prevailing limestone coarse aggregate, the Q0 values are high, ranging from 0.109 to 0.129, including the longitudinally grooved textures. Note that only lateral grooving appears to reduce the Q0 value to about 0.095, without a significant change in specularity. From gritty to polished samples there is only a slight increase in Q0, but there is a major shift in specularity, from R2 to R3.

THE EFFECT OF ACCUMULATED TRAFFIC

With regard to the time of measurement, a shift can be observed from lower specularity classes to higher ones. A comparison between Table 3 and Table 4 indicates that the Lindsay/Highway 7 test sections have lower classes assigned to them, mainly R2 and N3, whereas most Highway 401 test sections exhibit higher classes, because of the substantially higher accumulated traffic since construction. Further, on Highway 401, a shift of specularity can also be observed when going from the driving lane to the outer passing lane. This shift is typically from Class 2 to 3 or from Class 3 to 4 for both R and N classifications. At the same time, there is also an increase in brightness of the asphalt pavements with time or traffic accumulation, which is reflected in a shift of Q0 values. An attempt has been made to quantify these shifts versus accumulated truck traffic load, which has been estimated at about 0.76 and 19 units of 10,000,000 tons (1000 kg) for the outer passing lane and the driving lane, respectively. The corresponding shift of parameters (i.e., the differences in Q0 and S1) are: (a) difference in Q0, 0.01 (approximately) and (b) difference in S1, 0.3 to 0.6 (range for many types).

REDUCED REFLECTANCE COEFFICIENT MATRIX

A representation of a reduced reflectance coefficient matrix is shown in Figure 1. The coefficient is a function of β and tan γ. Another representation is given in Figure 11, which shows visual images printed by a computer. The plastic plottings cover an area that corresponds to a roadway area measured in multiples of mounting height, H, as follows:

- From -4H to +12H longitudinally, and
- From zero to +3H laterally (one side only).

Note that this is one-half the space angle by which the value Q0 has been defined (2,4-6).

In particular, Figures 11a to 11d represent the matrices of the standard R tables (1). Figures 11e and 11f are typical matrices measured on the Highway 401 test sections. Their shape is comparable to one of the standard R shapes shown to the left or to a shape between them. This means that there is sufficient similarity between measured and standardized matrices for the traditional method of classification.

However, special attention must be given to the shape of the surface shown in Figure 11g, which represents an HL-3 pavement type containing limestone coarse aggregate from the Highway 7/Lindsay test section No. 15. The aggregate was observed to be highly polished. Although this test section has been nominally classified as R3 or N3, the measurements on this surface fall out of the traditional CIE classification system because there is no provision to take into account the second hump along the longitudinal axis. Such an odd case has never been reported. If this discovery turns out to be of some importance, it should probably be named the Ontario Hump; however, the particular pavement does not belong to the preferred standard designs and should be avoided in any case because of low skid resistance. The parameters listed for this pavement have been calculated in the usual manner, but probably result in underestimating its specularity. A more suitable class would probably be R4 or N4.

In order to estimate the magnitude of change in luminance design calculations when a shift in specularity class occurs (Q0 being constant), the data in Table 6 are presented. The data indicate the percent differences in maximum, minimum, and average luminance values for each Class R1, R2, R3, compared to those using the standard R4 as input. The values given in Table 6 are based on a typical example and do not represent maximum possible differences.

TABLE 6 Percent Luminance Change

Specularity Class	R4	R3	R4	R1
Average	0	+3.3	+4.6	+7.5
Maximum	0	+1.1	+13.9	+16.8
Minimum	0	+0.7	+6.5	+7.8

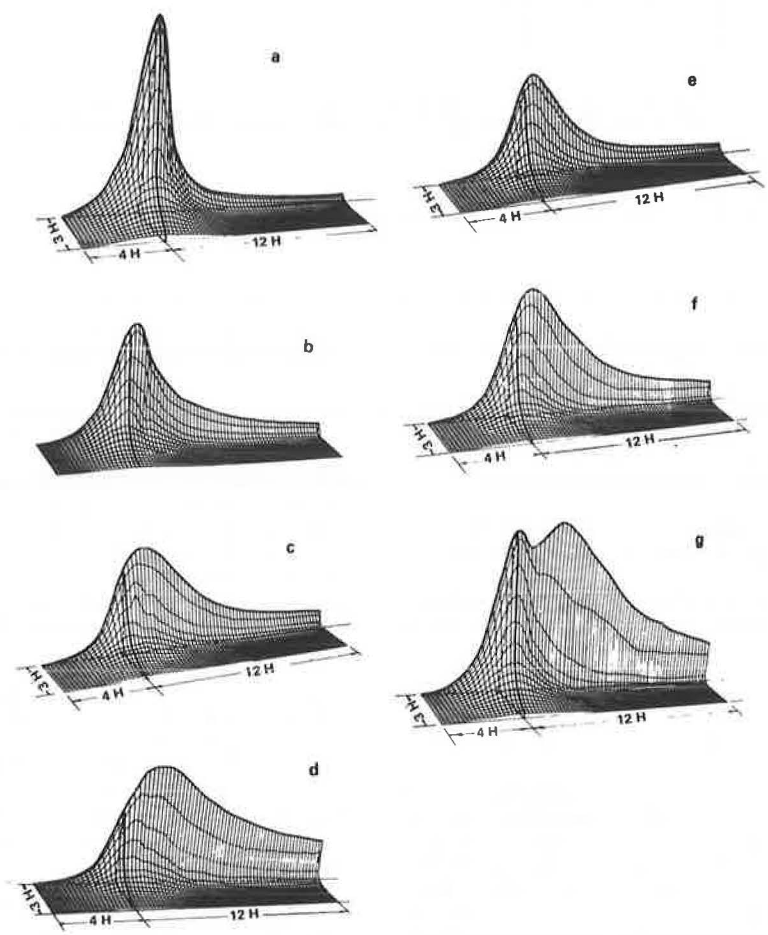

FIGURE 11 Visual representations of reflectance matrices.

INVESTIGATION OF CHANGES IN VIEWING ANGLE

All measurements and classification work on reflectance matrices to date were based on a viewing angle of 1 degree (α = 1 degree). This angle, as a rounded value, is related to (what was believed to be) the prevailing or most critical viewing distance of an automobile driver, namely 80 to 100 m ahead of his current position, so that he could see a critical size object (a 20 cm cube) in time to take evasive action. Drivers of trucks, buses, and vans, however, view objects from a more elevated eye level and their viewing angle for the same distance ahead is larger than 1 degree. On the other hand, drivers of sports cars may view the road surface from an angle much smaller than 1 degree. Further, it would simplify field measurements of luminance on road surfaces if the viewing angle could be set to a larger angle of, for example, 1.5 or 2 degrees without substantial error or difference in the results. For all these reasons, it is important to study the influence of the viewing angle α.

The photometer shown in Figure 6 was modified to allow an adjustment of the angle of view from α = 0.75 degrees to α = 3 degrees. Reduced reflectance coefficients were measured on three core samples, each from three different sections of Highway 401 (Sections 2, 11, and 14). Figures 12-14 show the averages from three samples of the parameters Q0, S1, and S2 plotted versus the angle α. The following observations can be made:

- In Figure 12, there is little difference in Q0 for α = 0.75 degrees and α = 1 degree, but there is a sharp drop in Q0 from α = 1 to 1.5 degrees and some further decrease toward α = 2 degrees. The total drop in Q0 is about 12 to 15 percent.
- In Figures 13 and 14 there is also a downward trend of the specularity parameter with increasing viewing angle α up to 10 percent at α = 2 degrees. All these drops in parameters appear to level off between α = 2 to 3 degrees.

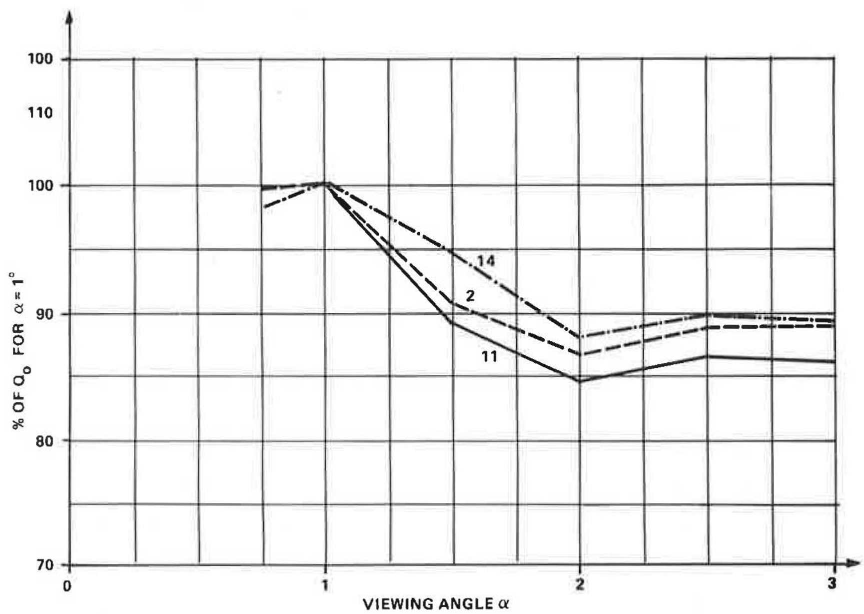

FIGURE 12 Brightness parameters Q0 versus viewing angle α.

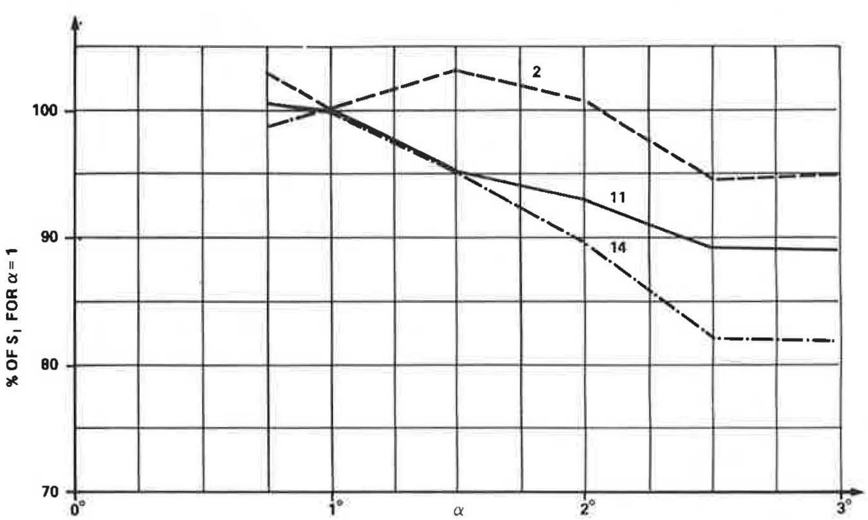

FIGURE 13 Specularity parameter S1 versus viewing angle α.

- Generally speaking, viewing angles of 2 or 3 degrees result in less specularity and less overall brightness compared with the standard 1-degree angle.

CONCLUSIONS AND RECOMMENDATIONS

It is possible to measure pavement reflectance matrices using the photometer equipment built at the University of Toronto, based on averages of three core samples of 150 mm (6 in.) diameter, and to classify most pavement types within the CIE system.

The more than 400 samples measured in Ontario represent about 100 pavement types including differences of wear under traffic, that is, counting the driving, center, and passing lanes of the same section as different types.

Reflectance parameters Q0, S1, and S2 were established for each type from matrix tables of reduced reflectance coefficients measured on at least three samples from each type. All pavement types were then classified in accordance with the CIE or IES classes R1, R2, R3, and R4; in accordance with the IES classes N1, N2, N3, and N4; and in accordance with the 240 standard surfaces in the Erbay Atlas.

Some pavement types were subjected to more measurements and to a subsequent statistical analysis in order to obtain an estimate of standard deviations for various levels of such experimental measurements. It was found that measurement procedure

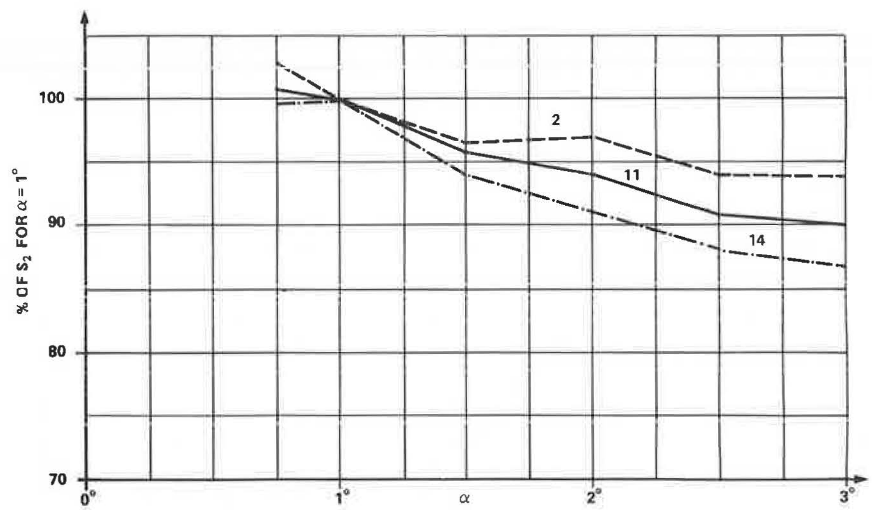

FIGURE 14 Specularity parameter S2 versus viewing angle α.

TABLE 7 Recommended Design Values for Southern Ontario

COMPOSITION	R or N CLASS	BRIGHTNESS
steel slag, open grade	R2, N3, R3	Q0 = 0.06
traprock, open grade	R3, R4	Q0 = 0.07
blend of igneous & lime, open grade	N3	Q0 = 0.09
limestone, open grade	N3	Q0 = 0.10
steel slag, dense friction course	R4 or N4	Q0 = 0.075
blast furnace slag, dense friction course	R2 or N2	Q0 = 0.075
traprock, dense friction course	R2, N3, R3	Q0 = 0.065
blend of igneous & lime, dense f.c.	R3 or N3	Q0 = 0.085
limestone, dense friction course	R2, N2, R3, N3	Q0 = 0.10
concrete limestone plain	R3 or N3 old: R4	Q0 = 0.12
concrete limestone lateral grooves	R3 or N3 old: R4	Q0 = 0.095

NOTE: The higher specularity class is valid for older pavements.

or small sample size were not critical for any kind of classification, but that variations in the surface texture of a lane or section sometimes exceed specified classification boundaries. Sometimes outside and inside wheelpath textures fall in two different classes but these were nevertheless recorded as an average in this paper.

The aforementioned classification was carried out with regard to specularity only, and the four classes, either R or N, can be regarded as sufficiently accurate for design purposes. However, the parameter Q0 should be estimated more accurately by considering the surface course composition and aggregate.

Asphalt pavements exposed to traffic become gradually brighter and more specular, which is reflected in increases of Q0 and S1 (and S2), respectively. The physical reasons are that aggregates become more exposed or cleansed of asphalt and more polished or flattened.

More specifically, with regard to the luminance method of design, the data in Table 7 are presented and can be used for the necessary input of reflectance parameters.

Some measurements were carried out with varying viewing angle α. It was found that brightness (Q0) and specularity (S1/S2) decrease somewhat with increasing α toward 1 or 2 degrees.

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Influence of Leading Vehicle Turn Signal Use on Following Vehicle Lane Choice at Signalized Intersections

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ABSTRACT

The findings of a phenomenological study of a rarely addressed subject are discussed: the degree to which turn signals are properly used at signalized intersections and the effect that nonuse has on the lane-choice behavior of subsequent through vehicles. The situation studied involved a lane drop at the far side of the intersection. Three experiments were conducted at two locations to observe the lane preferences of isolated subject vehicles and three cases of car-following. The study revealed that a considerable proportion of left turners failed to properly indicate their movement intentions and this had a significant effect on following through vehicles. Lane choice was also found to be affected by the distance to the lane drop and by the traffic signal display. On the basis of these findings additional study of this subject is recommended.

The driving task involves the response of a driver to numerous stimuli generated by the environment, the traffic control system, and other vehicles on the roadway. Cues from other vehicles are given by their location, their status, and their actions, current or impending. Because of their critical nature in terms of traffic safety, certain leading vehicle actions are accompanied by reinforcing warnings to following drivers. A prime example of this situation is the universal use of brake lights. Concerning these, Rockwell and Treiteter (1) conducted an experiment relating to "both night and day study of car following for no signal display, for the conventional brake light, for the tri-light system denoting brake and accelerator action, and for an acceleration information display of horizontal rows of green and red lights to indicate the magnitude of the leading vehicle's acceleration or deceleration."

It is noteworthy that the preceding study investigated alternative ways by which information about the actions of the leading vehicle are relayed to the follower automatically in the hope that the resulting improvement in intervehicular communication would contribute to the enhancement of traffic safety and efficiency.

Another method of communication between vehicles is the turn signal, which is used to apprise other vehicles of impending lane-changing or turning movements. The study of turn signals appears to have been generally confined to their design aspects emphasizing the ability of other drivers to perceive and discriminate the message conveyed. Thus, a study by Attwood (2) investigated "a driver's ability to detect and interpret rear-signal information... (for) ... two types of signals, one with brake and turn-signal combined under the same lense, the other with brake and turn-signals under separate lenses."

Incidentally, in his randomized-block factorial design, Attwood also examined the effect of four levels of blood alcohol and six levels of stimulus complexity. Another study by Lea and Associates (3) attempted to discover whether the color of turn signals (i.e., red versus amber) had any effect on the accident involvement of vehicles.

It is of interest to note that the differences sought by these studies are conditional on the actual use of turn signals when warranted. But, unlike the case of brake lights when the signal is given automatically, the use of turn signals involves a good deal of driver discretion and, in fact, turn signals are not always used even when required by the traffic codes that typically provide "... whenever the operation of any other vehicle may be affected by this movement (starting, turning, or stopping), the driver shall give a signal plainly visible to the driver of such other vehicle of the intention to make such movement," and, "a signal of the intention to turn right or left when required shall be given continuously during not less than the last 100 feet travelled by the vehicle before turning (4)."

Vehicle inspection programs required by many states invariably include the testing of the operating condition of turn signals. Again, the physical soundness of turn signals would be immaterial in those instances when the driver fails to use them.

The findings of a small-scale study are reported that attempted to quantify the degree and impact of turn signal use in a rather restricted case representing an early step in this direction. Specifically, the study measured the frequency with which certain leaders of vehicular pairs do signal their intention to execute a left turn at signalized intersections and the effect that signaling and the lack of it have on the lane-choice behavior of the following-through vehicle.

THE SITUATION STUDIED

The study addressed the situation where through vehicles approaching a signalized intersection have a choice between using the center lane of the approach in common with left turners and the adjacent lane that is designated for the exclusive use of through vehicles. The choice of the exclusive through lane presents the possibility of a penalty due to the need to merge left on the far side of the intersection because of a lane drop. Thus, the choice of the center lane places a through vehicle under the risk of being delayed by left turners ahead, whereas the choice of the exclusive through lane may cause a through vehicle delays in having to yield to vehicles occupying the center lane within the length of the merging area. The study attempted to quantify the effect that the display versus the failure to display a turn signal has on the lane choice of subsequent through vehicles. The influence of two other factors was also examined. These factors were the traffic signal phase (i.e., red or green) and the length of the merging area on the far side of the intersection.

EXPERIMENTAL SITES

The experiment was originally envisioned to take place at a single site. This plan was subsequently modified to include a second location where the length of the merging area could be controlled. Figure 1 shows the first site, where observations were made on the eastbound Dole Street approach to the T-intersection shown. The approach consists of two 10-ft lanes. No separate left-turn lane and no special turn phasing of the traffic signal was in effect during the periods of data collection. Thus, the center lane of the approach was open to both through vehicles and left turners into the East-West Road. Moreover, the curb lane is dropped at a distance of 250 ft from the intersection because of a narrow bridge.

Subsequent to the collection of data at the first intersection, a second site was selected in order to consider the effect of locational differences (Experiment 2) and the effect of merging area length (Experiment 3). Figure 2 shows the geometric characteristics of the second site, that is, the northbound Keeaumoku Street approach to its intersection with Wilder Avenue. The approach consists of three

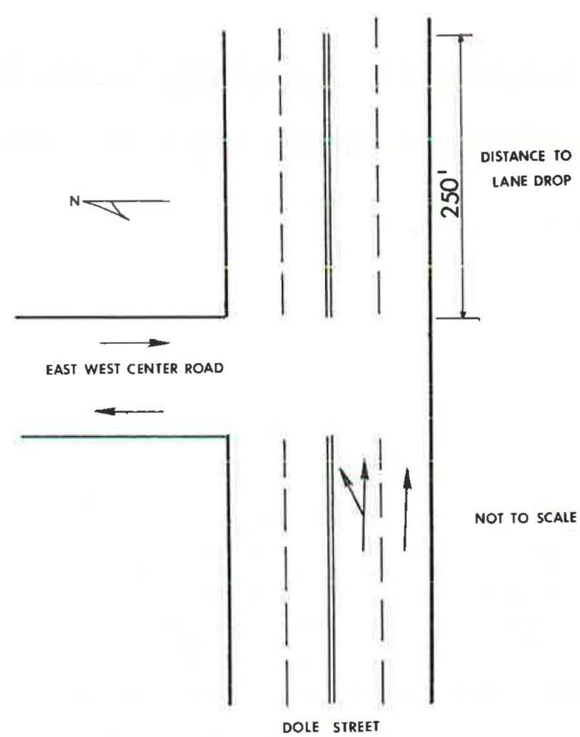

FIGURE 1 The Dole Street site.

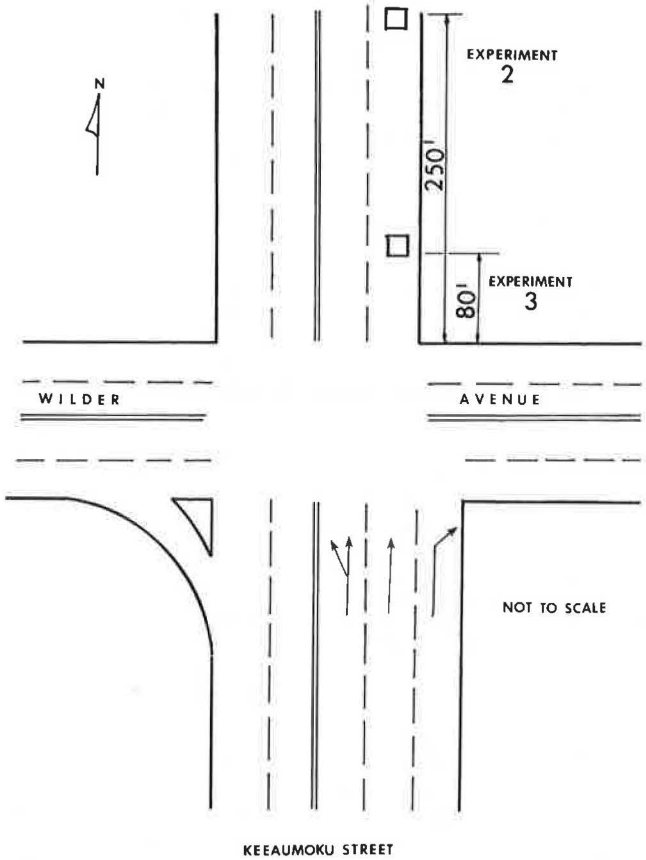

FIGURE 2 The Keeaumoku Street site.

lanes: an exclusive right-turn lane, an exclusive through lane, and a through-and-left center lane. The latter two lanes were relevant to the study.

Parenthetically, because of the light level of traffic examined, any frictional effects between the right-turn lane and the middle lane were considered to be unimportant. The lane drop at this location was due to the presence of a parking lane on the far side of the intersection. Consequently, the distance available for merging could be controlled by the location of the first parked vehicle encountered by through traffic. Two sets of observations were taken at this location. The first corresponded to a merging distance about equal to that of the Dole Street site, the other involved a parked vehicle at a distance of about 80 ft from the intersection (or about one-third the distance in the other two experiments). In all three experiments, the observations were disaggregated according to the traffic signal phase.

EXPERIMENTAL PLAN

The operation of signalized intersections gives rise to a great diversity of vehicular interactions caused by a variety of factors including (but not limited to) the level of demand at the intersection. For example, the lane choice of an approaching through vehicle is affected not only by a turn signal display ahead but also by the number and lane occupancy of vehicles between the subject vehicle and the intersection. Because the main objective of the study was to quantify the effects of turn signals, it became necessary to confine the field observations to cases where the influence of other factors was reasonably reduced. As a result, the test observations were restricted to cases that involved a maximum of two vehicles in a car-following situation, the second vehicle of a pair being a subject vehicle. For purposes of comparison, data relating to the lane choice of isolated through vehicles were also collected. In all three experiments, the response variable consisted of the lane choice of subject (i.e., through) vehicles. Including the case of isolated through vehicles and depending on the action engaged in by the leading (i.e., stimulus-inducing) vehicle in a pair, the following cases were identified for field measurement.

1. Isolated subject vehicle,
2. Subject vehicle subsequent to a left turner that displayed a turn signal,
3. Subject vehicle subsequent to a vehicle in the center lane but displaying no turn signal, and
4. Subject vehicle subsequent to a vehicle in the exclusive through lane.

Figure 3 shows the four cases that were observed under green and red traffic signal conditions at each experimental site. The observations that were included in Case 3 were categorized according to whether the leading vehicle traveled straight ahead, whether it turned left without signaling, or whether it displayed the turn signal late. A delayed display was one that occurred after the subject vehicle committed itself to a particular lane. This event was most pronounced during the red phase in instances when the turn signal was displayed after the subject vehicle came to a complete stop or even after the leading vehicle entered the intersection during the subsequent green phase. The effect of turn signal use on the lane choice of subject vehicles could be found by comparing Cases 2 and 3. As far as a subject vehicle is concerned, a leading vehicle in the center lane that displays no turn

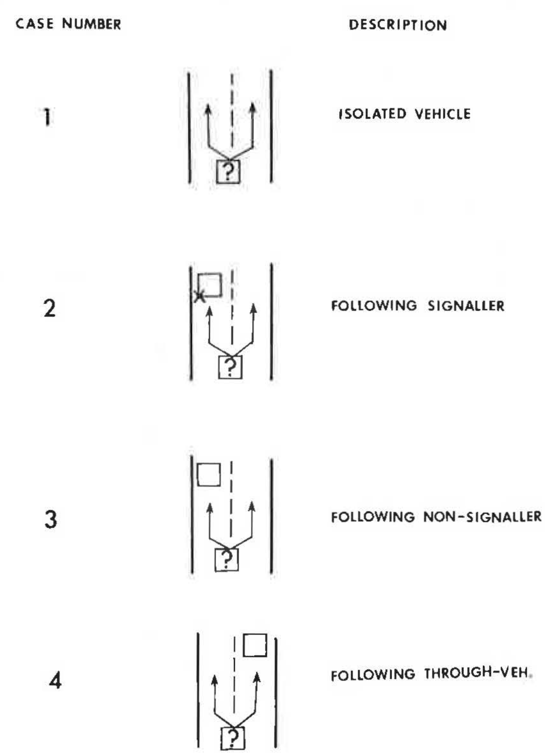

FIGURE 3 The cases studied.

signal may be either a left turner or a through vehicle, a situation that gives rise to the lane-choice predicament described earlier.

DATA COLLECTION

An observer equipped with a special form on which the relevant cases were listed was positioned at a distance of approximately 250 ft upstream from the intersection site. The observer viewed vehicles as they approached the intersection and manually recorded on the forms the lane choice of vehicles qualifying as subject vehicles. All observation sessions were conducted under clear and dry conditions during off-peak periods when excessive queueing was not present. The observer visited the sites on a number of days and continued to collect data until a sufficient sample of about 50 observations became available for the least frequently occurring case. Incidentally, the case in which the leading vehicle of a pair occupied the through lane ahead of a subject vehicle was the rarest of the sought-after cases for all three experiments.

As noted earlier, data corresponding to the four cases were collected for both the red and green displays of the traffic signal. It should be indicated here that the assessment of car-following dynamics that were being observed presented relatively more difficulties of judgment during the green as compared with the red phase. This was true because during the green phase drivers have more freedom to control and adjust their speed, to overtake, and to maneuver their vehicles. In addition, other clues besides the turn signal are available to them regarding impending movements by vehicles ahead. For example, a leading vehicle that is motionless within the intersection, especially in the presence of opposing traffic, is most probably in the process of executing a left turn irrespective of whether it displays a turn signal or not. Moreover, late lane changes and overtaking would be much easier during the green phase under the traffic levels considered because the subject vehicle is not required to come to a stop but can proceed to clear the intersection without severe interruptions. For these and other reasons, it was the opinion of the observer that the data obtained during the red phase are more reliable than those obtained during the green phase.

DATA ANALYSIS

Degree of Turn Signal Use

Table 1 presents the overall counts of the field observation sessions for the three experiments conducted at the two locations described earlier. Experiment 1 corresponds to the Dole Street site, Experiment 2 corresponds to the Keeaumoku Street location when the merging distance was approximately the same as in Experiment 1, and Experiment 3 corresponds to the Keeaumoku site when the merging distance was about 80 ft or one-third of that which existed during the other two experiments. The four cases within each experiment are those shown in Figure 3.

In Table 2a the data in Case 2 (i.e., late or no signal) are disaggregated according to whether the leading vehicle eventually proceeded through the intersection straight ahead or whether it executed a left turn. These findings reveal the degree to which leaders of vehicular pairs in the center lane who did not display the turn signal were, in fact, turning left (Column 5). Also shown in the table are the observed numbers of left-turning leaders who probably used the turn signal (Column 1). Using these values, it is possible to compute the percentage of left-turning leaders of vehicular pairs who failed to properly give an indication of their intended maneuver (Column 4). It is clear that both categories constituted considerable proportions of their respective totals. That is, based on frequency alone, the proportion of drivers who neglected to use the turn signal cannot be ignored.

The data in Table 2a indicate that there exist differences in turn signal use apparently related to the traffic control phase. The data in Table 2b aid the examination of this possibility by presenting the results of chi-square tests that compared the use of turn signals between the green and red phases for each of the three experiments. Adapting the terminology of <u>same</u> for failing to reject the hypothesis of equality of proportions at the 0.05 level of significance, <u>different</u> for rejecting the hypothesis at the 0.5 level but failing to do so at the 0.01 level, and <u>very different</u> for rejecting the hypothesis at both levels, the data in Table 2b indicate that the percentage of nonsignaling left turners (irrespective of how computed) was affected by the traffic signal control at the short-merging-distance Keeaumoku Street site. For the other Keeaumoku experiment, the computed chi-square value approaches the theoretical 0.05-level value. In all other instances, the hypothesis of equality could not be rejected. Combined, these findings may portend other differences besides the length of the merging area. To examine this possibility further, the chi-square test was employed to compare the proportions of nonsignalers between experiments.

The data in Table 2c indicate the results of pairwise comparisons between experiments when non-

TABLE 1 Lane-Choice Data

	Green Phase				Red Phase			
Case	Center Lane	Through Lane	Center (%)	Through (%)	Center Lane	Through Lane	Center (%)	Through (%)
1	162	84	66	34	104	46	69	31
	129	57	69	31	112	39	74	26
	163	72	69	31	157	34	82	18
2	27	123	18	82	9	70	11	89
	20	75	21	79	15	80	16	84
	30	84	26	74	22	56	28	72
3	97	42	70	30	57	33	63	37
	43	23	65	35	65	40	62	38
	53	24	69	31	89	62	59	41
4	49	23	68	38	58	11	84	16
	31	20	61	39	47	8	85	15
	34	17	67	33	43	7	86	14

Note: The three rows associated with each case correspond to Experiments 1, 2, and 3, respectively.

TABLE 2 Degree of Turn Signal Use

(a) Raw Data

EXPERIMENT	GREEN PHASE					RED PHASE				
	(1) CASE #2 TOTAL	(2) CASE #3 LEFT	(3) CASE #3 THROUGH	(4) (2) AS % OF (1)+(2)	(5) (2) AS % OF (2)+(3)	(1) CASE #2	(2) CASE #3 LEFT	(3) CASE #3 THROUGH	(4) (2) AS % OF (1)+(2)	(5) (2) AS % OF (2)+(3)
1. Dole	150	81	58	35	58	79	50	40	39	56
2. Keeaumoku	95	31	35	25	47	95	34	71	26	32
3. Keeaumoku	114	37	40	25	48	78	51	100	40	34

(b) Green versus red

EXPERIMENT	(2) AS % OF (2)+(3)	(2) AS % OF (1)+(2)
1. Dole	0.165* (same)**	0.488 (same)
2. Keeaumoku (long)	3.661 (same)	0.103 (same)
3. Keeaumoku (short)	4.386 (diff.)	7.294 (very diff.)

(*) computed chi-square

(**) see text

(c)

EXPERIMENT	GREEN		RED	
	1	2	1	2
2	2.307 (same)	—	6.574 (very diff)	—
3	2.089 (same)	0.017 (same)	10.988 (very diff)	0.054 (same)

(d)

EXPERIMENT	GREEN		RED	
	1	2	1	2
2	4.145 (diff)	—	4.519 (diff)	—
3	4.771 (diff)	0.000 (same)	0.016 (same)	5.701 (diff)

signalers are taken as the proportion of all nonsignaling leaders of vehicular pairs traveling in the center lane (i.e., Case 3). With respect to the observations taken during the green phase, the three experiments indicate no differences at the 0.05 level of significance. During the red phase, on the other hand, the two Keeaumoku Street experiments were found to be the same but were either different or very different when compared with the Dole Street experiment: a lower percentage of nonsignalers were found in the former as compared to the latter (i.e., combined 33 percent versus 56 percent). In this instance, then, a locational difference emerged. Parenthetically, the major differences between the two locations include the existence of an exclusive right-turn lane at Keeaumoku, the potential conflicts between subject vehicles with opposing traffic turning left at the same site, and possibly some uncertainty on the part of drivers approaching the Keeaumoku intersection about the length of the merging zone on the far side of the intersection.

According to Table 2c, the same "locational" effect between Dole Street and Keeaumoku Street is observed with respect to the percentage among all observed left turners that failed to properly signal during the green phase. However, the percentage corresponding to the red phase indicated mixed results: a higher proportion of nonsignalers was found at the short-merging distance site vis-a-vis the other Keeaumoku experiment. Moreover, this higher percentage was statistically the same as that corresponding to the Dole Street experiment. Without this anomaly, the pattern of locational differences would be the same as before, although more pronounced.

Behavior of Subject Vehicles

The lane-choice behavior of through vehicles admissible within the cases identified for the purposes of the study were analyzed next. The overall results of the observation sessions that are given in Table 1 were subjected to three groups of analysis as follows:

1. Case by case comparisons between pairwise combinations of experiments (Table 3);
2. For each experiment, case by case comparisons between the red and green phases of the traffic signal (Table 4); and
3. For each experiment, comparisons between cases by traffic control phase (Table 5).

All analyses applied the chi-square test. The results are discussed using the terminology of same, different, and very different as defined previously.

Table 3 presents the experimental chi-square values that resulted from the case by case comparisons by traffic control phase between pairs of experiments. The data in Table 3a indicate that the lane-choice behavior of subject vehicles for the two long-merging-distance experiments at Dole and Keeaumoku Streets was the same. On the other hand, the short-merging-distance experiment showed some differences from both long-merging-distance experiments: the results were found to be the same for all cases under green and for Cases 3 and 4 under red, but differences were detected in Cases 1 and 2 under red.

The lane-choice proportions in the case of isolated subject vehicles (Case 1) and in the case of

TABLE 3 Case by Case Comparisons for Pairs of Experiments

(a) Experiments 1 vs. 2

CASE	GREEN	RED
1	0.591 (same)	0.869 (same)
2	0.350 (same)	0.701 (same)
3	0.444 (same)	0.042 (same)
4	0.694 (same)	0.046 (same)

(b) Experiments 1 vs. 3

CASE	GREEN	RED
1	0.675 (same)	7.745 (very diff)
2	2.646 (same)	7.001 (very diff)
3	0.021 (same)	0.456 (same)
4	0.026 (same)	0.085 (same)

(c) Experiments 2 vs. 3

CASE	GREEN	RED
1	0.000 (same)	3.236 (same)
2	0.789 (same)	3.927 (diff)
3	0.218 (same)	0.227 (same)
4	0.382 (same)	0.006 (same)

TABLE 4 Case by Case Comparisons by Signal Phase

CASE NO.	EXPERIMENT 1	EXPERIMENT 2	EXPERIMENT 3
1	0.512 (same)	0.949 (same)	9.290 (very diff)
2	1.705 (same)	0.876 (same)	0.084 (same)
3	1.032 (same)	0.184 (same)	2.124 (same)
4	4.931 (diff)	8.286 (very diff)	5.209 (diff)

subject vehicles behind signaling left turners (Case 2), were found to be very different between the short-merging-distance experiment at Keeaumoku Street vis-a-vis the Dole Street experiment (Table 3b). This difference emerges in a milder form (apparently due to locational similarities) from the comparison of the two Keeaumoku Street experiments (Table 3c). The general conclusion that may be drawn here is that the length of the merging zone affects the lane choice of isolated vehicles and through vehicles that are behind a signaling left turner. The raw data in Table 1 quantify the reasonable expectation that a larger proportion of subject vehicles would choose the center lane when the merging distance is shortened. The contribution of the traffic control on this tendency is examined next.

The data in Table 4 indicate the results of an analysis that compared separately for each experiment the lane-choice behavior of subject vehicles for each of the four cases when the traffic control phase is varied. In all three experiments the traffic signal was found to affect the lane choice of through vehicles subsequent to a leader who occupied the through lane (Case 4): proportionately more subject vehicles chose the center lane during the red phase than during the green phase. Additionally, the short-merging-distance experiment showed a peculiarity vis-a-vis the two long-merging-distance experiments: more isolated subject vehicles chose the center lane on red as compared to green in the instance of the short-merging-distance experiment. This finding, of course, is both reasonable and consistent with the findings of earlier analyses (Table 3), which, when taken together with the present findings, reveal a strong interaction between the merging distance and the traffic signal in the case of isolated vehicles (Case 1), but, in the case of vehicles subsequent to a signaling left turner (Case 2), the merging distance alone appears to be the predominant factor.

Finally, the question of the effect of turn signal use or non-use on the lane choice of subject vehicles was examined by comparing their lane-choice proportions between the various cases for each experiment. The data in Table 5 indicate overwhelming preference for the through lane when comparing proportions in the case of signaling leaders (Case 2) and all other cases irrespective of the traffic signal display. Of interest is that, during the green phase, no difference was detected in lane choice in the presence of nonsignaling leaders traveling in the center lane (including eventual left turners) vis-a-vis cases not involving a signaling left turner. This finding was not consistently true during the red phase. The comparison of vehicles following a leader in the through lane (Case 4) versus the case of vehicles following nonsignaling leaders in the center lane (Case 3) indicated a consistent difference attributable to a higher proportion of center-lane users among the former in agreement with earlier findings (Table 4). The two locations were reversed with respect to the remaining two comparisons in Table 5b, both involving the case of isolated vehicles. At Dole Street, no difference is observed between isolated vehicles and vehicles following a nonsignaling leader in the center lane. Additionally, at the same location, more center lane users were found among vehicles subsequent to leaders in the through lane than among isolated vehicles. In the last two comparisons, the reverse was found to be true at the Keeaumoku site. The cause of this reversal appears, more than anything else, to be a higher preference for the center lane on the part of isolated vehicles on Keeaumoku Street, a situation that may be explainable by the locational differences discussed earlier.

TABLE 5 Between-Case Comparisons by Signal Phase

	(a) Green Case #1	(a) Green Case #2	(a) Green Case #3	(b) Red Case #1	(b) Red Case #2	(b) Red Case #3
Case #2	85.530 (very diff)			69.501 (very diff)		
Case #3	0.623 (same)	78.975 (very diff)		0.917 (same)	47.686 (very diff)	
Case #4	0.121 (same)	54.140 (very diff)	0.066 (same)	5.322 (diff)	78.494 (very diff)	8.381 (very diff)

Experiment No. 1

	(a) Green Case #1	(a) Green Case #2	(a) Green Case #3	(b) Red Case #1	(b) Red Case #2	(b) Red Case #3
Case #2	58.900 (very diff)			79.589 (very diff)		
Case #3	0.397 (same)	31.797 (very diff)		4.368 (diff)	44.194 (very diff)	
Case #4	1.340 (same)	23.048 (very diff)	0.236 (same)	2.914 (same)	69.716 (very diff)	18.186 (very diff)

Experiment No. 2

	(a) Green Case #1	(a) Green Case #2	(a) Green Case #3	(b) Red Case #1	(b) Red Case #2	(b) Red Case #3
Case #2	57.541 (very diff)			72.523 (very diff)		
Case #3	0.008 (same)	33.808 (very diff)		22.594 (very diff)	19.453 (very diff)	
Case #4	0.142 (same)	24.164 (very diff)	0.066 (same)	0.406 (same)	40.719 (very diff)	12.200 (very diff)

Experiment No. 3

SUMMARY AND CONCLUSIONS

The results of a small-scale exploratory phenomenological study of a rarely addressed subject have been discussed: the degree of use of turn signals by left turners at signalized intersections and the effect of their use and non-use on the lane-choice behavior of subsequently approaching through vehicles. The specific situation studied involved a lane drop at the far side of the intersection, which gave rise to a predicament on the part of subject vehicles relating to the choice of lane between a common left/through center lane and a through lane adjacent to it. For comparative purposes, data on the lane choice of isolated subject vehicles and of subject vehicles subsequent to a leader in the exclusive through lane were also collected. Three experiments were conducted in an attempt to determine the effect of turn signal use as distinguished from the effect of other factors such as the traffic signal display and the merging distance to the lane drop.

The study revealed that a considerable proportion of left turners failed to properly indicate their movement intentions. Between 30 and 60 percent of the nonsignaling leaders observed eventually turned left, and between 25 and 40 percent of the left-turning leaders observed failed to properly use the turn signal.

The study also quantified the following reasonable phenomena:

1. A shorter merging distance was found to increase the percentage of followers of signaling left turners who chose the common lane;
2. An interaction effect was found between traffic signal control and merging distance on the lane-choice behavior of isolated subject vehicles; and
3. The traffic signal control was found to affect the lane choice of vehicles subsequent to leaders occupying the through lane.

As would be expected, subject vehicles following signaling left turners indicated an overwhelming preference for the through lane vis-a-vis the common left-and-through approach lane. It is of interest, however, that in certain instances subject vehicles behaved similarly when isolated from other vehicles as compared to cases in which they followed nonsignaling leaders, even though the latter could be eventually executing a left turn. In other words, a number of subject vehicles would have selected a different approach lane had they been apprised of the turning intention of the vehicle ahead. The ultimate implication of this discrepancy on traffic safety and efficiency must await further examination. With respect to traffic safety, a study of the correlation between certain types of conflicts on the one hand and the pattern of turn signal use on the other within the general framework of conflict analysis (5) may prove fruitful. In addition, based on the findings of the current study, it appears that the investigation of other situations involving the use of turn signals (for example, lane changing) and the study of more complex cases in the vicinity of signalized intersections are warranted.

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Cost-Effectiveness Evaluation of Rural Intersection Levels of Illumination

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ABSTRACT

Lighting is often installed at rural intersections to improve the safety of night traffic operations at these locations. However, there are no generally accepted design criteria that define the levels of illumination required at rural intersections. The objective of this research was to evaluate the cost-effectiveness of rural intersection levels of illumination. Six lighting systems were installed at a rural, unchannelized intersection of two two-lane highways. Speed-profile and traffic-conflict studies were conducted on an uncontrolled approach to the intersection. The studies were conducted at night at each level of illumination as well as with no lighting. The data were analyzed to determine the safety- and cost-effectiveness of each level of illumination. The results of the research indicated that, for a given luminaire wattage, two-luminaire systems provided safer traffic operations than did one-luminaire systems; and the safest operations were observed under a two 200-watt high-pressure-sodium (HPS) luminaire system. The results of the cost-effectiveness analysis revealed that lighting was not warranted at rural intersections with main highway average daily traffic less than 3,250 vehicles per day. At higher volume intersections a two 200-watt HPS luminaire system was the most cost-effective.

Lighting is installed at rural intersections to improve the safety of night traffic operations at these locations. But although several studies (1) have found improvements in highway safety as a result of intersection lighting, generally accepted warrants that define the circumstances under which the costs of installing and maintaining rural intersection lighting are justified do not exist. In addition, current guidelines for fixed roadway illumination (2-4) do not deal directly with rural intersection lighting. Thus, there are no generally accepted design criteria that define the levels of illumination required at rural intersections.

The objective of the research reported here was to evaluate the cost-effectiveness of rural intersection levels of illumination. In this research the safety-effectiveness of six levels of illumination on an uncontrolled, unchannelized approach to a rural intersection, as well as a condition of no lighting, was compared, and the most cost-effective level of illumination was identified. The procedure, findings, and conclusions of this research are presented in this paper.

PROCEDURE

Traffic-operations studies were conducted on an uncontrolled, unchannelized approach to a four-legged intersection of two rural highways. The studies consisted of the measurement of speed profiles of vehicles on the approach and the simultaneous observation of traffic conflicts on the approach. The studies were conducted at night at six different levels of illumination as well as with no lighting at the intersection.

Initially the study site was not lighted, and the traffic-operations studies were conducted at night with no illumination. When these initial studies were completed, lighting was installed at the intersection. The six lighting systems that were installed and studied are listed in Table 1. The average horizontal levels of illumination and the uniformity ratios maintained within the intersection by the six lighting systems are also given in Table 1.

TABLE 1 Lighting Systems Studied

Lighting System[a]	Avg Maintained Horizontal Illumination[b] (footcandles)	Avg/Min Uniformity Ratio[b]
One 100-watt HPS[c]	0.27	5.4
Two 100-watt HPS	0.53	2.5
One 200-watt HPS	0.39	7.8
Two 200-watt HPS	0.78	3.1
One 400-watt HPS	0.77	15.5
Two 400-watt HPS	1.70	3.3

[a]The luminaires were mounted at 400 ft and located in a catercorner configuration. The luminaire in the one-luminaire system was located on the far side of the intersection relative to the study approach.
[b]Within the intersection.
[c]High-pressure-sodium, Type II, medium-distribution, cutoff luminaires.

The speed-profile and traffic-conflicts data collected were analyzed to assess the safety effects of the six levels of illumination and a condition of no lighting. The following measures of safety-effectiveness were computed from the data for each level:

- Standard deviation of the average approach speed,
- Standard deviation of the deceleration between 900 and 300 ft before the intersection,
- Standard deviation of the deceleration between 300 and 100 ft before the intersection, and
- Overall traffic-conflicts rate.

It was assumed that the safety of traffic operations improved with lower values of each of these measures.

In addition to the calculation of these measures of safety-effectiveness, a cost-effectiveness analysis was conducted to determine the most cost-effective level of illumination. The measure of cost-effectiveness used was total annual cost: the sum of the annual cost of installing and maintaining the lighting system plus the annual cost of accidents expected to occur on the uncontrolled approaches with its use. The lighting system with the lowest total annual cost was determined to be the most cost-effective level of illumination.

The research used the Omaha Public Power District's (5) annual costs for the installation and maintenance of lighting systems. (This was the power district within which the study site was located.) The specific costs used were those for installations typical of rural intersection lighting projects designed by the Nebraska Department of Roads.

The annual accident costs were computed by using the average accident rate on the uncontrolled approaches to unchannelized, unlighted rural intersections of two-lane highways in Nebraska. It was determined from nearly 4 years of accident data (6) that the average accident rate was 1.06 accidents per million entering vehicles. It was also determined that the accident severity distribution was 5 percent fatal, 48 percent nonfatal injury, and 47 percent property-damage-only accidents. An expected accident cost of $12,175 per accident was computed by applying this distribution to 1980 accident costs of the National Safety Council.

The effect of level of illumination on the accident rate was assumed to be in proportion to the ratio of its expected accident involvement rate to the expected accident involvement rate of the condition of no lighting. The expected accident involvement rates of the levels of illumination and the condition of no lighting were computed by applying their observed average approach speed distributions to the relationship between night accident involvement rate and speed, which was determined by Solomon (7) from a study of rural highway sections. In addition to determining the most cost-effective level of illumination for the traffic volume conditions at the study site, the cost-effectiveness analysis was also conducted over the range of average daily traffic (ADT) levels from 500 to 7,500 vehicles per day (vpd), the approximate maximum ADT for the design of rural two-lane highways in Nebraska.

FINDINGS

The measures of safety-effectiveness computed for each level of illumination and a condition of no lighting at the study site are presented in Table 2. These values indicate that with respect to each of the measures the safest traffic operations occurred under the two 200-watt high-pressure-sodium (HPS) luminaire system. In general, according to these measures, traffic operations under the one 100-watt and two 100-watt HPS luminaire systems were not safer than those under no lighting system. Although

TABLE 2 Measures of Safety-Effectiveness of Six Levels of Illumination and Condition of No Lighting at Study Site

Lighting System	Standard Deviation: Avg. Approach Speed (mph)	Standard Deviation: Deceleration 900-300 ft[a] (fps/s)	Standard Deviation: Deceleration 300-100 ft[a] (fps/s)	Overall Traffic Conflicts Rate (no./100 vehicles)
None	8.4	0.81	1.86	3.4
One 100-watt	8.7	0.83	1.94	3.6
Two 100-watt	8.1	0.83	1.78	3.4
One 200-watt	8.1	0.82	2.01	3.0
Two 200-watt	7.6	0.67	1.42	1.9
One 400-watt	7.8	0.68	1.48	2.9
Two 400-watt	7.9	0.68	1.42	2.4

[a]Before the intersection.

these measures indicate that traffic operations under the one 400-watt and two 400-watt HPS luminaire systems were safer than those under no light system and under 100-watt lighting systems, they indicate that the two 200-watt HPS luminaire system had the lowest overall traffic conflicts rate of all systems studied. Also, the measures of safety-effectiveness indicate that at each wattage level traffic operations were safer under a two-luminaire system than they were under a one-luminaire system.

The results of the cost-effectiveness analysis of the levels of illumination at the study site are presented in Table 3. The system that had the lowest annual accident cost was the two 200-watt HPS luminaire system as would be expected from the previous discussion of the measures of safety-effectiveness. In addition, this system had the lowest total annual cost and therefore it was the most cost-effective lighting system at the study site. The total annual costs of the one 100-watt, two 100-watt, and two 400-watt HPS luminaire systems were higher than the cost of no lighting system.

TABLE 3 Total Annual Costs of Six Levels of Illumination and Condition of No Lighting at Study Site

Lighting System	Annual Installation and Maintenance Cost ($)[a]	Annual Accident Rate[b] (no./yr)	Annual Accident Cost[b,c]	Total Annual Cost ($)
None	0	0.263	3,200	3,200
One 100-watt	162.60	0.279	3,397	3,560
Two 100-watt	325.20	0.241	2,929	3,255
One 200-watt	185.88	0.243	2,957	3,145
Two 200-watt	371.76	0.227	2,763	3,145
One 400-watt	226.44	0.240	2,919	3,145
Two 400-watt	452.88	0.234	2,847	3,300

[a]District-owned and maintained system, dusk-to-dawn lighting, steel standards, 40-ft mounting height, and underground wiring.
[b]On the uncontrolled approaches.
[c]Based on 1980 accident costs of the National Safety Council ($170,000 per fatal accident; $6,700 per nonfatal injury accident; and $980 per property-damage-only accident).

The results of the cost-effectiveness analysis conducted over the range of ADT levels representative of those found at unchannelized intersections of rural two-lane highways in Nebraska are shown in Figure 1. These results indicate that for main highway ADTs lower than 3,250 vpd a condition of no lighting system results in the lowest total annual costs. Therefore, no lighting system is warranted at rural, four-legged, unchannelized intersections of two-lane highways with main highway ADT less than 3,250 vpd. However, for main highway ADT greater than 3,750 vpd a two 200-watt HPS luminaire system is warranted.

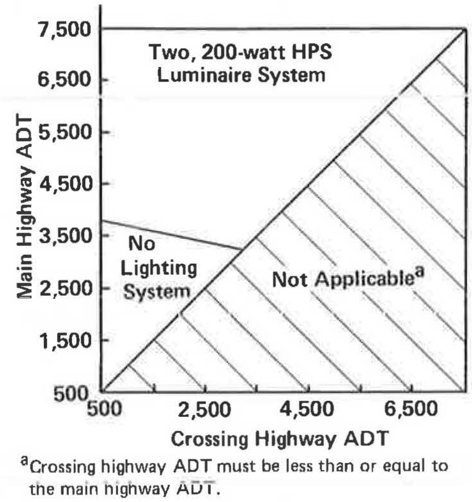

FIGURE 1 Most cost-effective lighting systems at unchannelized intersections of rural two-lane highways in Nebraska.

CONCLUSIONS

Based on the results of this research, the following conclusions were reached on the safety effects of lighting on traffic operations on uncontrolled approaches to unchannelized, two-way stop-sign-controlled intersections of rural two-lane highways.

1. For a given luminaire wattage traffic operations are safer with a two-luminaire system than with a one-luminaire system.
2. The safest traffic operations were observed with a two 200-watt HPS luminaire system.
3. Traffic operations with 100-watt HPS luminaire systems were not safer than those with no lighting system.

The results of the cost-effectiveness evaluation indicated that the most cost-effective lighting system was the two 200-watt HPS luminaire system, but it was only warranted at intersections with main highway ADT greater than 3,750 vpd. No lighting system was warranted at intersections with main highway ADT less than 3,250 vpd.

ACKNOWLEDGMENT

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Using Computer-Generated Pictures to Evaluate Headlamp Beam Patterns

EUGENE I. FARBER and VIVEK D. BHISE

ABSTRACT

A computer graphics system for generating pictures showing a driver's view of the roadway at night, as illuminated by a particular headlamp system, is described. These night scene images realistically depict the brightness patterns on the road surface and the illumination of pedestrians and lane lines produced by the headlamps. Veiling glare and backlighting from opposing headlamps can also be shown. The system was developed to permit comparative evaluations of existing and "drawing board" beam patterns for appearance, subjective qualities, and the visibility of various elements in the driver's field of view. The system also provides a direct pictorial representation of the numerical results obtained from various visibility models. The development of the computer graphics system is described in the context of other analytical approaches to headlamp evaluation that have been developed during the last 10 years. The pictures are pure analytical in origin; they are generated from candlepower tables representing the intensity distribution of a given headlamp and specifications of the shape, location, and reflectance of elements in the visual field. The basic algorithm for generating the pictures, including the geometric and photometric calculations, is discussed in general terms. Pictures representing five different European and U.S. tungsten and halogen systems are presented and compared with regard to appearance, aesthetics, and visibility. Differences between the systems are apparent, and the pictures are in accord with the results of analytical calculations of visibility. The pictures illustrate the complex trade-offs required in beam pattern design and help demonstrate that some important aspects of headlamp performance may not be apparent to the casual observer.

Figure 1 is a computer-generated representation of a driver's eye view of a night highway scene as illuminated by a particular headlamp system--tungsten 4000s in this case. The picture was created by a program called IVIEW developed by the Ford Motor Company Safety Research Office. IVIEW was written to supplement other computer-based analytical approaches to headlamp evaluation that have been developed at Ford during the last 10 years. The picture allows proposed beam patterns to be previewed for appearance and subjective qualities while still on the drawing board. Obviously, this is a method that can be applied equally well to fixed lighting systems. Before describing the IVIEW system, it will be useful to describe the context in which it was developed.

BACKGROUND

Researchers at the Safety Research Office at Ford Motor Company have been working on various headlighting models since the early 1970s. A description

NOTE: The printed reproductions of the computer graphics images accompanying the text have less resolution and dynamic range than the photographic originals from which they were made. The originals were direct color Polaroid prints or 35 mm slides of the image that appears on the computer graphics screen. Readers interested in seeing an example of an original image should contact Eugene Farber at (313) 322-1972.

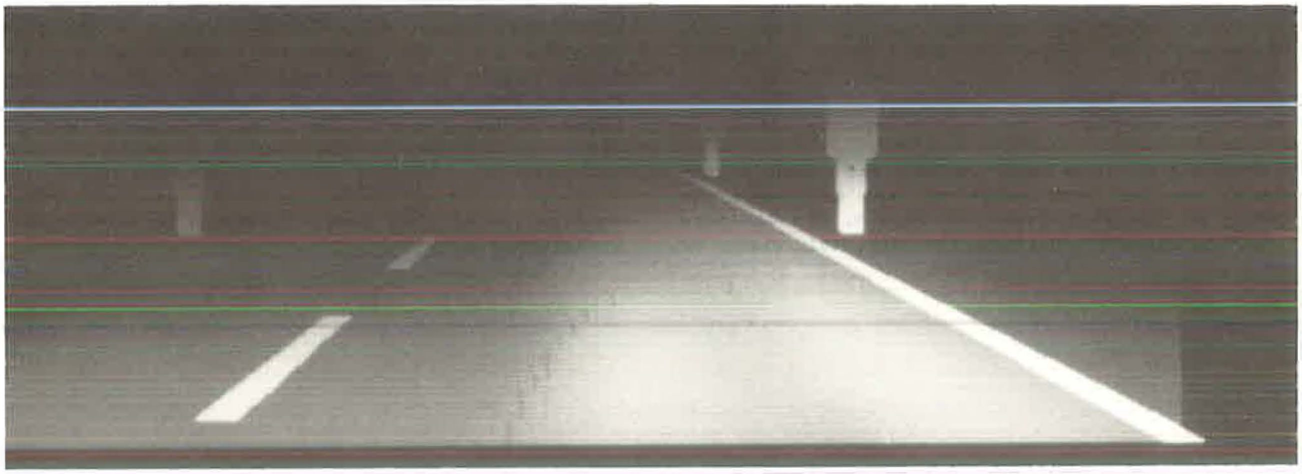

FIGURE 1 Computer generated night roadway scene.

of the headlighting system evaluation model was presented to the Society of Automotive Engineers (SAE) in 1977 (1). This model has since become known as the Comprehensive Headlamp Environment Systems Simulation (CHESS). CHESS is a computer program that evaluates a headlamp system for its ability to meet the visual needs of night drivers. The model was developed to provide a broader and more comprehensive assessment of headlamp systems than is possible in seeing distance field tests. The computer-generated pictures of night driving scenes that are the subject of this paper represent an attempt to further broaden the basis for evaluating automotive headlamps.

The CHESS program accepts as its basic input the candlepower distributions of each of the lamps in the system to be evaluated, and conducts a large number of simulated seeing distance and discomfort glare tests under many different preprogrammed road and traffic conditions which, in aggregate, constitute a Standardized Test Route. The Figure-of-Merit output by the model is the percentage of the distance traveled on this simulated test route in which the seeing distance to pedestrians and pavement lines and the discomfort glare experienced by opposing drivers meet certain acceptance criteria.

The Standardized Test Route simulated by CHESS is a representation of a series of highway sections in the form of a file of environmental parameters that have an influence on visual performance in night driving. The simulation includes such parameters as highway type, road geometry (hills and curves), lane configuration, ambient illumination and glare from fixed sources, traffic and pedestrian density, and the reflectance characteristics of the road surface, pedestrians, and lane lines. The data for the test route were obtained from extensive field measurement programs, surveys, and the open literature. The route is certainly typical, if not actually representative, of U.S. night driving conditions.

The seeing distance calculations in CHESS are performed by an integral seeing distance model that is based on the human visual performance literature and validated in field studies (2). Computation of glare effects is based on published discomfort glare formulations, modified and validated on the basis of highway tests (3).

A great advantage of the CHESS model is that it permits objective evaluation of beam patterns and headlamp systems that are still on the drawing board. Thus, alternative beam patterns can be evaluated and compared to each other and to existing systems before any hardware prototypes are built. CHESS can also reduce the need for costly and time-consuming field tests of hardware prototypes. The CHESS model and background research are described in detail by Bhere et al. (1).

Applications of the CHESS model have indicated that overall, driver visual performance as computed by CHESS does not vary a great deal from one lamp system to another. Table 1 gives Figure-of-Merit scores and the percentage of opposing drivers likely to be discomforted by glare for each of five low beams. These are (a) a U.S. type 4000 tungsten lamp, (b) a U.S. type H4656 halogen lamp, (c) a conceptual drawing board beam pattern that is a modification of the H4656 beam pattern, (d) a hardware prototype lamp designed to produce the beam pattern of the modified H4656, and (e) a European halogen lamp. The Figure-of-Merit scores range from 62.1 for the European lamp to 66.8 for the drawing board H4656. A difference of about two points is statistically reliable.

The Figure-of-Merit is more sensitive to environmental conditions and to the driver's visual capabilities than to the range of characteristics of current and proposed headlamp systems (1). This is partly because large increases in candlepower are required to provide useful increases in visibility, and because such increases in candlepower produce concomitant increases in glare. The need to trade off candlepower against glare to produce a subjectively acceptable beam pattern and to meet government standards has produced a considerable degree of uniformity among U.S. headlamps.

Objective and Subjective Factors

Nevertheless, there are individuals who have strong preferences for one headlamp system or another. In-

TABLE 1 Results of CHESS Model Applications

Lamp System	Figure-of-Merit	Percentage of Opposing Drivers Likely to be Discomforted
U.S. tungsten 4000	66.5	8.7
U.S. halogen H4656	64.6	8.1
Modified H4656 (drawing-board prototype)	68.2	11.3
European halogen	62.1	4.9
U.S. halogen prototype (hardware prototype)	66.3	13.6

dustry people who are familiar with the CHESS model are sometimes disturbed by the disagreement between their subjective assessment of a headlamp and the CHESS output. These disagreements arise because people and CHESS emphasize different aspects of beam performance. An individual might be quite comfortable with a system that combines low glare with good, even foreground illumination but is a poor pedestrian detector. Unless a driver actually hits or nearly misses a pedestrian, he might never become aware of the difficulty. The CHESS model would know, however, and would give bad marks for poor down-the-road visibility. On the other hand, CHESS does not know or care about near foreground illumination because it has not been so programmed. The reason for this is that no objective data exist to quantify beam pattern aesthetics, the subtle effects of a beam pattern on a driver's sense of ease and comfort.

Simply summarized, CHESS evaluations are based only on quantitative driver visual performance measures. An additional capability was desired that would provide the means for previewing the appearance and subjective qualities of proposed beam patterns and also provide a pictorial supplement to the CHESS output that would actually show seeing distance and glare effects. Accordingly, a system was developed for using high resolution computer graphics to generate drivers' eye views of headlamp beam patterns. The remainder of this paper is devoted to a description of this methodology and to a discussion of preliminary results.

The IVIEW System

The IVIEW program generates a driver's eye view of a night road scene on an Advanced Electronic Devices high resolution color terminal. The color terminal is driven by a CDC Cyber 176 computer, using a specialized package of routines that is accessible from FORTRAN. The image on the screen can be photographed to make 8 x 10 Polaroid stills or 35 mm slides. The program itself makes use of many of the analytical routines developed for the CHESS model.

Projecting Candlepower

The IVIEW program begins with the candlepower distribution of the headlamp to be evaluated. Figure 2

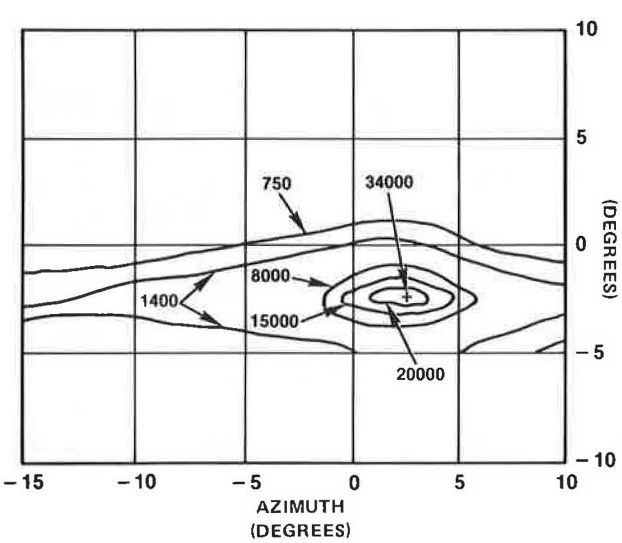

FIGURE 2 Isocandela diagram.

shows a headlamp isocandela diagram. The horizontal and vertical axes are in degrees of azimuth and elevation with respect to the optical centerline of the lamp, represented by the point where the axes cross. The contours are lines of constant candlepower. Figure 3 shows the isocandela diagram superimposed on a road scene. This is the view of the scene from under the hood, looking out from behind the headlamp. The candlepower projected at a point in the scene can be read directly from the diagram. Of course, this diagram shows only selected candlepower contours; in practice, a dense grid of values is used to permit accurate interpolation. Thus, once the vertical and horizontal angles of some point have been specified, the corresponding candlepower can be read off or interpolated from a table.

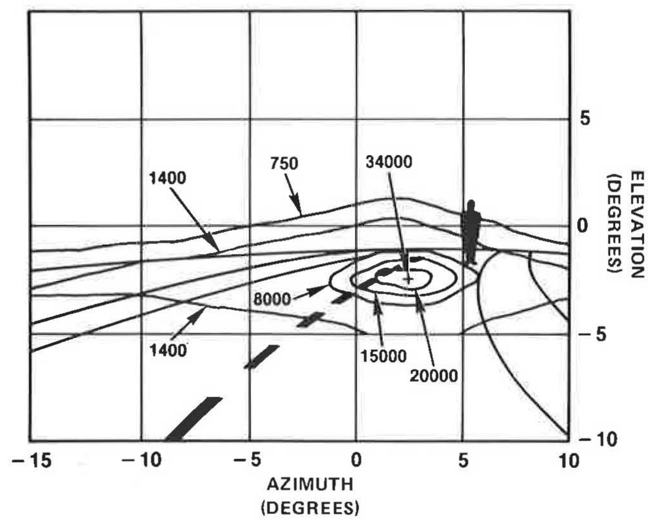

FIGURE 3 Projecting candlepower.

The Driver's Perspective

The view in Figure 3 is from the perspective of the headlamp. The object is to show what the driver sees, as in Figure 1. The axes of Figure 1 are also in degrees of azimuth and elevation, but here the perspective is that of the driver's eye. The location of any point in the driver's visual field can be expressed in terms of the horizontal and vertical displacement and in degrees from the optical axis, a line straight out from the driver's eye. The zero-zero point--the projection of the optical axis--is the vanishing point, the center of the horizon line. The extent of the field in Figure 1 is 12 degrees left and right and from 2 degrees up to 6 degrees down.

Relative brightness is plotted in this field. And although the brightness values in the picture are the result of many transformations from the original scene, depending on the reproduction medium, at a minimum, ordinal relationships are preserved. As noted later with slides, it is possible to reproduce exact scene brightness levels over a range of perhaps 100 to 1.

Mapping Brightness

How are the brightness values obtained? The procedure involves the straightforward application of trigonometry and analytic geometry. Given the azimuth and elevation of a point in the driver's visual field, the eye height above the ground, and the location of the headlamps relative to the eye, the azimuth and elevation of that point relative to the optical axis of a headlamp are easily determined. The geometry is illustrated in Figure 4. After the angular coordinates for the point have been determined with respect to the lamp axis, the candlepower impinging on that point can be read off the isocandela diagram, as described earlier. The distance from the headlamp to the point is also obtainable analytically and is used to calculate the illumination according to the distance squared formula,

$$E = I/D^2 \tag{1}$$

where E is illumination, I is the luminous intensity (candlepower), and D is the distance from the lamp to the point. This procedure is repeated for each headlamp in the system, and the illumination values so obtained are summed to determine the total illumination of the point. Then the illuminance is given by

$$L = RE \tag{2}$$

where R is the reflectance of the point of interest. Specifying the visual scene is thus largely a matter of defining a reflectance map, that is, specifying the shape, location, and reflectance of the objects to be included in the picture.

Scanning the Field

The picture-building process begins with the road surface. The width of the roadway, its reflectance, and the reflectance of the shoulder are specified as part of the descriptive data made available to the IVIEW program. This effectively divides the scene into four areas: the sky, the road surface, and the areas to either side of the road surface that are treated as extended shoulders. The portion of the driver's visual field below the horizon is scanned in quarter-degree steps. At each step, the geometric and photometric calculations described previously are performed to determine the brightness of that point. This process shows the pattern of brightness on the surface produced by the lamps. If the reflectance of the road surface and the shoulder are different, the scene will appear as in Figure 5. In this particular picture, the reflectance of the surface was set at 4 percent and the reflectance of the shoulder was set at 2 percent.

Adding Pedestrians and Delineation

The method for adding objects to the field is somewhat different. There are separate subroutines for superimposing lane lines, pedestrians, and other classes of objects on the scene. For lane lines,

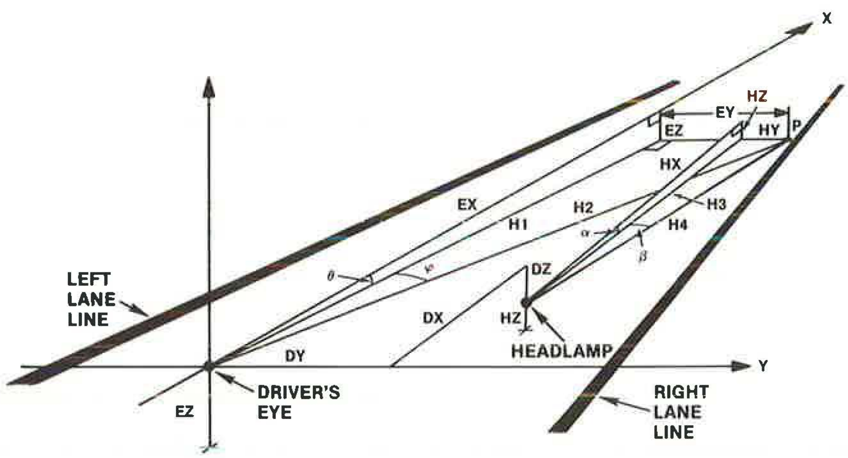

FIGURE 4 Geometric relationship between the driver's eye, the headlamp, and a road surface point, P.

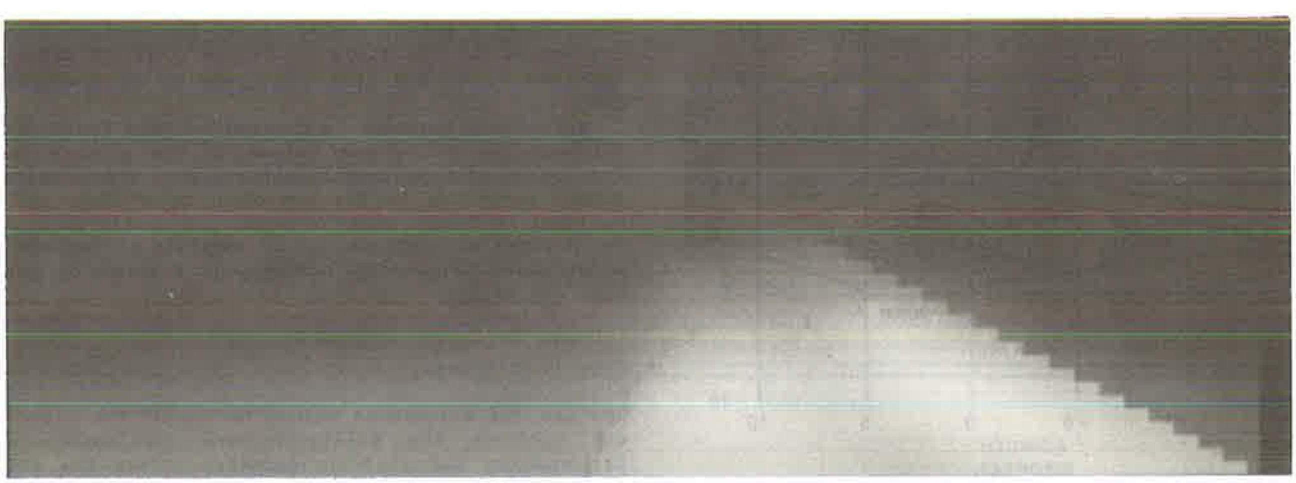

FIGURE 5 Headlamp beam pattern projected onto the roadway and shoulder.

only the width, reflectance, and centerline gap length need be specified. The program automatically locates them laterally at the lane edges and road center. These areas are then subjected to a quarter-degree scan, using the geometric and photometric calculations described previously to determine the brightness of the corner points of quarter-degree long segments of the lines. Linear interpolation is used to perform a fine-grained (1/20 degree) fill of each segment. In Figure 1 the lines are 6 in. wide and their reflectance is 12 percent.

Pedestrian images are produced in much the same way. Each image is made up of a set of small quadrilaterals. The brightness of the corner points of each of these is determined, and linear interpolation is used to perform a fine-grained fill. In Figure 1 the pedestrians on either side of the road have 25 percent reflectance and are located at 100, 200, and 300 ft down the road from the headlights.

Finally, there is a utility routine that can be used to generate general shapes. This routine takes any convex polygon and breaks it into quadrilaterals. [Note that in a convex polygon, straight lines connecting any pair of vertices lie wholly within the polygon.] Given the reflectance of the objects, the brightness of each corner point can be determined from the candlepower tables and photometric calculations as described previously. The brightness values of points inside the quadrilateral are determined by linear interpolation, and a fine-grained fill is performed. This procedure can be used to produce a picture of any object that can be conceived as a polygon or set of polygons.

Figure 6 shows an additional capability of the IVIEW system: glare from an opposing vehicle's headlight. Veiling glare is depicted, the curtain of haze produced in the eye by the scattering light from a glare source. It does not represent the bloom of light often observed around light sources in a hazy atmosphere. At each point in the picture, the amount of haze is correct for an observer looking at that point in the field, that is, for that particular angle between the glare source and the fixation point. Fry's formulation is used to calculate this effect (4). Note that the haze is greatest at the glare source and falls off rapidly as this angle increases.

Illumination from the opposing lamps also contributes to the brightness of the road surface and the lane lines. The opposing lamps clearly illuminate the left edge delineation, which, in the unopposed case, is barely visible. The opposing lamps also illuminate the left shoulder and road edge, backlighting the pedestrians on that side. The second pedestrian on the left side stands in a pool of light produced by specular reflectance components off the pavement from the opposing lamps so that the lower legs are blackly silhouetted in negative contrast. The backlighting effects produced by the lamps from an opposing vehicle are obviously important in the overall performance of a beam pattern and need to be taken into account to obtain a balanced appraisal of a lamp system.

COMPARING LAMPS

With this background, the pictures can now be examined and compared to the beam patterns of the lamp systems given in Table 1.

U.S. and European Beam Patterns

Figure 7 shows a representation of the illumination produced by a typical 5.75-in. tungsten 4000 low beam from a four-lamp system. The bottom picture shows the scene without glare, and the top picture shows the same scene but with glare from an identical system 320 ft away. The beam pattern in this lamp is sharply tuned; that is, much of the light is in the hotspot, aimed down and to the right. Compare this to Figure 8, which shows a current European halogen lamp with a more broadly distributed beam.

Actually, the European lamp is quite intense along the right shoulder, and the seeing distance to pedestrians on the right is about 10 percent longer. The difference is discernible in the picture. The more distant right side pedestrians are somewhat more visible in Figure 8. On the other hand, despite the more uniform appearance of the European beam pattern, the tungsten 4000 produces more illumination along the left side of the road than does the European lamp. Comparing Figures 7 and 8 shows the left side pedestrians to be somewhat brighter in Figure 7 (the tungsten 4000 lamp). In fact, Figure 8 (the European lamp) shows the tops of the left side pedestrians to be visible in negative contrast against the sky. European beam patterns vary from lamp type to lamp type, and the system shown here is not necessarily typical.

Opposing Lamps

The glare is clearly more intense in the tungsten 4000 (Figure 7); nevertheless, with both systems,

FIGURE 6 Representing veiling glare from opposing headlamps.

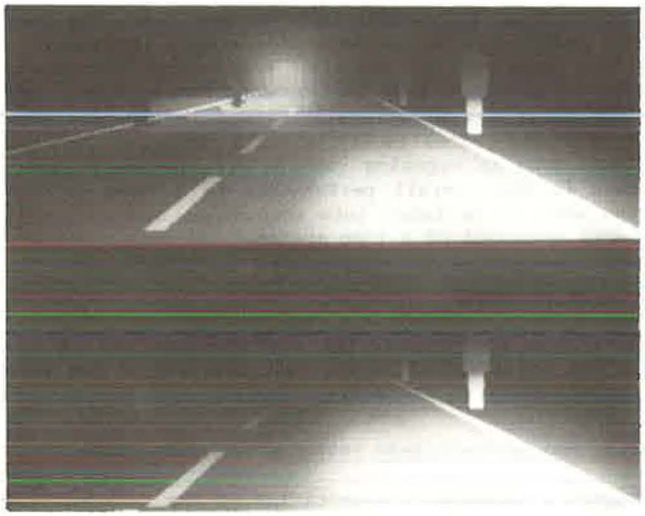

FIGURE 7 U.S. tungsten 4000 low beams with and without glare from identical opposing lamps.

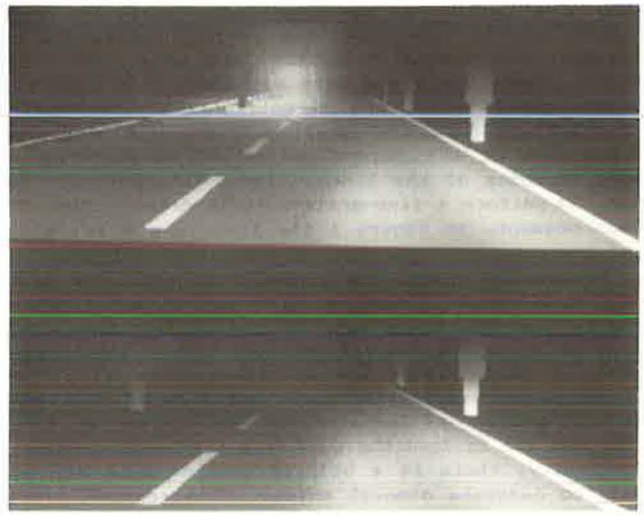

FIGURE 9 U.S. H4656 halogen low beams.

glare from the opposing vehicle wipes out the third pedestrian on the left and makes the distant right side pedestrian somewhat less visible. The European lamp (Figure 8) is more intense along the right shoulder, with the result that the two closer left side pedestrians are more sharply silhouetted against the backlit road surface and shoulder.

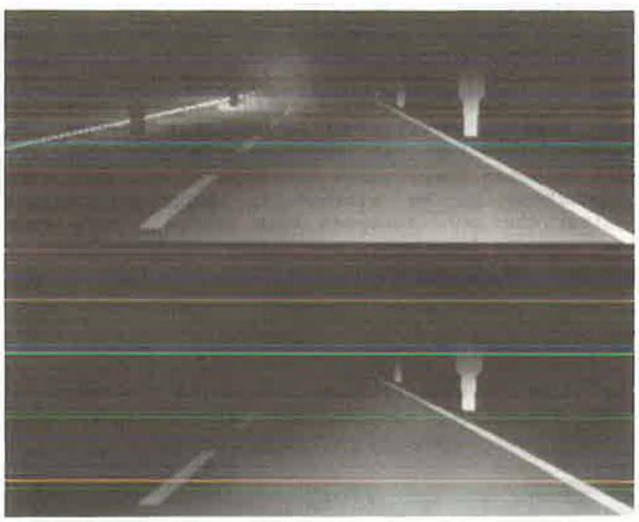

FIGURE 8 European halogen low beams.

U.S. Halogen Lamps

Figure 9 shows the projection of the beam pattern from a rectangular type H4656, 35-watt U.S. halogen lamp. Although there is a distinct hotspot in the H4656 beam, it is less intense than the tungsten 4000 (Figure 7), and the beam pattern is more spread out. In comparison with the European halogen lamp (Figure 8), the type H4656 beam concentrates more candlepower in the left foreground. Comparing Figures 7, 8, and 9 shows that the left side delineation and nearer pedestrians are slightly more visible with the H4656.

Candlepower tables for existing lamps are obtained by photometering actual samples. However, it is a simple matter to alter the resulting table or to make up a completely different conceptual beam pattern for research purposes. Figure 10 was produced by such a drawing board beam pattern. This beam pattern is a modification of the H4656 pattern. An effort was made to add more down-the-road light and a more even foreground spread without significantly worsening the glare. The modified beam pattern is noticeably brighter across the foreground. Also, careful comparison of the two pictures shows that left and right side delineation and right side pedestrians are slightly more visible with the modified version. Visibility of left side pedestrians is about the same. This is consistent with the results of seeing distance calculations that show the modified beam to give 12 to 14 percent more seeing distance than a typical H4656 lamp. The modified beam also has a somewhat higher CHESS Figure-of-Merit (overall performance score), although it does produce slightly more glare.

Figure 11 shows a beam pattern photometered from a prototype lamp that was designed to produce the modified drawing board version of the type H4656

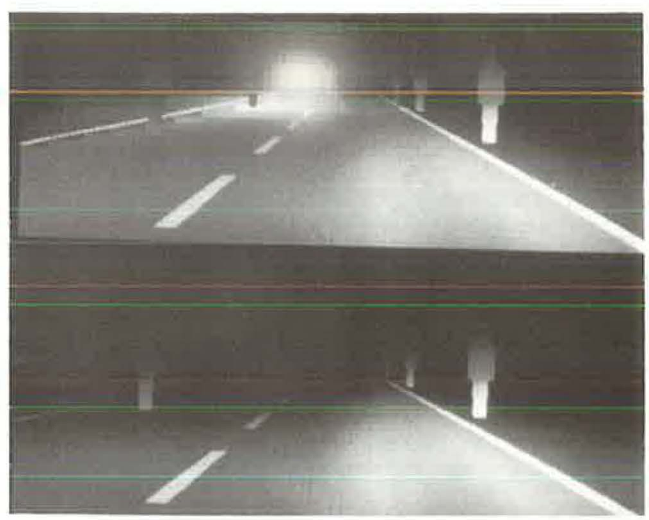

FIGURE 10 Modified U.S. halogen low beams—conceptual prototype.

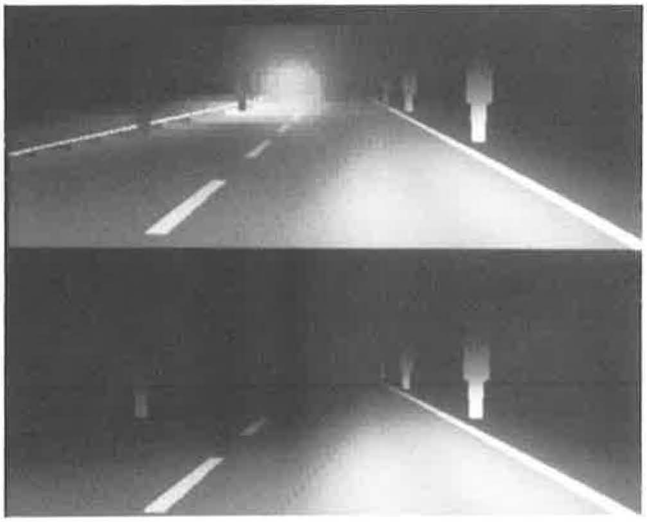

FIGURE 11 Modified U.S. halogen low beams—hardware prototype.

beam shown in Figure 10. The beam patterns appear to be quite similar, but the prototype is actually a little brighter. It gives a slightly longer seeing distance than the drawing board beam pattern but also produces considerably more glare. The differences are probably because the actual beam pattern is slightly less controlled than the conceptual one, that is, in the real lamp, not all the light goes where it is supposed to go. It is easier to draw pictures of beam patterns than it is to realize them in hardware.

Such are the complexities of comparing beam patterns. Which is better? Which would you rather have on your car? Which would you rather encounter? Look again at Figures 7 and 8, the U.S. tungsten 4000 and the European halogen beam patterns. Each has advantages and disadvantages that can be expressed in terms of objective and quantifiable measures of driver visual performance. Subjectively, the European lamp produces low glare levels and a uniform spread of light that is pleasing and comfortable. Nevertheless, the CHESS model indicates that the tungsten 4000 lamp has a significant edge in overall performance (see Table 1). That is because in the world represented by the CHESS Standardized Test Route, the situations in which the tungsten 4000 outperforms the European beam pattern occur more frequently than situations in which the reverse is true.

CONCLUSIONS

The pictures clearly demonstrate the trade-offs that are inherent in any workable beam pattern. They also show in what aspects of beam pattern design one system might be performing better or worse than another. This information should provide useful guidance for refining an evolving prototype. However, using the system to determine which of several competing beam patterns is best overall could be misleading. This is because different lamps excel under different driving conditions, and a given picture can represent only one of the wide range of conditions routinely encountered in night driving. Nor can the pictures convey the actual physical discomfort produced by glare from opposing lamps. Discomfort glare is an important consideration because it is the major constraint on low beam intensity.

The driver's eye view pictures accompanying this paper represent only a fraction of the 10,000 to 1 brightness range of the actual night driving scene. Good quality photographic paper can reproduce a brightness range of from 25 to 1 or 40 to 1. With projected slides, the range is about 100 to 1. Even within this range, the nonlinearities in the imaging and photographic processes guarantee that the scene and image brightness values will not correspond 1 to 1. It is feasible, however, to incorporate a calibration curve in the IVIEW program to control the nonlinearity and produce a slide that does agree closely in brightness values with the real scene over a 100 to 1 range.

Also, in principle, two slides of the same scene could be overlaid and carefully registered to produce the full 10,000 to 1 range-of-brightness values. This is possible because the composite density at any point is the product of the individual densities. Other refinements to the IVIEW system are possible, including the incorporation of many of the operational factors that characterize the Standardized Test Route, such as curves and hills, wet pavements, fixed lighting, degraded lane delineation, and multilane roadways. These refinements would make it possible to visually preview and compare headlamp systems on any section of the Standardized Test Route to determine directly why one or the other is better under a given set of conditions.

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