TRANSPORTATION RESEARCH RECORD 703

Work Zone Traffic Control

TRANSPORTATION RESEARCH BOARD

COMMISSION ON SOCIOTECHNICAL SYSTEMS
NATIONAL RESEARCH COUNCIL

NATIONAL ACADEMY OF SCIENCES
WASHINGTON, D.C. 1979

Transportation Research Record 703
Price $3.00
Edited for TRB by Susan Singer-Bart

mode
1 highway transportation

subject areas
40 maintenance
52 human factors
53 vehicle characteristics
54 operations and traffic control

Transportation Research Board publications are available by ordering directly from TRB. They may also be obtained on a regular basis through organizational or individual affiliation with TRB; affiliates or library subscribers are eligible for substantial discounts. For further information, write to the Transportation Research Board, National Academy of Sciences, 2101 Constitution Avenue, N.W., Washington, DC 20418.

Notice
The papers in this Record have been reviewed by and accepted for publication by knowledgeable persons other than the authors according to procedures approved by a Report Review Committee consisting of members of the National Academy of Sciences, the National Academy of Engineering, and the Institute of Medicine.

The views expressed in these papers are those of the authors and do not necessarily reflect those of the sponsoring committee, the Transportation Research Board, the National Academy of Sciences, or the sponsors of TRB activities.

Library of Congress Cataloging in Publication Data
National Research Council. Transportation Research Board.
Work zone traffic control.

(Transportation research record; 703)
1. Traffic engineering–Addresses, essays, lectures. 2. Roads–Maintenance and repair–Addresses, essays, lectures. I. Title. II. Series.
TE7.H5 no. 703 [HE333] 380.5'08s [388.3'12]
ISBN 0-309-02849-3 ISSN 0361-1981 79-19285

Sponsorship of the Papers in This Transportation Research Record

GROUP 3—OPERATION AND MAINTENANCE OF TRANSPORTATION FACILITIES
Adolf D. May, University of California, Berkeley, chairman

Committee on Traffic Safety in Maintenance and Construction Operations
Kenneth A. Brewer, Iowa State University, chairman
Roy W. Anderson, R. Clarke Bennett, William J. Buglass, Pierce E. Cody III, Albert L. Godfrey, Sr., Jerry L. Graham, Jack B. Humphreys, Frank N. Lisle, Charles R. Miller, Larry T. Perkins, Roger G. Petzold, Stephen H. Richards, Ben C. Russell, Lynne Smith, Kenneth C. Townley, William L. Williams

Adrian G. Clary, Transportation Research Board staff

The organizational units and officers and members are as of December 31, 1978.

Contents

Human Factors Considerations in Arrow-Board Design and Operation

Beverly G. Knapp and Richard F. Pain, BioTechnology, Inc., Falls Church, Virginia

This paper addresses three questions: (a) Does the flashing arrow board have more than one inherent meaning to the driver, according to the display configuration? (b) Can certain design characteristics of arrow boards be optimized to convey the desired message? and (c) Will certain operational characteristics of arrow boards optimize the communication of the display message? A threefold approach was taken: (a) a review of pertinent literature, (b) performance of some limited field studies, and (c) application of human factors expertise and judgments. The two experimental studies attempted to discern meanings associated with arrow-board configurations. Subject responses to film clips of arrow-board operations were gathered. The results indicate that the arrow board is strongly associated with lane closure and that the use of an on-off blinking arrow is favored over the sequencing chevrons or sequencing arrow stem followed by the stem plus head. Arrow-board design and operation can be manipulated and optimized as long as drivers can perceive a discrete, clear directional arrow as an indication of lane closure. Violation of this for other traffic management purposes leaves drivers uncertain as to exactly what their behavior should be. The following key recommendations are made: (a) the preferred operation of the arrow board is in the single on-off blinking arrow mode, (b) the blinking arrow should not be used as a cautionary display only, (c) 360° lens hoods should be used to cap dispersing light to passing drivers and to direct the flashing lights outward in a straight line perpendicular to the arrow board, (d) dimming of luminance could be upgraded to be more sensitive to inclement weather conditions and to begin dimming with lesser diminution of daylight, and (e) arrow boards should be placed at the beginning of the taper (construction zone).

This paper addresses human factors considerations regarding the design, use, and operation of flashing arrow boards. These traffic-control devices typically are used either alone or in conjunction with other devices to alert and guide the driver safely through a hazardous highway construction or maintenance zone. The arrow signal is used to attract attention to an aberrant situation in the roadway ahead that is a violation of the driver's expectation.

The literature (1) and our field observation document that arrow boards are warning signals that have very high target value. They present a visible image capable of being detected from distances of over 1.6 km (1 mile) (2). Various state highway traffic manuals and state-conducted studies (3-5) recommend the use of arrow boards for driver management through hazardous zones. Closer scrutiny of the types of arrow boards available (6-9) and their actual use on the roadway reveals a marked diversity in displays and messages communicated to the driver. For example, an arrow board may be used to close one or more lanes of traffic, to protect equipment and workers on a highway shoulder, or simply to reinforce a driver's position to the right or left of a given lane line or barrier. In turn, this arrow board may flash a single on-off arrow, a series of sequencing chevrons, a sequential arrow stem followed by the stem with the arrow head, or simply an array of lights in a square or bar configuration, which indicates no direction (see Figure 1). The question then arises because of varying arrow configurations available for use and various placements of arrow boards (on the shoulder or in the lane), How effective are these devices in conveying their intended meanings to the driver? Does the arrow board inherently connote lane closure, lane diversion from a given path but requiring no merge, or just a caution to be cognizant of shoulder work? Does the chevron or the on-off arrow convey the desired meaning more effectively or are they interchangeable?

Since the use of arrow boards is becoming more widespread, there seems to be a particular need to specify and examine the different arrow configurations and the situations in which they are used to determine how effectively they communicate a given message to the driver.

DRIVER UNDERSTANDING AND EXPECTANCY OF ARROW BOARDS

The arrow board is a specific example of a high-target-value advance warning device used to alert individuals to a hazard zone ahead. Much has been documented about its use in hazard zones (3, 5, 10, 11). These documents and others consist of state highway department research field studies and construction zone field manuals that encourage and advocate the use of arrow boards to divert traffic around a hazardous zone. The salient point of these studies is that the arrow board is a powerful advance warning device, easily detectable from distances of 1.6-4.8 km (1-3 miles) away.

The high target value of these devices is in direct accordance with human factors principles, which dictate that an individual must be given a strong, clear signal in order to elicit his or her proper response (12, 13). Although applied human factors and perceptual psychological literature do not address arrow boards as such, the use of such signals is analogous to presenting to the individual any strong target signal in the midst of visual noise so that he or she may recognize an aberrant, hazardous, and novel situation and respond to it properly (14-18).

Essentially, then, highway studies and documents promote the arrow board as an effective device for diversion of traffic around work zones, and theoretical human factors literature supports the arrow board as an effective example of an easily detectable target in an advance warning system. Neither source, however, focuses on exactly what a given arrow configuration tells the driver to do when he or she detects this strong signal.

In Figure 1 a process of deduction is required to determine, from current literature, what each configuration may be telling the driver to do. Although various field studies have shown that the arrow board seems to tell drivers "get out of your lane—merge ahead" (1, 4, 5, 19, 20), only passing reference to the kind of arrow board most useful in doing this is found. This is most notable in the California study (5), where the arrow board caused drivers to merge even though, in certain conditions, the device was not actually placed in a lane (i.e., it was placed on the shoulder). No inherent superiority of flashing arrows over sequencing arrows or sequencing chevrons was truly shown, except for a slight but significant degradation of the chevrons during nighttime operation.

Considerable emphasis has been placed on target potency, and this is well established. However, once a driver sees the arrow board, his or her subsequent behavior depends on what meaning is attached to the arrow. It is particularly important, then, to determine whether a meaning is attached to the arrow and which configuration conveys a unitary message to the greatest number

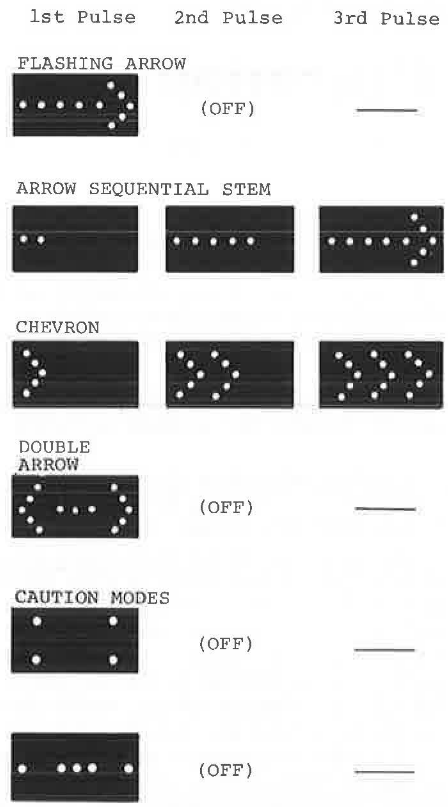


Figure 1. Various flashing arrow-board modes.

of drivers. To this end, two small-scale studies were performed.

Study Number 1

The first study attempted to ferret out the actual meaning of the arrow-board configuration and arrow-board placement (i.e., on the shoulder or in the lane). Nine short film clips were made from the driver's view as he or she approached an arrow board on the same stretch of roadway. Each of the nine clips represented a different mode of the arrow board in combination with placement either in the lane or on the shoulder. Figure 2 shows the nine conditions along the abscissa of the summary graph. Each condition was presented twice, in random order. The 20 respondents consisted of 9 females and 11 males, who ranged in age from 18 to 50 years (mean 29.7) and had a mean driving experience of 13.8 years. After each film clip was shown, subjects were required to select one of four responses, as shown in Figure 2. In addition, each subject was to indicate how confident he or she was in this response on a scale of 1 to 5. Essentially, three hypotheses were tested:

1. There is no difference in accuracy and confidence in interpretation of different arrow configurations;
2. There is no difference in the meaning associated with blinking arrows, sequencing arrows, chevrons, and blinking lights; and
3. There is no difference in the meaning between arrows placed in a lane and those on the shoulder.

Figure 2 is a summary of the results obtained. About 95 percent of the subjects were confident that the arrows and chevrons connoted a lane closure ahead. Mere blinking lights stirred more confusion than they aroused meaning. Arrows and chevrons seem to indicate a lane closure for roughly 75 percent of the subjects, even when the arrow board was in fact placed on the shoulder and a merge was not actually required. This is a reinforcement of the California findings (5). Here is empirical evidence that drivers mainly understand that the flashing arrow means a lane closure ahead. Unanswered questions and problems remain, however.

1. Simple inspection shows no clear superiority of arrows over chevrons (or vice versa),
2. Respondents do not seem to be able to recognize when the lane is open or closed by virtue of arrow-board placement, and
3. The role of the caution or blinking lights needs in-depth examination in terms of its usefulness, in view of the apparent confusion that surfaced.

The first consideration, a rank order of effectiveness among blinking arrows, sequencing arrows, and sequencing chevrons in effecting the lane closure, spurred us to perform a second study, which used a forced-choice technique to single out a superior arrow configuration.

The second and third considerations, shoulder placement and caution mode presentation, dictate further refinement and replication of the study to make definitive conclusions. This was not within the scope or the resources of this project. However, the shortcomings of this study should be pointed out so that future efforts can attempt to clarify these points.

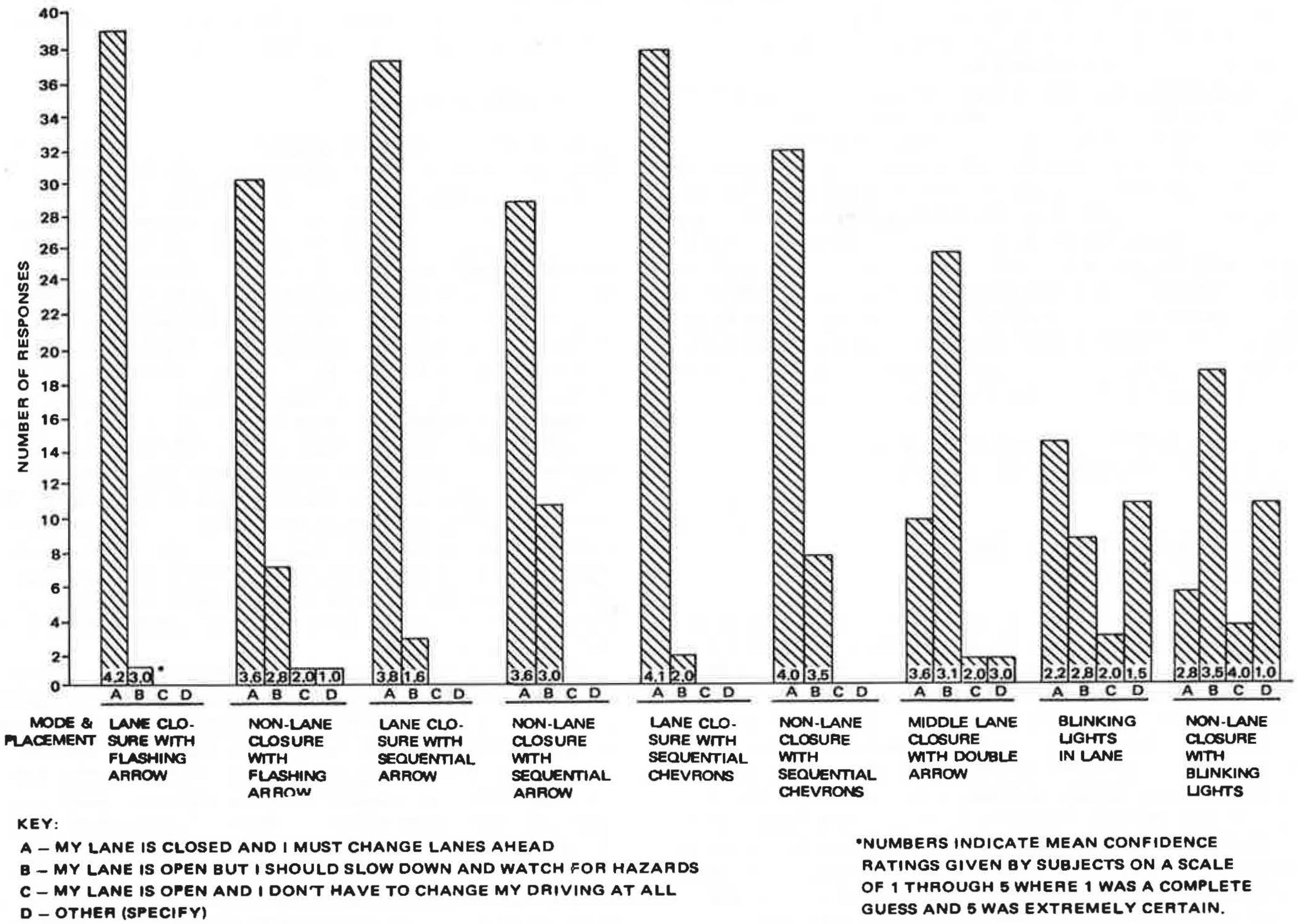
Since the film efforts, sample size, and composition were limited and unrefined, data analysis did not proceed beyond qualitative inspection. Given the opportunity to repeat this study, the first refinement would be a much improved series of film clips, which would clearly show the difference between shoulder placement and lane placement. The respondents who viewed the film clips expressed much confusion on this matter. The second refinement would be to obtain a larger, more representative sample of the driving population, perhaps as many as 100 respondents. In this way, some statistical stability in responses would be gained, and some analysis could be made of the demographic variables of age, sex, and driving experience. Finally, the confusion about the caution mode-blinking lights configuration suggests that this display ought to be evaluated as a separate entity to determine optimum caution configurations and whether, in fact, the arrow board is appropriate for this message at all. The need for experimental investigation into this matter is apparent.

Study Number 2

This study addressed the question of whether the three arrow configurations (blinking arrow, sequencing arrow, and sequencing chevrons) relay essentially interchangeable messages in directing the driver to vacate a lane, or whether one mode is clearly superior and more effective in conveying this meaning. Six short film clips were prepared to present two modes simultaneously, side by side, so respondents could choose one in a paired-comparison experimental model. Given three modes, six pairs for comparison required evaluation (22):

Trial	Left	Right
1	Sequential arrow	Blinking arrow
2	Sequential arrow	Sequential chevron
3	Blinking arrow	Sequential arrow
4	Blinking arrow	Sequential chevron
5	Sequential chevron	Sequential arrow
6	Sequential chevron	Blinking arrow

Figure 2. Summary of results, analytical study no. 1.



Consequently, two arrow boards, side by side, were filmed flashing the above pairs. The six short clips were then shown to a subject sample of 109 drivers at the Midwest Research Institute and the Federal Highway Administration (FHWA). The respondent's task was simply to indicate, by checking either left or right, which mode of the two presented in each film clip best conveyed the meaning of lane closure. A summary table of results in Table 1 shows the proportion that selected each mode for each of the six cells.

It was judged appropriate to go no further in data analysis than simple inspection since this was a preliminary study and the sample was not representative of the driving public. As can be seen, the blinking arrow and the sequential chevrons clearly outdistance the sequential arrow. However, the blinking arrow and the sequential chevrons do not separate out significantly between themselves, which indicates that these might be interchangeable in their use. These data, added to evidence available from the literature, suggest some reasons to advocate the blinking arrow over the chevrons: (a) the California study (5) showed the superiority of a blinking mode at night; and (b) human factors design principles suggest some target-value advantage for a single on-off flashing operation rather than a multiflashing array (12, 13, 22).

Summary

It is apparent that the flashing arrow board, operating in a directional mode, connotes lane closure ahead. Other uses of the board, such as cautionary devices on a shoulder or flashing a cautionary display, are confusing to the driver, interfere with other necessary control functions, and may even cause him or her to initiate and negotiate a lane merge when one is not necessary (5). The presence of an arrow board that does not serve as part of a lane closure operation is a particularly hazardous situa-

Table 1. Selection of arrow-board display modes in six-paired comparison trials.

Trial Number	Left	Proportion Selecting (%)	Right	Proportion Selecting (%)
1	Sequential arrow	23	Blinking arrow	77
2	Sequential arrow	30	Sequential chevron	70
3	Blinking arrow	63	Sequential arrow	37
4	Blinking arrow	55	Sequential chevron	45
5	Sequential chevron	67	Sequential arrow	33
6	Sequential chevron	41	Blinking arrow	59

tion. The use of arrow boards for other than lane closures (e.g., diversion or detour) should be addressed by further research. It seems most efficacious to restrict its use to lane closure to conform with the meaning drivers now associate with it.

In addition, the arrow-board display seems to convey its message best when it operates in a single on-off blinking-arrow mode. Although the sequential chevron provides a strong directional indication to the driver, it must go through three pulses to convey its message as opposed to two pulses for the blinking arrow. The meaning of the three pulses has a greater tendency to be degraded if displayed at night, blocked by large trucks, or when diffused under adverse weather conditions (because of the tremendous amount of light given out on the final pulse) (2). Further research into these questions with a larger, more representative sample of drivers under various ambient light conditions is recommended.

HUMAN FACTORS CONSIDERATIONS RELATED TO DESIGN OF ARROW BOARDS

Performance Criteria Versus Engineering Criteria

The crucial issue, from a human factors point of view, is simply to ensure that the arrow board displays its directional image well in advance of the hazardous lane closure ahead, so that the driver is able to safely and effectively merge his or her vehicle into a parallel lane. The actual engineering details of how the board is made to operate in this fashion are secondary to its effectiveness in conveying this warning message to the driver. The human factors research does not, therefore, suggest quantitative absolute dimensions for building arrow boards. What it does is dictate how arrow boards must perform so that engineers can build them to meet these performance criteria as technologically and cost-effectively as possible.

The basic concept behind use of the arrow board is to warn of a hazard and to effect a proper behavioral response from the driver. Viewed in this context, then, the sighting of an arrow board is subject to principles developed in decision sight distance (DSD) research. DSD has been defined in concept as (23)

> The distance at which a driver can detect a signal (hazard) in an environment of visual noise or clutter, recognize it (or its threat potential), select appropriate speed and path, and perform the required action safely and efficiently.

As applied to arrow boards, these devices serve as warnings of the hazard ahead, and thus their signal must be detectable from recommended distances, which are derived from experimental research on DSD. One useful table of such design distances is found in a report by McGee, Moore, Knapp, and Sanders (24), which found that, at an operating speed of approximately 90 km/h (55 mph), DSD for the hazard should be approximately 305 m (1000 ft). A table of similar values for various design speeds is found in Table 2. In essence, the flashing signal must be detectable and clearly recognized by 99 percent of the drivers at an absolute minimum distance of 2.4 km (1.5 mile). To provide for high traffic densities, which limit safe gaps for merging, and occasional high-speed drivers, an optimum performance standard is as follows:

1. Presence of blinking lights detectable at 2.4 km,
2. Arrow symbol and direction of arrow recognizable at 1.6 km (1 mile),
3. Above conditions possible for 98 percent of the driving public, and
4. Above conditions possible for both day and night conditions, urban and rural freeways.

Design Characteristics

In light of the above recommendations of a DSD guideline, the literature demonstrates that flashing warning lights have a high attention value well in excess of that required for detection (2, 14). Many research studies in the applied psychological literature indicate the assets of flickering lights (25) and brightness contrast in this original detection task (26). In fact, Swezey (27) speaks of the crucial importance of brightness contrast of the target against its background: Flashing lights against a flat black produce maximum effectiveness. In this same vein, target size and luminance are addressed as a signal detection task by recent researchers (28, 29). Also, Benignus and others (30) demonstrated experimentally that steady-rate signals are superior in influencing and capturing the attention of subjects as the rate of on-off flashes increases. This is in agreement with the findings of Ruden and others (14). When these experimental findings are matched with current design specifications available on arrow boards now in use, no drastic design changes from a human factors point of view are necessary.

Consider a brief inventory of five typical arrow-board manufacturers' design specifications, as given in Table 3 (6-9, 34). The size of the boards available [all approximately 123×246 cm (48×96 in)], the lens size, and the flash rates (for warning devices only—no symbolic message included—flash rates in the 50-230/min range are optimum, but where an arrow must be recognized, slower rates of 40-50/min are optimum) are all reasonable for signal detection well in advance of the prescribed minimum sight distance of 305 m (1000 ft). Current arrow boards are more than adequate as detectable light sources for the optimum sight distances noted earlier. However, research does not describe arrow recognition distances. Our informal observations suggest that arrows are recognizable at approximately 1.6 km (1 mile) away, but further testing is recommended.

A note may be made here about manufacturers' visibility specifications. Some definition should be given of visibility: detection of the signal lights themselves or recognition of the arrow as a signal. The latter should be specified as the criterion, and a standard method should be established for testing conformance to the criterion.

Human factors considerations may be centered around some degradation of the board's capabilities, as a function of placement and sight distance available. Two points might be made. First, on a high-speed, controlled-access facility, where a lane closure is initiated by the flashing arrow, a sight distance of more than 1.6-3.2 km (1-2 miles) for the arrow board may actually constitute a hazard, since this sighting is usually not a recognition of the arrow image (2, 14). In this regard, a bigger, more powerfully flashing target, upgraded from those already available, might inspire a real hazard too far in advance of proper assimilation of the intended message. Second is a point related to the high-powered nature of the image displayed and questions arrow board use on freeways versus arterial locations. It seems intuitively obvious that in most urban arterial locations, other devices for channelizing and diverting traffic would be much more cost-effective than an arrow-board display in an already close-up, slow-moving corridor. These questions of excessive sight distance in

Table 2. Recommended DSD.

	Premaneuver					
Design Speed (km/h)	Detection and Recognition (s)	Decision and Response Initiation (s)	Maneuver—Lane Change (s)	Total (s)	DSD (m) Computer	DSD (m) Rounded for Design[a]
40	1.5-3.0	4.2-6.5	4.5	10.2-14	113-156	120-160
60	1.5-3.0	4.2-6.5	4.5	10.2-14	170-233	170-230
80	1.5-3.0	4.2-6.5	4.5	10.2-14	227-311	230-310
100	2.0-3.0	4.7-7.0	4.3	11.2-14.5	306-397	310-400
120	2.0-3.0	4.7-7.0	4.0	10.7-14	357-467	360-470
140	2.0-3.0	4.7-7.0	4.0	10.7-14	416-544	420-540

Note: 1 km/h = 0.62 mph; 1 m = 3.28 ft.

[a] Rounded up to the nearest 10 m for the low value and up or down to the nearest 10 m for the upper value.

Table 3. Inventory of typical arrow-board specifications.

Manufacturer/Model	Size of Board (cm)	Lens Size (cm)	Lens Type/Color	Lens Lamp[a] (cd)	Flash Rate per min	Duty Cycle, On/Off (%)	Visibility (km)	Contrast (color of back panel)
Casell Early Warner III	99 × 192	12.8	PAR 46 amber	8630	30-40	50	Arrow—4.8 Chevron—3.2	Flat black
Dietz arrowboard	123 × 246	12.8	PAR 46 amber	8630	30	50	N/A	Flat black
EMPCO-Lite #6075 "The Hydra"	123 × 246	12.8	PAR 46 amber	8630	30-55	50	N/A	Flat black
Protect-O-Flash Advance Warner	123 × 230	12.8	PAR 46 amber	8630	50	50	N/A	Flat black
Royal Signal System's Tri-Function	123 × 246	12.8	PAR 46 amber	8630	35-50	50	N/A	Flat black

Note: 1 cm = 0.39 in; 1 km = 0.62 mile; 1 cd = 1.02 candle power.

[a] All manufacturers' specifications state that automatic dimmers are available and that they commence dimming when ambient light drops below 4.9 cd.

arterial situations should be resolved by further research. No available literature documents this question.

Since typical arrow boards, such as those specified, generally display as much as approximately 129 400 cd (132 000 candlepower) at a flash rate of 30-55 flashes/min, this powerful image may need to be dimmed as ambient light conditions darken. Thus, the automatic dimming feature found in most boards is commended and advocated (see note to Table 3).

Summary

The current design specifications for arrow boards are more than adequate to meet display criteria. In fact, the displayed image may be overwhelming in some situations (i.e., urban arterials) and indiscernible as an arrow, if even detectable as flashing lights [i.e., >3.22 km (2 miles) sight distance]. To reiterate the salient point, the arrow board is in service to divert traffic from a lane to be closed ahead. Manipulation of lenses, board sizes, heights, lens spacings, and board mountings conceivably may be manipulated for optimization from a technical viewpoint.

However, the key is that the arrow be displayed clearly and distinctly, so that drivers recognize a need to perform a safe merge maneuver. This decision point will be dependent on the geometries of the lane closure involved and, therefore, an individual judgment in each situation, based on the performance criteria supplied above.

HUMAN FACTORS CONSIDERATIONS RELATED TO ARROW-BOARD OPERATION

Design and operations of any system are contingent on one another. For example, currently available and recommended flash rates for arrow boards are found in Table 3 and mentioned previously, but these are also items of arrow-board operation. The flashing lights themselves, in terms of luminance, intensity, and glare, are important to vehicle control as the driver encounters the flashing display. Other variables to be addressed in arrow-board operation pertain to placement, angularity, and ambient light conditions.

Light Intensity-Glare

The arrow board displays as many as 10 bulbs of 8630 cd (8800 candlepower) each at once, which is an intensity of approximately 86 300 cd (88 000 candlepower). This is intense enough to capture the attention of the driver, as shown in various studies (2, 14, 32, 33). Again, the power of the flashing image is such that a vehicle passing close up may be subjected to a glare condition, especially at night or in inclement weather (2). Also, this much light completely eliminates a driver's adaptation to darkness. This could pose a problem for drivers when it is quite dark (i.e., no artificial illumination) past the arrow board. Two simple design principles address this potential problem: (a) lens hoods found on arrow boards and (b) automatic dimmers found on most arrow boards. The lens hoods recommended are the 360° type, which encase the entire lens, and not the 180° traffic light type found on some boards. The 360° lens hoods are best at capping dispersed light to passing drivers and, in turn, direct the flashes outward in a straight line, perpendicular to the board. Other techniques could be used but are probably not as cost-effective. One alternative is a focused lens and the other is a polarized lens. If an arrow is not recognizable at 1.6 km (1 mile), these same techniques could be used to improve arrow definition and, hence, recognition distance.

Since no empirical data were found to document the glare problem of arrow boards, particularly at night, we conducted a brief field investigation of this phenomenon. In this investigation photometer readings of the ambient conditions, the background of the board, and the lamps themselves were recorded. These readings were taken after dark. Various subjects were asked to drive toward the arrow board and tell when they experienced

1. Detection of the arrow board as a flashing signal,
2. Recognition of the arrow image,
3. Beginning of image deterioration (glare or distortion), and
4. Any discomfort because of light intensity (glare).

The experiences reported by the subjects were expected. First detection from afar consisted of a flashing set of lights. Second response, some distance later, was the resolution of an arrow board. Not until in very close proximity to the arrow board did the subjects experience a discomforting glare sensation—from approximately 30.5-61.0 m (100-200 ft) up to parallel with the board. This was a conservative situation, in that heavy traffic created ambient light before and after the arrow board. This item is crucial, however, since motorists can be blinded in a split second and perform a dangerous swerving maneuver or completely lose their adaptation to the dark after going past the arrow board. We can assume that the glare effect near the arrow board is enhanced in fog and other inclement conditions (2). Since the effect occurs only in fairly close proximity to the arrow board, it seems particularly imperative to use the 360°-lens hood on each lamp. This way, the driver will be protected from the then extraneous flashes when he or she is parallel to the arrow board. A final word might be said to advocate a further dimming potential of the boards. Current capabilities commence dimming of luminance as ambient conditions fall below 4.9 cd (5 candlepower). This could be upgraded to be more sensitive to inclement weather conditions and begin dimming with lesser diminution of daylight. Also, a further reduction of intensity (5 to 10 percent) at night would probably not degrade arrow-board performance and would have a small impact on glare reduction but probably would not result in design or operational savings.

Angularity and Placement

The literature addresses the question of angularity of alignment with respect to the oncoming driver, in most cases, based on the general human factors design principles for visual displays. In general, a driver is best attracted by a straightforward, direct image, which attracts his or her attention and conveys the intended message (13). This means that optimum placement of the arrow board is head-on to the driver, perpendicular to the shoulder of the roadway. Any slight deviation of this would probably be appropriate only in a curved roadway situation, where the driver might encounter the arrow board from other than exactly head-on. This is consistent with the intent of the arrow board to move drivers out of a lane, not to change driver behavior in all lanes of travel.

The placement issue can be looked at from two dimensions: (a) shoulder versus lane placement and (b) beginning of cone taper in a construction zone versus deeper into the zone. The shoulder versus nonshoulder question is directly related to the meaning conveyed by the directional arrow board. Since the empirical data and various literature sources indicate that the arrow board connotes lane closure, the most effective placement of the board is directly in the lane that is being closed. The role of the arrow board on the shoulder to indicate some warning of hazards was discussed earlier. Placement of the arrow board at the beginning of the lane closure in a construction zone is the most effective position for the driver. This is documented by various research reports [i.e., Graham and others (19)] and many state highway traffic manuals [i.e., New York State (3)], which advocate this placement. This placement is also in accordance with experimental evidence (33). In this study, the symmetry of the visual pattern of the construction zone was violated if an arrow board was placed deep in the zone. The primary function of the arrow board is to give the driver initial warning from afar that a hazard situation is ahead and that a lane shift is required. After the driver nears the zone, the other channelizing devices, such as cones and barricades, direct the driver. Therefore, arrow-board placement is most efficacious at or very near the beginning of the lane closure because it is the first signal to be recognized and processed.

The implementation of arrow boards must be correct or their advantages will be lost. For example, on local highways, a contractor was observed to have placed an arrow board and other devices at exactly the distances specified by a state-prepared plan. However, the arrow board was over the crest of a hill and could not be seen until drivers were within a few hundred meters of the zone.

Ambient Light Conditions

Most factors of importance related to the use of arrow boards under varying ambient conditions have been alluded to in previous sections. The California study (5), for example, tested the effectiveness of arrow boards in causing drivers to shift lanes and demonstrated the superiority of the flashing or blinking on-off arrow over sequencing chevrons at nighttime. It is also documented that flashing lights, in general, are a strong beacon and attract immediate attention at night (2, 14) but fade to near indiscernibility in bright sunlight. Since most arrow boards have automatic dimming features, which can also revert to manual controls, the primary recommendation in this regard is to expand both the upper and lower limits of intensity capabilities so that the arrow board may be automatically or manually as sensitive as possible to changing ambient conditions. As stated in one report (2): "Viewing conditions are often far less than optimum due to glare, fog, and rain, and moving or intermittent visual signals are several times more likely to be detected than nonmoving or steady signals under the same viewing conditions." As such, this information is adapted from research on barricade flashing lights and railroad grade-crossing signal lights. No empirical evidence exists about signal detection of the arrow image under various adverse ambient conditions, except in the California study (5), which was limited in scope.

Arrow-Board Height

Current mounting heights, whether on a trailer or truck, appear adequate for arrow boards to meet the performance criteria recommended above. Further raising of the board would not prevent possible visual blockage by trucks but would add to the expense of the device. Therefore, no changes are recommended, at least from a human factors viewpoint.

CONCLUSIONS

The human factors considerations relevant to the flashing arrow board are exclusively devoted to enhancement of its performance as a powerful advance warning signal of a hazardous zone ahead. It was deemed crucial to determine the exact nature and meaning that the directional arrow image conveys to the driver. Initial empirical evidence was presented to show that it most often means to vacate a lane ahead. Since the use of the arrow board is becoming more widespread and encouraged by various highway agencies, it is most beneficial to exploit the power of this device to connote lane closure, and lane closure only, and to use other methods for other traffic hazard situations.

The arrow-board design and operation can be manipulated and optimized according to engineering expertise, but drivers require a discrete, clear directional arrow as an indication of lane closure. Violation of this, either in design, operation, or use of arrow boards for other traffic management purposes, places an uncertainty within drivers as to exactly what their behavior should be.

Recommended Arrow-Board Practices

1. The preferred operation of the arrow board is in the single on-off blinking-arrow mode;
2. The blinking arrow should not be used as a cautionary display only (i.e., for shoulder work);
3. A 360°-lens hood should be used to cap dispersing light to passing drivers and to direct the flashing lights outward in a straight line, perpendicular to the arrow board;
4. Dimming of luminance could be upgraded to be more sensitive to inclement weather conditions and to begin dimming with lesser diminution of daylight; and
5. Arrow boards should be placed at the beginning of the taper (construction zone).

The other typical design features (i.e., board size, color, lens type, and flash rate) of arrow boards meet basic human factors recommendations.

Recommendations for Further Research

1. Can arrow boards be used on arterial streets?
2. What configuration should be used for other than lane closure? (All possible light configurations for an arrow-board matrix were described and judged by two traffic engineers and two human factors engineers as candidates for a warning signal. Two candidates survived, and are suggested for further developmental research. The first is two "X" symbols wig-wagging back and forth. The second candidate is slashes, which would angle away from the shoulder or in the direction of a diversion. Both symbols would be used in the blinking, not sequential, mode.)
3. How are arrow boards detected under various ambient conditions?

ACKNOWLEDGMENT

The opinions and conclusions expressed in this publication are those of the authors and not necessarily those of any sponsoring agency. Funding for the analytic portion of this research came from the Federal Highway Administration through a subcontract from Midwest Research Institute.

REFERENCES

1. F. Copple and P. Milliman. Flashing Arrow-Bar Traffic Control Signs: A Report of Comparison Testing. Michigan Department of State Highways, Research Rept. R-613, Nov. 1966.
2. S. Hulbert and A. Burg. A Human Factors Analysis of Barricades, Flashers, and Steady Burn Lights for Use at Construction and Maintenance Work Sites. Technical Committee, American Traffic Services Assn., Washington, DC, Dec. 1974.
3. G. Cummins and others. Traffic Safety Manual. New York State Thruway Authority, April 1976.
4. N. Morton. The Effect of Arrow Board Trucks on Merging Traffic. Bureau of Traffic, Illinois Department of Transportation, Jan. 1976.
5. J. J. McAllister and D. L. Kramer. Advance Warning Arrow Sign Study. California Department of Transportation, July 1974.
6. Casell Company. Early Warner III. Napa, CA.
7. R. E. Dietz Company. Dietz Arrowboard. Syracuse, NY.
8. Empco-Lite. Safety Guard Advance Warning System. Elgin, IL.
9. Royal Industries. Introducing the Totally New Dual and Bi-Function Message Panels. Los Angeles.
10. Sign Support Study. Recommendations From Maintenance Sign Committee to Maintenance Operating Committee, Arizona Department of Transportation, Nov. 1974.
11. Manual of Traffic Controls: Warning Signs, Lights, and Devices for Use in Performance of Work Upon Highways. California Department of Transportation, 1977.
12. E. J. McCormick. Human Factors Engineering. McGraw-Hill, New York, 1970, pp. 53-63, 154-155.
13. W. F. Grether and C. A. Baker. Visual Presentation of Information. In Human Engineering Guide to Equipment Design (H. P. Van Cott and R. G. Kinkade, eds.), McGraw-Hill, New York, 1972.
14. R. J. Ruden and others; MBA Associates. Motorists' Requirements for Active Grade Crossing Warning Devices. Federal Highway Administration, Oct. 1977.
15. J. Howard and D. M. Finch. Visual Characteristics of Flashing Roadway Hazard Devices. HRB, Bull. 255, 1960, pp. 146-157.
16. T. F. Foody and W. C. Taylor. An Analysis of Flashing Systems. HRB, Highway Research Record 221, 1968, pp. 72-84.
17. W. Muhler and J. Berkhout. Detection Distance in Daylight of Roof-Mounted Emergency Vehicle Lights. Journal of Safety Research, Vol. 8, No. 2, 1976, pp. 50-58.
18. R. S. Hostetter, P. C. Harrison, and E. L. Seguin. Roadway Delineation Systems. NCHRP Rept. 130, 1972, Appendix B.
19. J. L. Graham, R. J. Paulsen, and J. C. Glennon; Midwest Research Institute. Accident and Speed Studies in Construction Zones. Federal Highway Administration, FHWA-RD-77-80, June 1977.
20. S. C. Shah and G. L. Ray; Louisiana Department of Highways. Advance Traffic Control Warning Systems for Maintenance. Federal Highway Administration, Baton Rouge, LA, FHWA-LA-105, HPR72-1M, July 1976.
21. J. P. Guilford. Psychometric Methods. McGraw-Hill, New York, 1954, pp. 154-177.
22. A. Crawford. The Perception of Light Signals: The Effect of Mixing Steady and Irrelevant Lights. Ergonomics, 1963, No. 6, pp. 287-294.
23. G. J. Alexander and H. Lunenfeld. Positive Guidance in Traffic Control. Federal Highway Administration, Superintendent of Documents, Stock No. 050-001-00094, April 1975.
24. H. W. McGee, W. Moore, B. G. Knapp, and J. Sanders; BioTechnology, Inc. Decision Sight Distance for Highway Design and Traffic Control Maneuvers. Federal Highway Administration, DOT-FH-11-9278, Jan. 1978.
25. A. Remole. Border Enhancement During Flicker Stimulation Effect of Retinal Location. Vision Research, Vol. 15, No. 12, 1975, pp. 1385-1388.

26. D. J. Tolhurst and D. S. Dealy. The Detection and Identification of Lines and Edges. Vision Research, Vol. 15, No. 12, 1975, pp. 1367-1372.
27. R. W. Swezey. Brightness Contrast Effects on Recall of Projected Highway Sign-Type Stimulus Material. Journal of Applied Psychology, Vol. 59, No. 3, 1974, pp. 408-410.
28. N. Osaka. Target Size and Luminance in Apparent Brightness of the Peripheral Visual Field. Perceptual and Motor Skills, Vol. 41, No. 1, 1975, pp. 49-50.
29. S. Magnussen and A. Glad. Effects of Steady Surround Illumination on the Brightness and Darkness Enhancement of Flickering Lights. Vision Research, Vol. 15, No. 12, 1975, pp. 1413-1416.
30. V. A. Benignus and others. Monitoring Performance as a Function of Rate of Ready Signals. Perceptual and Motor Skills, Vol. 43, No. 3, pt. 1, 1976, pp. 815-821.
31. Field Evaluation of Barricade Warning Lights. Technical Subcommittee on Lights, American Traffic Services Assn., Kansas City, MO, May 1974.
32. R. B. Goldblatt; KLD Associates. Guidelines for Flashing Traffic Control Devices. Federal Highway Administration, FHWA-RD-76-190, July 1976. NTIS: PB 264 801.
33. V. G. Bruce and M. J. Morgan. Violation of Symmetry and Repetition in Visual Patterns. Perception, Vol. 4, No. 3, 1975, pp. 239-249.
34. Protection Services, Inc. Protect-O-Flash Advance Warner. Harrisburg, PA.

Notice: The Transportation Research Board does not endorse products or manufacturers. Trade and manufacturers' names appear in this paper because they are considered essential to its object.

Experimental Evaluation of Markings for Barricades and Channelizing Devices

Richard F. Pain and Beverly G. Knapp, BioTechnology, Inc., Falls Church, Virginia

The study reported in this paper attempts to determine quantitatively the optimum design characteristics of channelizing devices. This was accomplished by performing a series of four laboratory studies. The following marking parameters were studied: (a) the design and configuration of stripes, (b) the width of stripes, (c) the color ratio of stripes, (d) the meaning of various design configurations, and (e) the detectability of visible areas (height to width combinations). For each of the experiments, 30 drivers recorded their detection and identification responses to stimulus slides, which were presented tachistoscopically. Data reduction consisted of analyses of a derived index score, which was a summary of the total response. The results allow some limited recommendations regarding channelizing: (a) optimal stripe width is a 20- or 15-cm (8- or 6-in) stripe for 15 cm or greater rails, (b) desirable ratio of white-to-orange coloring favors equal white to orange or more white, (c) optimal stripe design configurations are first vertical then horizontal, (d) chevrons connote directional meaning to drivers, (e) vertical panels elicit better performance than horizontal bars or trapezoid shapes, (f) there was little useful difference between type 1 and type 2 barricades, and (g) a tall, narrow vertical panel image is recommended over a shorter, wider device.

This research consists of four laboratory examinations of the design and marking configuration of orange and white stripes as displayed on a number of panel and barricade forms. This research serves a need to standardize and make uniform the displays on traffic controls through and around construction zones, since the safe and efficient movement of traffic through these zones is a crucial issue today. The objective is to provide a quantitative evaluation and recommend optimal designs for use of traffic-control devices in work zones. The recommended designs will be further tested later in the closed and full field evaluation tasks of the National Cooperative Highway Research Program (NCHRP) study on evaluation of traffic controls for street and highway work zones.

EXPERIMENTAL METHOD

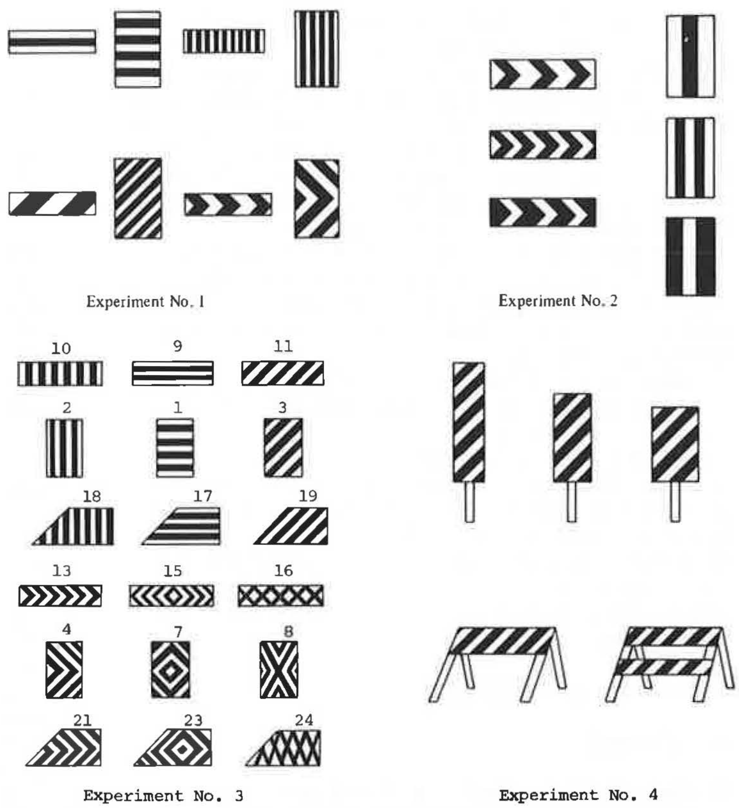
A primary driver activity is the acquisition of visual information about the highway and its immediate environs. A wide variety of visual configurations confront the driver, who must constantly search the roadway for appropriate guidance and navigational cues. This search and detection process is particularly important in a work or construction zone setting, where there are unexpected changes in the roadway and many distracting visual cues.

A laboratory setting was used to investigate traffic-control-device markings for construction zones. Although a laboratory experiment is not intended to be a direct simulation of the driving task, it can be made more relevant if the subjects' tasks are similar and the information load is similar to that of driving. To accomplish this, a general experimental method that emphasizes search and detection performance was designed.

A visually noisy and fairly abstract background was created. Four of these pictures were placed together to form a square; each quadrant of the square was the same picture. Small stimuli [e.g., bar or panel of a particular stripe width, orange-to-white color ratio, height-to-width ratio, and stripe design (horizontal, vertical, 45° slant, chevron)] were placed on one quadrant of the square, and another picture was taken. The resultant slide was then projected tachistoscopically at a fast speed (0.4 or 0.8 s). The subject's task was to search the four quadrants, identify the type of design, and identify the shape (bar or panel). Figure 1 presents selected samples of the device stimuli used for each of the four studies. The placement of device stimuli was completely random in the slides both for choice of quadrant and placement within the quadrant.

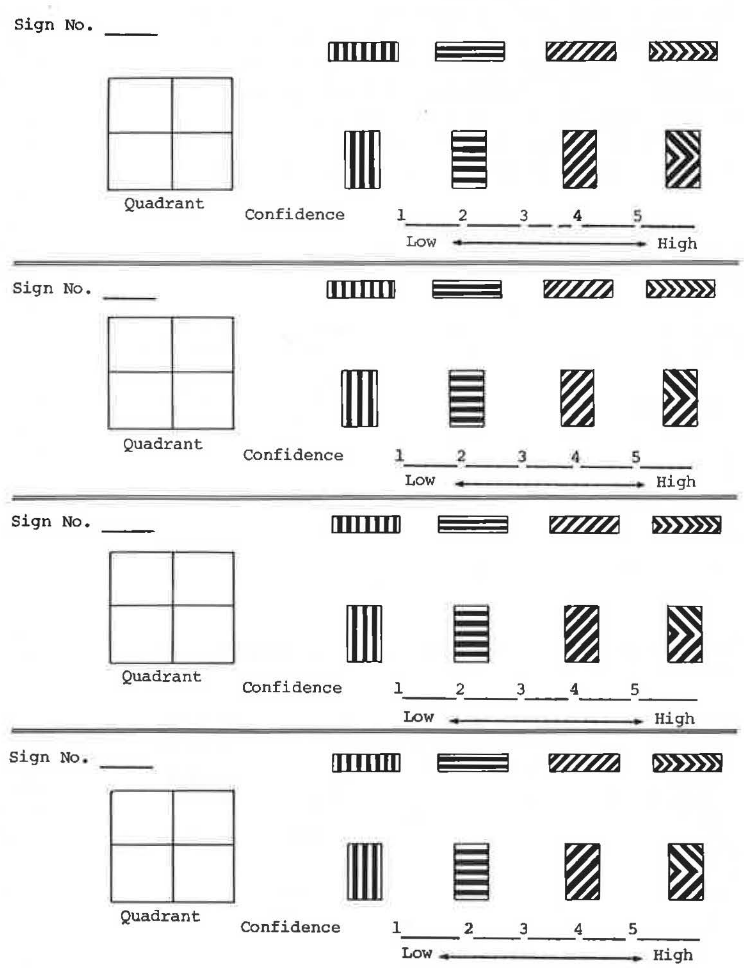
The measures of performance by subjects responding to these stimuli were, thus, a Q-score-quadrant detection, C-score-configuration identifica-

Figure 1. Sample stimuli used in each of the four experiments.



Note: Numbers in Experiment No. 3 refer to corresponding stimuli listed in Table 5.

Figure 2. Sample subject response sheet (experiments 1 and 2).



tion, and S-score-shaped identification. A subject scored 1 for correct response in each or 0 if incorrect. These three were then summed for each stimulus for each subject to obtain a combined index score of performance for each stimulus. All subjects in each experiment saw all stimuli; therefore, the same basic subjects-by-treatments analysis of variance (ANOVA) could be applied to the performance data obtained. This basic model prevails in all four studies.

In addition, subjects were asked to indicate how confident they were of their responses on a scale of 1 (low) to 5 (high). All of these measures were collected on a response sheet, a sample of which may be seen in Figure 2.

Thirty licensed drivers, aged 17-60, were tested for each of the four experiments. Subject information collected after each experiment included age, sex, driving experience, and comments (if any). The stimulus slides were shown and subjects marked their response sheets. In experiment 1, two exposure durations were used, 0.4 and 0.8 s. An initial analysis of the data indicated that the 0.8-s speed was not effective for discrimination between stimuli and was, therefore, not used in subsequent experiments.

ANALYSES AND RESULTS

Experiment 1

The purpose of the first experiment was to determine the optimum stripe width for use on channelizing devices. Simulated 10-, 15-, and 20-cm (4-, 6-, and 8-in) stripes were studied. ANOVA of the mean index performance score was applied to subjects by treatments. In general, the sample of subjects tested was representative of the driving population so that differences due to age, sex, and driving experience were not a factor.

In view of the possible interaction between stripe width and device shape (bar or panel), separate ANOVAs were performed for each. Tables 1 and 2 and Figures 3 and 4 present the results obtained. The primary finding in each of these sets of data is that the 10-cm simulated width is clearly inferior to the 15- and 20-cm widths. In most cases, there is statistical significance between the 10-cm and the 20-cm widths. Discriminability between the 15- and 20-cm widths is not as clear nor is it significant, which suggests that either would be acceptable for use on devices in the real world. The 20-cm width would seem to be preferable (especially for larger-size devices), since the 20-cm scores are all superior to those for 15 cm and 10 cm (particularly in the case of the panels). However, the 20-cm stripe is neither consistently nor significantly superior to the 15-cm stripe.

The general superiority of the 20-cm width and the concurrent acceptability of the 15-cm width does not address smaller issues, such as the discrepancy between horizontal bar versus panel detection and the inferiority of the slanted and chevron configurations. Although performance in 20-cm detection is clearly superior or at least equivalent in all cases, the chevron, and to a lesser extent, the slanted design are, in general, detected poorly. The poor detectability of the horizontal bar of 10- and 15-cm stripes is also apparent. These underlying trends, which surfaced as a subset of the primary finding, account for the significant interactions between width and configuration. Although they do not discount the impact of the basic finding of the 20-cm width's superiority, they do suggest the need for a more detailed analysis of the data. The data invite more complex hypotheses about the elements that actually operate in the perceptual stimulus characteristics of these designs, but this was beyond the scope of this small study.

The final procedure in experiment 1 was the generation of a table of correlation coefficients to determine whether any relationship existed between the subjects' actual performance (index score) and their confidence in each response. This is presented in Table 3. The general trend supports increased confidence with actual correctness of response; most coefficients were at least 0.45. A few exceptions are somewhat notable, though. Twelve percent (six coefficients) are below 0.25, as shown by the note in the table. A reexamination of actual placement of these stimuli on the stimulus background reveals that five out of six of these were in fact placed by the random iterations at the very periphery of the quadrants; four out of six were along the bottom edge. This is an entirely new dimension of consideration—peripheral versus central fixation and target-search behavior. Detailed examination of this phenomenon is outside of the scope of this study. However, it does explain some low scores and low confidence simply because of missed targets or, conversely, some spurious good scores due to chance guessing.

Table 1. Experiment 1: summary of ANOVA—stripe width versus panel.

Source of Variation	df	Sum of Squares	Mean Squares	F-Ratio Model Three
Width (A)	2	31.1302	15.5651	9.1297
Configuration (B)	3	158.9453	52.9818	30.5782[a]
Subjects (C)	31	240.7891	7.7674	
AB	6	27.0156	4.5026	2.4194[b]
AC	62	105.7031	1.7049	
BC	93	161.1380	1.7327	
ABC	186	346.1510	1.8610	
Total	383	1070.8724		

[a]Significant at 0.01 level. [b]Significant at 0.05 level.

Table 2. Experiment 1: summary of ANOVA—stripe width versus barricade.

Source of Variation	df	Sum of Squares	Mean Squares	F-Ratio Model Three
Width (A)	2	72.2500	36.1250	22.4912[a]
Configuration (B)	3	74.6536	24.8845	14.2624[a]
Subjects (C)	31	292.5182	9.4361	
AB	6	96.1667	16.0278	9.1822[a]
AC	62	99.5833	1.6062	
BC	93	162.2630	1.7448	
ABC	186	324.6667	1.7455	
Total	383	1122.1016		

[a]Significant at 0.01 level.

Experiment 2

The purpose of the second experiment was to determine the optimal color ratio (white to orange) to be used on barricades, panels, and drums. Each design configuration examined in experiment 1 (horizontal, vertical, slant, chevron) was prepared in a simulated 15-cm stripe width using white-to-orange color coverage of 1:2, 1:1, and 2:1. Since in experiment 1 many demographic and methodological variables were considered and re-

Figure 3. Configuration versus stripe width (panels).

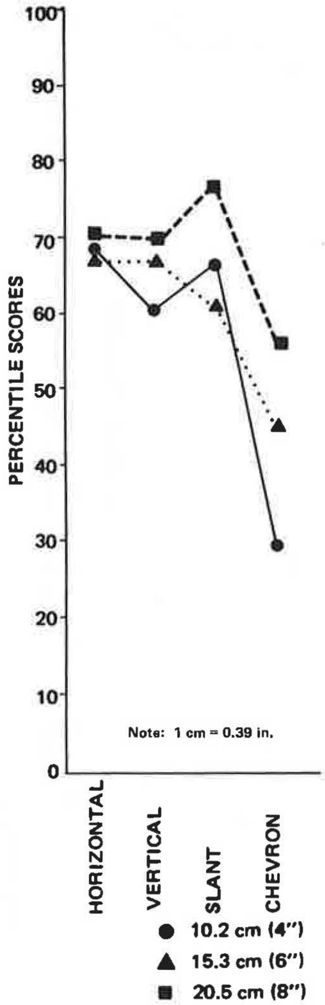


Figure 4. Configuration versus stripe width (bars).

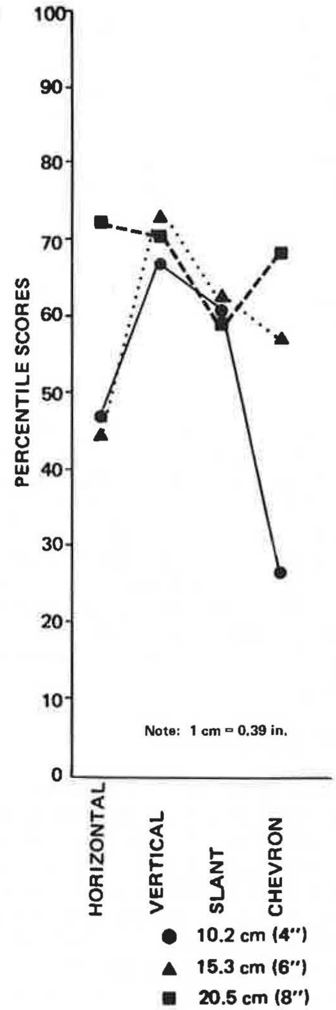


Table 3. Correlation coefficients—total index score x confidence level.

Shape	Width (cm)	Horizontal Dark	Horizontal Light	Vertical Dark	Vertical Light	Slant Dark	Slant Light	Chevron Dark	Chevron Light
Panel	10	0.63	0.59	0.60	0.12[a]	0.81	0.88	0.62	0.21[a]
	15	0.59	0.45	0.59	0.64	0.44	0.63	0.48	0.66
	20	0.60	0.30	0.49	0.19[a]	0.52	0.40	0.66	0.81
Bar	10	0.43	0.10[a]	0.30	0.57	0.58	0.70	0.58	0.35
	15	0.73	0.16[a]	0.54	0.60	0.55	0.78	0.78	0.50
	20	0.74	0.47	0.40	0.23[a]	0.54	0.63	0.64	0.41

Note: 1 cm = 0.39 in.
[a]Below 0.25.

fined, one high-speed stimulus exposure (0.4 s) was used for all of the slides in this study, and the responses were analyzed directly for the primary objective—color ratio. The ANOVA was performed (subjects-by-configuration-by-color ratio), again using the total index score achieved for each stimulus presentation. Table 4 and Figure 5 present the results of this analysis.

The significant F-ratio for color ratio clearly indicates sufficient performance differences due to this factor. In general, the 2:1 white-to-orange ratio is slightly advantageous, although not significantly different from the 1:1 ratio results. The 1:2 white-to-orange ratio (essentially an overabundance of orange) appears to be inferior, although in most cases, not significantly so. Logic helps explain these findings. The stimuli were, in general, seen against a multicolored, visually noisy background, which simulated some cluttered construction areas found in the real world. The bright white stands out best against this general dim melange of background noise. Were the bright orange viewed in a more clean, open, white pavement-type situation, results might reflect some superiority of more orange than white because of the changed contrast. Thus, the laboratory conditions have more effectively addressed the more common, visually cluttered and dim ambient condition in which equal or more white than orange contrasts with the background.

The general trend of higher performance scores for the horizontal and vertical configurations than

Table 4. Experiment 2: summary of ANOVA—subject versus configuration versus color ratio.

Source of Variation	df	Sum of Squares	Mean Squares	F-Ratio Model Three
Configuration (A)	5	336.7795	67.3559	21.9871*
Color ratio (B)	2	80.4306	40.2153	7.8983*
Subjects (C)	31	896.4288	28.9171	
AB	10	184.4444	18.4444	4.5351*
AC	155	474.8316	3.0634	
BC	62	315.6806	5.0916	
ABC	310	1260.7778	4.0670	
Total	575	3549.3733		

*Significant at 0.01 level.

for the slant and chevron patterns continued, as in experiment 1. The traditional slanted pattern in current use does emerge, however, as superior in the 1:1 ratio. In fact, t-tests performed between various pairs of designs reveal the significant superiority of the vertical and horizontal patterns over the chevron design. The small but significant F-ratio for the interaction between color ratio and configuration invites the examination of more complex issues that are not within the scope of this study.

Experiment 3

This experiment was conducted to determine the inherent meaning conveyed by the stimuli used in the previous experiments. For example, do the chevrons and stripes indicate a particular direction or path to follow if a driver encounters them in the real world? The test of this question was a forced-lane choice, a left or right divergence from the center lane of travel on encountering one of the devices in the center lane ahead.

In this task, 24 devices were presented twice each in random order (see Figure 1 for examples). Therefore, two trials were obtained for each stimulus. Simple frequency counts of the number that chose the right lane versus the number that chose the left lane were accumulated. Then z-scores were computed to see whether these proportions differed significantly from the chance expectation (50-50) that no particular lane bias exists, given any particular stimulus design. Table 5 presents the 24 devices shown and the proportions and z-scores obtained for each.

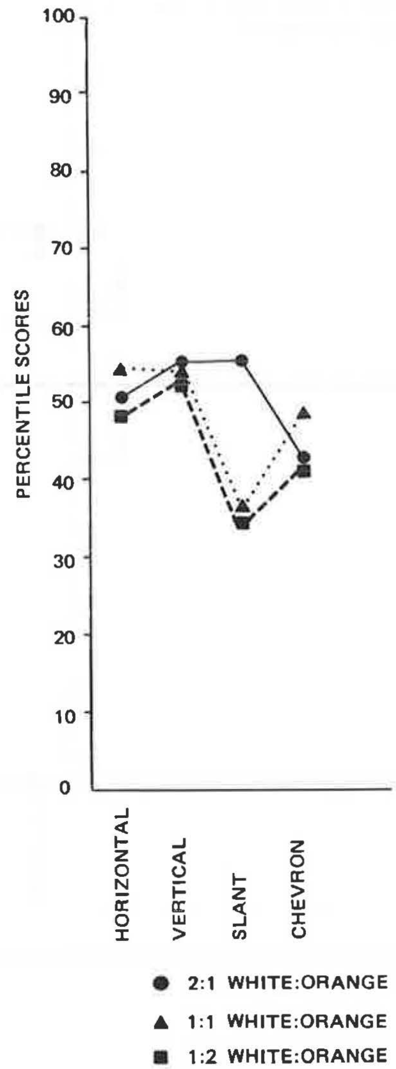
The significant z-scores are marked as shown. Three basic trends may be seen in these data.

1. The chevron effectively indicates a direction on bars and panels,
2. The new shape device (the one side points left instead of straight) clearly indicates a direction to the left regardless of the design, and
3. The slanted and even horizontal and vertical lines seem to indicate a direction to many drivers in the bar shape but not in the panel shape.

When the chevron is viewed in isolation against a reasonably uncluttered background (as was the case here), it looks exactly like a series of arrow heads indicating a direction to the right or left. In fact, this was the exact response. In one case a chevron bar pointing to the left elicited an unambiguous left-lane choice on both stimulus trials.

The shape or form of the new device induced many drivers to select the left lane because its

Figure 5. Configuration versus color ratio (white to orange).



pointed edge indicated this direction. This shows the stronger impact of sign forms, even when the design configuration within the form actually pointed to the right (i.e., slant-right design). In this case, the power of the chevron configuration was diminished by the power of the shape of this form because as many drivers selected a left lane as a right lane when chevrons pointing right appeared on the new form. This, incidentally, is in accordance with basic perceptual principles, which state that form perception and response are more basic than design symbology, which in turn is more basic than verbal message (1, 2).

The bar shape in the center lane apparently looks like a more realistic channelizing device than the panel-shape appearance of a panel or drum. Many drivers selected a right-lane choice on seeing a vertical or horizontal bar. This may well indicate the natural tendency to avoid obstacles by moving to the right rather than to the left. The lone barricade is a fairly common sight on urban arterials and a lot of drivers may be used to circumventing it to the right rather than risk oncoming traffic to the left.

Note that the above conclusions are based on choice and common sense data rather than on actual performance data, as in the preceding portion of

Table 5. Directional decision proportions and Z-scores obtained for channelizing devices.

Stripe Pattern/ Sign Configuration Stimuli	Trial No. 1 Proportion Left	Right	Z-Score	Trial No. 2 Proportion Left	Right	Z-Score
1. Horizontal panel	0.42	0.58	1.00	0.37	0.63	1.53
2. Vertical panel	0.27	0.73	2.63[a]	0.39	0.61	1.34
3. Slant right panel	0.41	0.59	0.96	0.23	0.77	2.96[a]
4. Slant left panel	0.53	0.47	0.32	0.58	0.42	0.89
5. Chevron right panel	0.06	0.94	4.98[a]	0.06	0.94	4.92[a]
6. Chevron left panel	0.97	0.03	5.14[a]	0.97	0.03	5.14[a]
7. Double chevron panel	0.40	0.60	1.09	0.33	0.67	1.86
8. X panel	0.34	0.66	1.72	0.29	0.71	2.32[b]
9. Horizontal bar	0.27	0.73	2.63[a]	0.37	0.63	1.53
10. Vertical bar	0.26	0.74	2.79[a]	0.32	0.68	2.01[b]
11. Slant right bar[c]	0.23	0.77	3.13[a]	0.26	0.74	2.79[a]
12. Slant left bar[d]	0.52	0.48	0.11	0.37	0.63	1.53
13. Chevron right bar	0.09	0.91	4.64[a]	0.06	0.94	4.92[a]
14. Chevron left bar	1.00	0.00	5.66[a]	1.00	0.00	5.59[a]
15. Double chevron bar	0.38	0.62	1.53	0.45	0.55	0.55
16. X bar	0.39	0.61	1.34	0.33	0.64	1.86
17. Horizontal new	0.97	0.03	5.20[a]	0.94	0.06	4.80[a]
18. Vertical new	0.88	0.12	4.19[a]	0.97	0.03	5.20[a]
19. Slant right new	0.91	0.09	4.53[a]	0.84	0.16	3.69[a]
20. Slant left new	0.84	0.16	3.85[a]	0.94	0.06	4.80[a]
21. Chevron right new	0.53	0.47	0.36	0.48	0.52	0.22
22. Chevron left new	0.47	0.53	0.45	0.48	0.52	0.22
23. Double chevron new	0.60	0.40	1.09	0.84	0.16	3.69[a]
24. X new	0.78	0.22	2.83[a]	0.81	0.19	3.35[a]

[a] α p < 0.01 > 2.58.
[b] α p < 0.05 > 1.96.
[c] Stripes slant from lower left to upper right corner.
[d] Stripes slant from lower right to upper left corner.

the experiment and the previous two experiments.

The detection and identification of the chevrons and slanted patterns was, in general, poorer than that of the horizontal and vertical designs. However, chevrons (and to some extent slanted lines) seem to convey some directional meaning. The vertical and horizontal designs seem to evoke meaning only when displayed in a fairly realistic form that is recognizable as a barricade. That a chevron carries a strong meaning but is not as easily detectable as some other patterns, which apparently have no meaning, is not discouraging. In general, the ultimate fate of these channelizing devices is not to stand alone and be detected as a single unit but rather to be one small part of an entire device array. A large enough, bright enough chevron barricade would probably be highly detectable, but this is not a cost-effective approach. The larger question is the effective detection of these device elements as a part of a larger array.

Experiment 4

The final experiment was designed to determine the most effective height-to-width ratio of barricades and vertical panels. The four-quadrant background of embedded stimuli was again projected tachistoscopically to subject drivers. The design configuration was held constant by using the traditional slanted stripes on the barricades and panels. Thus, the only detection parameters were stimulus location (quadrant) and stimulus shape (type 1 barricade, type 2 barricade, or vertical panel). Illustrations of these stimuli are found in Figure 1.

A short digression is in order on the terms type 1 and type 2, which are used in this report. A type 1 barricade is simply defined as a barricade that has one rail. Similarly, a type 2 barricade is a device that has two rails. This is inferred from the absence of a definition in the standard manual, the Manual on Uniform Traffic Control Devices (MUTCD) (3). The MUTCD provides minimum design specifications for the type 1 and type 2 barricades, independent of each other. These specifications include rail width, rail length, stripe width, and height. Only by inference from the text and drawings is a type 1 assumed to be a one-rail device since a specification for it is that it have a "single reflectorized face" (two-sided), and the type 2 must have two such rail faces. Thus, no relationship of any kind is drawn between the display areas of the one rail or the two rail. For example, a one rail might be one-half of the display area of a two rail or two-thirds of it. Other state and utility manuals [i. e., Texas A&M Traffic Control Manual (4) or New York State's manual (5)] also avoid any real definition or relation of type 1 and type 2 devices, although the Texas manual does state that a type 1 shall consist of "an upper rail on an A-frame support; and a type 2, an upper and lower rail."

This report focuses on the perceptual aspects of a one-rail versus a two-rail display by having one given display area be shown as both two small rails and also as one large rail, each of the same total area.

The data were subjected to two ANOVAs, subjects-by-width-by-barricade type or, for panels, by height by using the index score. Tables 6 and 7 present the results of these analyses.

Table 6 provides F-ratios for the type 1 and type 2 barricades in conjunction with increasing rail width. A glance at the F-ratio and Figure 6 immediately reveals that the main effects are not significant. A divergence occurs between rail width and the number of rails, beginning with width number 2. This suggests that some type of interaction occurs between these parameters, and in fact the F-ratio for these is significant. The nature of this function is not entirely clear from the data, however. No simple principle can be derived that states that increasing area of display should be separated from one into two rails of a given visual area for optimal detection because there are too few data points to define the exact function.

A fairly simple conclusion is evident, however. When the area is very small or very large, one-rail or two-rail displays interchange reasonably well. Within the medium range of areas (widths 2

Table 6. Experiment 4: summary of ANOVA—subjects versus height versus barricade type.

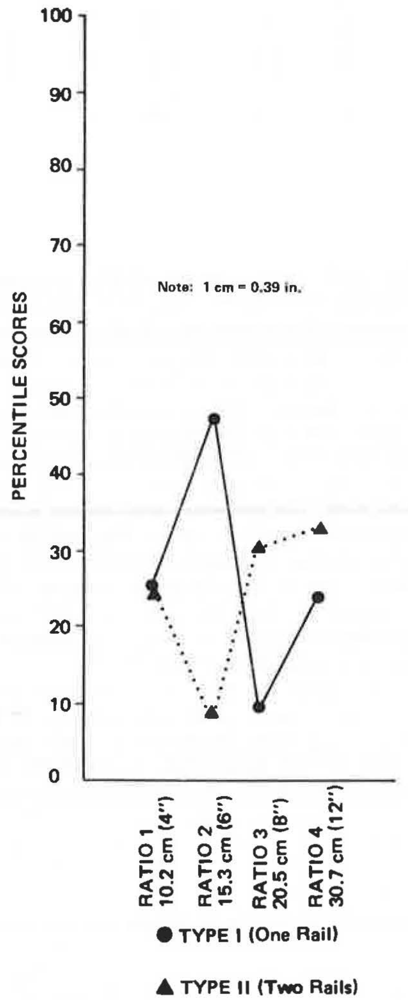
Source of Variation	df	Sum of Squares	Mean Squares	F-Ratio Model Three
Rails (A) type 1 or type 2	1	0.1856	0.1856	0.2589
Rail size (B)	3	5.0720	1.6907	2.6261
Subjects (C)	32	52.1667	1.6302	
AB	3	49.6477	16.5492	20.7062[a]
AC	32	22.9394	0.7169	
BC	96	61.8030	0.6438	
ABC	96	76.7273	0.7992	
Total	263	268.5417		

[a]Significant at 0.01 level.

Table 7. t-ratios—number of barricade rails versus rail height.

Pair of Conditions	t-Ratio	df	Significance
Type 1 versus type 2, ratio 1	0	32	NS
Type 1 versus type 2, ratio 2	6.95	32	0.01
Type 1 versus type 2, ratio 3	4.05	32	0.01
Type 1 versus type 2, ratio 4	1.50	32	NS
Ratio 1 versus ratio 2, type 1	4.51	32	0.01
Ratio 1 versus ratio 2, type 2	2.60	32	0.05 only
Ratio 1 versus ratio 3, type 1	3.73	32	0.01
Ratio 1 versus ratio 3, type 2	1.29	32	NS
Ratio 1 versus ratio 4, type 1	0.14	32	NS
Ratio 1 versus ratio 4, type 2	1.60	32	NS
Ratio 2 versus ratio 3, type 1	5.96	32	0.01
Ratio 2 versus ratio 3, type 2	4.37	32	0.01
Ratio 2 versus ratio 4, type 1	3.81	32	0.01
Ratio 2 versus ratio 4, type 2	4.54	32	0.01
Ratio 3 versus ratio 4, type 1	2.41	32	0.05 only
Ratio 3 versus ratio 4, type 2	0.36	32	NS

Figure 6. Rail height versus type 1 and 2 barricades.



and 3) a breakdown occurs. To test these conclusions, t-tests were performed by comparing the various rail and width combinations as pairs. These ratios are presented in Table 7. As given in the table, the two extreme widths (1 and 4) do not significantly differentiate between type 1 or 2 barricades. The divergence occurs in the middle, as suggested above. A one-rail, type 1 barricade is significantly better at width 2, but this reverses at width 3. The suggestion is that the individual rails must be at a minimum width before they are plainly detectable; then, if there are two rails instead of one, this presents a larger total image, which is easily detectable. This is supported by noting the significant t-ratio between the two-rail widths (2 versus 4 and 2 versus 3 comparisons) which are highly significant, yet the one-rail width (3 versus 4) is significant at $\alpha = 0.05$ only. Essentially, after a minimal area is achieved, two rails to contain the area are optimal, but one rail is only adequate in one certain case. The smallest width seems to present a detection problem for both one- and two-rail displays.

A more complex set of factors than just simple area of display as seen in one or two rails is operating here. Apparently, the total image projected by the barricade of bars and stripes that slice up the visual background is related to such factors as the way this is embedded in the background and where it is against the background. A simple height-to-width function does not adequately describe all of the factors that operate in this situation.

Table 8 and Figure 7 provide the results for the height-to-width analysis for vertical panels. Here the findings are clean and neat. A more narrow image, whether short or tall, is more easily detected and identified. There is no significant interaction between these factors. This finding is in harmony with data from the previous laboratory studies, which indicate the overall superiority of the vertical panel as a detection stimulus. Apparently, the eye best detects a clear vertical image against the very cluttered, dim background of the display. Those targets that have more width than height, not only within the design but in overall form, were generally less effective in detectability.

RECOMMENDATIONS

Based on the laboratory studies, several recommendations applicable to channelizing devices and to further testing them in field studies are presented. One qualification should be mentioned: Stimuli were only seen against a dim background with high visual noise. Another background may produce somewhat different results. Asterisks indicate which recommendations will be field tested in later tasks.

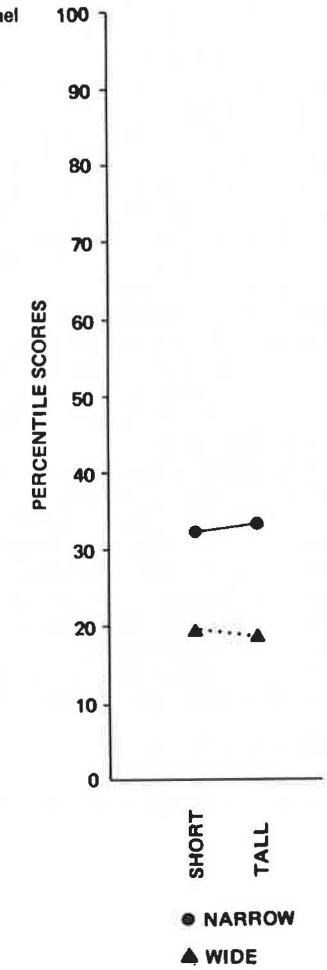
*1. In terms of shape, the vertical panel is somewhat superior to the barricade. Because the differences were not extensive, field testing is highly recommended. This testing should consider the effect of a long array of devices as opposed to the single devices seen in the laboratory studies.

Table 8. Experiment 4: summary of ANOVA—subjects versus panel width versus panel height.

Source of Variation	df	Sum of Squares	Mean Squares	F-Ratio Model Three
Panel height (A)	1	0.0076	0.0076	0.0111
Panel width (B)	1	10.3712	10.3712	20.2627[a]
Subjects (C)	32	33.5606	1.0488	
AB	1	0.0682	0.0682	0.1486
AC	32	21.7424	0.6795	
BC	32	16.3788	0.5118	
ABC	32	14.6818	0.4588	
Total	131	96.8106		

[a]Significant at 0.01 level.

Figure 7. Panel height versus panel width.



*2. For vertical panels, a tall, narrow shape is recommended over a shorter, wider device. This clear-cut laboratory result should be tested further in the field, with the device located in an array and also in a visually cluttered work zone situation.

3. No clear-cut distinction between type 1 and type 2 horizontal barricades was found in the laboratory. Both types seem equally detectable. Field testing of the type 1 barricade is recommended. Manufacturing cost and logistical convenience were important considerations behind this recommendation.

The general detection and identification of the chevron was poor. Unless the strong directional image can be projected more successfully, the use of chevrons on barricades and panels of the size simulated in the laboratory is not recommended. With this qualification, the following recommendations are made:

*4. No one stripe pattern was clearly optimal in the laboratory test; however, the chevron was consistently the least detectable. Further study should be performed in the field with all the stripe configurations. If this is not possible, field studies should compare the most detectable combinations of stripe pattern and shape (horizontal stripes on a vertical panel and vertical stripes on a horizontal bar) versus the least detectable patterns (chevrons on either shape).

*5. The optimal stripe pattern, of the six tested, for denoting direction is the chevron. No other pattern gave directional meaning consistently enough to be considered a source of directional information. Since these configurations have only been tested singly, we recommend that they be studied in the field arranged in device arrays.

6. The offbeat design used (double chevrons, and X configurations) were either ineffective or spurious and are not recommended for further consideration.

7. The optimal stripe width is a 20- or 15-cm (8- or 6-in) stripe. The 20-cm stripe is preferable, particularly on larger devices, but the 15-cm width is currently used and the evidence is not strong enough to warrant a costly changeover.

8. The desirable ratio of white to orange on horizontal bars, vertical panels, or drums is equal white to orange (1:1). The results were not highly significant, but this is the ratio in current use. The cost and logistics of change would not be warranted according to these findings. Use of more orange than white is discouraged.

This recommendation is in the context of somewhat dim, visually noisy background as opposed to an open, white concrete pavement, which could optimize a higher ratio of orange. One exception to the above finding is that the slanted stripes in current use today are more easily detected when in a white-to-orange ratio of 2:1.

CONCLUDING REMARKS

The preceding results, discussions, and recommendations are based on four short laboratory experiments of rather limited scope and purpose. As with most research, more questions seem to be uncovered during the investigations than were answered. A larger problem than simple discernments of stripe widths, shapes, and configurations seems to underlie the data. The perception of lines, angles, and edges as they increase geometrically in size is not, apparently, a straight-line function that can be perfectly correlated with increased detectability. Rather, the actual image configurations achieved as these lines and forms interact with the display background draw on basic principles of perceptual organization and deserve further investigation. Quite simply, more data points are needed to describe and model the functions of optimal detectability as parameters of size, height, width, design stripe widths, angularity of designs, appearance against varying ambient backgrounds, and position within the entire visual field are altered. The search for a good visual image for detection necessitates the testing of many display sizes and configurations. Apparently a certain arrangement of the design elements against a certain background produces optimal detectability. Specification of these parameters based on current data is not possible.

The determination of optimal stripe widths, color ratios, and height-to-width ratios for barricades, panels, and drums was executed as the driver detected and identified these device simulations in isolation, against a background of visual clutter, designed to simulate informational loadings in the real world. In reality, these devices are not generally perceived alone but as a cluster or array that protects and channels traffic away from hazardous zones. Therefore, the design recommendations and findings are inputs to field tests that examine these individual devices in combination rather than alone.

Our purpose was not to generate the single channelizing device of optimum detectability but rather to generate input for field testing and to eliminate those elements that were rated consistently poor in performance. Our laboratory studies suggest the best and the worst designs that should be tried under real driving conditions so that their ability to display a hazard situation effectively and channel drivers around it with the least perturbance of normal driving can be evaluated.

ACKNOWLEDGMENT

The research reported here was conducted as a part of the NCHRP evaluation of traffic controls for street and highway work zones. The able assistance of those others associated with this project is gratefully appreciated. The opinions and conclusions expressed or implied in this report are those of the research agency. They are not necessarily those of the Transportation Research Board, the National Academy of Sciences, the Federal Highway Administration, the American Association of State Highway and Transportation Officials, or of the individual states participating in NCHRP.

REFERENCES

1. F. Attneave. Some Informational Aspects of Visual Perception. Psychological Review, Vol. 61, 1954, pp. 183-193.
2. I. Rock. An Introduction to Perception. Macmillan, New York, 1975, Chap. 8.
3. Manual on Uniform Traffic Control Devices, Part VI: Traffic Controls for Street and Highway Construction and Maintenance Operations. U.S. Department of Transportation, Sept. 1978.
4. Traffic Controls for Highway Construction and Maintenance Operations. Texas A&M Univ., Texas Office of Traffic Safety, 1977.
5. Traffic Safety Manual. New York State Thruway Authority, April 1975.

Abridgment

Visibility Requirements for Traffic-Control Devices in Work Zones

Hugh W. McGee, Wagner-McGee Associates, Alexandria, Virginia
Beverly G. Knapp, BioTechnology, Inc., Falls Church, Virginia

Highway safety officials are concerned that traffic-control devices used at work zones are not as visible as they should be due to insufficient reflective properties or because dirt has rendered them ineffective. It has therefore been suggested that a performance standard be established for reflective devices used in work zones. Accordingly, the objective of this study was to develop a performance requirement or standard for the detection and recognition of retroreflective traffic devices used in work zones.

The scope of the study was limited to an analytical exercise and drew on existing information and data where possible. The discussion focuses primarily on those channelization devices frequently used in work zones (i.e., drums, barricades, panels, and cones). The performance standard developed in this study was established from the principles of driver information needs and, specifically, the requirement for decision sight distance. The performance standard is presented in terms of visibility requirements, that is, the distance at which motorists should be able to detect and recognize the devices at night.

INFORMATION REQUIREMENTS FOR WORK ZONES

The concept of decision sight distance has been defined by Alexander and Lunenfeld (1) as

> The distance at which a driver can detect a signal (hazard) in an environment of visual noise or clutter, recognize it (or its threat potential), select appropriate speed and path, and perform the required action safely and efficiently.

It is one of the underlying components of the broader concept of positive guidance, which has been given the following operational definition (2):

> Any information carrier, including the highway, that assists or directs the driver in making speed or path decisions provides guidance information. Positive guidance information is provided when that information is presented unequivocally, unambiguously, and conspicuously enough to meet decision sight distance criteria and enhance the probability of appropriate speed and path decisions.

The work zone, in almost all instances, requires the motorist to make some change in speed and path. Therefore, by applying the principles of positive guidance and, more specifically, the concept of decision sight distance, one can develop analytical performance standards for reflective devices in work zones.

Information Handling Zones

A procedure described in the User's Guide (2) includes the determination of information handling zones. The whole process of positive guidance is based on the premise that the motorist has to contend with different hazards during the guidance level of driver performance (i.e., the driver's task of selecting a safe speed and path on the highway). Hence, one of the zones is referred to as the hazard zone.

A construction or work zone typically fits within the category of a highway condition hazard. As stated in the User's Guide (2), a condition hazard is "any location where the condition of the highway needs to be interpreted by the driver as a cause for extra caution". The primary hazard associated with any construction zone is the actual work site where people and machinery congregate; however, the devices that channel the motorist around this hazard become, paradoxically, hazards themselves. These devices (barricades, cones, drums, and panels), when placed across the lane, are obstacles that the motorist must avoid. Therefore, detection and recognition of these devices is critical to the successful negotiation of the work zone.

The next information handling zone defined in the positive guidance process is immediately upstream of the hazard and is referred to as the nonrecovery zone. This zone is defined as the distance required to execute an avoidance maneuver, or the point beyond which the motorist cannot avoid the hazard unless he or she resorts to erratic maneuvers. This distance corresponds to the stopping sight distance as described by the American Association of State Highway and Transportation Officials (AASHTO) (3). The nonrecovery zone starts at the beginning of each hazard zone and extends upstream for a distance. This distance corresponds to the stopping sight distance for the speed at which the vehicle was operating.

The next zone upstream from the nonrecovery zone is called the approach zone. This corresponds to the decision sight distance minus the stopping sight distance. The decision sight distance, which is marked off from the leading edge of the hazard zone, should be sufficient for the motorist to detect and to react safely and efficiently to the hazard. In principle, this distance should be the key element of a specification or performance standard for reflectivity of traffic-control devices applied in the work zone.

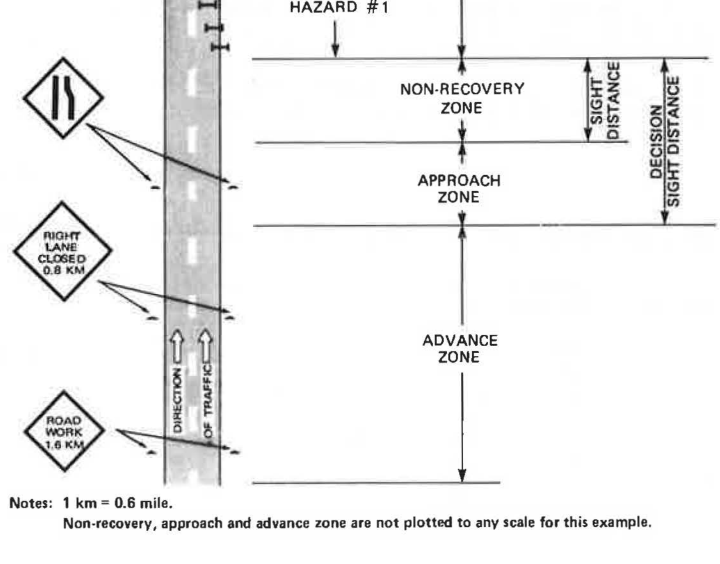
The final upstream zone is called the advance zone. By definition established in the positive guidance procedure, it represents the area where hazards or inefficiencies do not yet affect the driver's task. Hence, although labeled a zone, it is really unbounded on the upstream end. For the purpose of a work-zone situation, the advance zone would start where the first device that warns of a work zone ahead is visible to the motorist. The zone ends at the decision sight distance point (the beginning of the approach zone).

Figure 1 shows how each of the information handling zones fit together at a typical work-zone site. The example is a one-lane closure on a divided highway. Note that the nonrecovery zone and the approach zones are not plotted with respect to any particular longitudinal distance.

Decision Sight Distance Requirements

In a previous study (4), we developed specific criteria to be applied to the concept of decision sight distance.

Figure 1. Designation of information handling zones related to positive guidance procedure.



Notes: 1 km = 0.6 mile.
Non-recovery, approach and advance zone are not plotted to any scale for this example.

In that study, the conceptual definition of decision sight distance was translated into a hazard avoidance model, which was then employed to formulate appropriate values. The model describes a sequence of events that occur in hazard avoidance, starting from detection of the hazard and ending with the completion of the avoidance maneuver. The process is briefly described as follows:

1. Hazard becomes visible (time t_0)—This is the baseline-time point when the hazard is within the driver's sight line.
2. Hazard is detected (time t_1)—Driver's eye fixates on the hazard.
3. Hazard is recognized (time t_2)—The image on the eye is translated by the brain and the hazard is perceived as such.
4. Driver decides on action (time t_3)—Driver analyzes alternative courses of action and selects one.
5. Driver begins response (time t_4)—Driver initiates required action.
6. Maneuver is completed (time t_5)—Driver changes path or speed of vehicle.

The process is a simple additive model. The total time from the moment when the hazard is visible to the completion of hazard avoidance maneuver equals the sum of the incremental times for detection (t_0-t_1), recognition (t_1-t_2), decision (t_2-t_3), response (t_3-t_4), and vehicle maneuver (t_4-t_5).

Information from the literature plus some limited field experiments were used to develop times for the incremental steps and to prepare the specific decision sight distance criteria for highway work zones. The incremental times for the phases of the hazard avoidance process were determined to be as follows:

Process	Time (s)
Detection-recognition	1.5- 3.0
Decision-response	4.2
Maneuver (lane change situation)	4.5
Total	10.2-11.7

These time values can be applied to various operating speeds to arrive at the required visibility distances shown in Table 1. The lower values would be applicable to the rural environment or any situation where there is a lack of high background luminance, and the higher value is applicable to the urban environment or an area of high background luminance.

To present these visibility requirements in perspective, it is necessary to discuss the conditions for which they were developed and to which they apply:

1. The values apply to a work zone where a lane closure necessitates a lane change. This appears to be the most common situation and the one that requires the longest maneuver time.
2. The values are based on the assumption of a single vehicle approaching the work zone, not influenced by vehicles downstream. These distances should apply to a driver with 20/40 acuity (the requirement in most states) and the vehicle headlights at low beam.
3. These values are based on an unalerted driver and represent the upper percentile range of the driving public in terms of reaction and maneuver times. Furthermore, we assumed that the driver has only information from the devices in question (i.e., the barricades or panels) that are being used for channelization. This assumption ignores the fact that the motorist is alerted and informed by advance signs and other long-range detection devices, such as flashing-arrow boards or steady-burn lights. Although this assumption makes these values conservative, it is justified for safety reasons, since many work zones do not have the full complement of warning devices and therefore must rely on the reflectivity capabilities of the devices. Also, some drivers either give low primacy to the advance warning devices or simply fail to detect them.

Table 1. Visibility requirements for reflective devices at work zones.

85th Percentile Speed (km/h)	Detection Through Maneuver Time (s) Low	High	Visibility Distance (m) Computed	Rounded for Design
40	10.2	11.7	113-156	120-160
60	10.2	11.7	170-233	170-230
80	10.2	11.7	227-311	230-310
100	10.2	11.7	306-397	310-400
120	10.2	11.7	357-467	360-470
140	10.2	11.7	416-544	420-540

Note: 1 km/h = 0.62 mph; 1 m = 3.28 ft.

RECOMMENDED VISIBILITY REQUIREMENTS FOR REFLECTIVE DEVICES

From the visibility distances shown in Table 1, a performance standard for the reflective devices discussed here can be presented in a form appropriate for the Manual of Uniform Traffic Control Devices, such as:

The [barricade, panel, drum, cone] shall be installed and maintained so as to be visible at night under normal atmospheric conditions from a minimum distance of 275 m (900 ft) when illuminated by the low beams of standard automobile headlights.

The selection of 275 m seems reasonable, albeit somewhat arbitrary. It is nearly the midpoint of low-high values for 96.54 km/h (55 mph). This standard essentially ignores the fact that visibility requirements are less than 275 m at lower speeds. This is so because, for reasons of economy, the devices have been, and will continue to be, fabricated for use in all highway situations. Contractors or government agencies are not about to stockpile devices of varying reflectance qualities to be used at work-zone locations that vary by speed, environment, or any other variable.

This performance standard, although developed primarily for reflectance devices, should apply to any device used for channelization purposes in the work zone. This standard is not applicable to advance warning signs. Also, it should be considered a preliminary standard until further research is completed or until such time as this performance standard can be validated by field studies.

ACKNOWLEDGMENT

This paper is based on a study performed by BioTechnology for the Federal Highway Administration. The opinions, findings, and conclusions are ours and not necessarily those of the sponsor.

REFERENCES

1. G. J. Alexander and H. Lunenfeld. Positive Guidance in Traffic Control. Federal Highway Administration, Superintendent of Documents, Stock No. 050-001-00094, April 1975.
2. T. E. Post, H. D. Robertson, H. E. Price, G. J. Alexander, and H. Lunenfeld. A User's Guide to

Positive Guidance. Federal Highway Administration, Jan. 1977.
3. A Policy on Design Standards for Stopping Sight Distance. AASHO, Washington, DC, 1971.
4. H. W. McGee, W. Moore, B. G. Knapp, and J. H. Sanders; BioTechnology. Decision Sight Distance for Highway Design and Traffic Control Requirements. Federal Highway Administration, Jan. 1978.

Effects of Taper Length on Traffic Operations in Construction Zones

Jerry L. Graham, Douglas W. Harwood, and Michael C. Sharp, Midwest Research Institute, Kansas City, Missouri

The study dealt with a proposed taper length formula that yields shorter tapers at design speeds below 96 km/h (60 mph) than does the existing formula (L = WS, when S is in mph). This paper reports on a direct comparison of traffic operations using both the standard and proposed taper lengths in the same construction zones. Speed, erratic maneuvers, traffic conflicts, and lane encroachment data were collected at four sites, day and night, for a variety of design speeds and taper lengths. The analyses of the data collected do not imply that the proposed taper lengths are more hazardous than the standard taper length. Use of the proposed length did not produce a greater number of erratic maneuvers and slow-moving vehicle conflicts than did the standard or existing taper length. There was no indication that the proposed taper lengths resulted in a greater number of passenger vehicle or truck encroachments on adjacent lanes.

The Manual on Uniform Traffic Control Devices (MUTCD) (1) specifies that the length of lane-drop tapers in construction zones should be computed as

$$L = WS \quad (1a)$$

where

L = Minimum length of lane-drop taper (ft),
W = Width of offset (ft), and
S = Speed limit or 85th percentile speed (mph),

or for the metric computation,

$$L = WS/1.62 \quad (1b)$$

In application the speed (S) can be considered as the design speed of the construction zone (not necessarily that of the highway). The design speed is the maximum safe speed through the construction zone. An alternative formula has been proposed to replace the standard formula:

$$L = WS^2/60 \quad (2a)$$

or for the metric computation,

$$L = WS^2/157.5 \quad (2b)$$

A comparison of taper length computed by use of each of these formulas is shown in the table below (1 km/h = 0.62 mph; 1 m = 3.28 ft).

Design Speed (km/h)	Taper Length (m) Using L = WS/1.62 (W = 3.7 m)	Taper Length (m) Using $L = WS^2/157.5$ (W = 3.7 m)
96	220	220
89	201	185
80	183	152
72	165	123
65	146	98
56	128	75
50	110	55
40	91	38
32	73	24
25	55	14

At a design speed of 96 km/h (60 mph) a taper length of 220 m (720 ft) is computed using both formulas, at 72 km/h (45 mph) the taper length is 165 m (540 ft) using the standard formula and, using the proposed formula, 125 m (405 ft), only 75 percent as long as the standard taper length. At 50 km/h (30 mph) the standard taper length is 110 m (360 ft) and the proposed taper length is 55 m (180 ft), only 50 percent as long as standard; at 25 km/h (15 mph) the standard taper length is 55 m and the proposed taper length is 14 m (45 ft), 25 percent as long as the standard.

The proposed formula is theoretically appealing because the ability to stop and change direction is known to be inversely proportional to the square of the velocity. Therefore, if the standard taper length is adequate for 96 km/h (60 mph), then standard taper lengths for speeds less than 96 km/h are excessively long. Proponents of the revised formula point out the advantages of the shorter taper lengths: They require fewer traffic-control devices and, at urban sites, interfere with fewer driveways and intersections.

Opponents of the proposed formula believe that the taper lengths computed by the proposed formula are too short at low speeds [25 to 40 km/h (15 to 25 mph)] and that the short tapers are not sufficient to allow large vehicles such as trucks and buses to change lanes without encroaching on adjacent lanes and to prevent such large vehicles from turning over.

STUDY SITES

The alternative taper formulas were evaluated in four construction zones—one in Missouri and three in Florida. The design speeds of these four construction zones ranged from 25 to 72 km/h (15 to 45 mph). The characteristics of the four construction zones are described in Table 1.

Site 1 was studied in September 1976, in conjunction with earlier field work. These field studies considered the effects of funneling and reduction of lane width as

Table 1. Study site characteristics.

Site Number	Highway Type	Description of Traffic Control	Taper Initially Used	Design Speed (km/h)
1	4-lane undivided	Closure of right lane and diversion to two-lane detour roadway	Standard (MUTCD)	50
2	6-lane divided	Closure of right two lanes and crossover through median opening	Standard (MUTCD)	25
3	6-lane divided	Closure of right two lanes and crossover through median opening	Existing (shorter than proposed taper length)	50
4	6-lane divided	Closure of left two lanes	Standard (MUTCD)	72

Note: 1 km/h = 0.62 mph.

well as different taper lengths. Funneling is a technique of gradual reduction of the width of the traveled way by placing drums on each side of the open lanes on the approach to the construction zones. The table below shows the experimental design for the studies conducted at site 1. Experiments were conducted both day and night.

Experiment	Treatment	Level
US-1	Taper length	Proposed formula
	Funneling and lane width reduction	Not present
US-3	Taper length	Proposed formula
	Funneling and lane width reduction	Present
US-5	Taper length	Standard formula
	Funneling and lane width reduction	Not present
US-6	Taper length	Standard formula
	Funneling and lane width reduction	Present

Sites 2, 3, and 4 were studied in June 1977. Sites 2 and 4 involved direct comparison of the standard and proposed tapers without the consideration of other factors. Site 3 involved a unique situation. An existing 30-m (100-ft) median opening was used to cross traffic into the opposite roadway. The small median opening prevented the use of the standard taper and the existing taper was shorter than the proposed taper. At this site, the proposed taper was compared with the existing, shorter-than-proposed taper. The table below summarizes the experimental design for sites 2, 3, and 4 (1 km/h = 0.62 mph).

Site	Experiment	Design Speed (km/h)	Taper Formula
2	1	25	Proposed
2	2	25	Standard
3	1	50	Proposed
3	2	50	Existing condition
4	1	72	Proposed
4	2	72	Standard

FIELD STUDY PROCEDURE

The plans for the experiment were approved by the construction contractor and the cooperating state highway department before the studies began. Changes in the taper length were made only after the necessary approvals were obtained.

The first step in the field study was to install speed-measuring equipment. The basic mode of data collection used a series of tape switches connected to a 20-channel event recorder. Pairs of tape switches 30 to 15 m (100 to 50 ft) apart were placed in each lane at two locations in the zone. The tape switches were augmented by radar at a third location at sites 1, 3, and 4. The locations of the speed measurements included: (a) prior to the beginning of the zone, (b) on the approach to the zone, (c) in the entrance to the transition area, and (d) in the work area.

The approach area begins where the driver is first informed about the actual condition of the roadway ahead and the actions that will be required to travel through the work area. Although no physical restrictions narrow the roadway in the approach area, drivers often slow their vehicles and perform merging maneuvers as they adjust their speeds and positions based on their concepts of the safe path through the zone.

The transition area begins at the point where the normal roadway is altered laterally by devices such as cones, barricades, or barriers in order to channel traffic to the part of the roadway open through the work area.

The work area is that length of the roadway where work is being done or will be done. The work-area roadway may be completely closed to traffic or a portion of the roadway may be open through the work area. If the work area is open to traffic, traffic control should provide for the separation and protection of motorists and construction workers.

The switches were connected by wire to the event recorder. When a vehicle crossed the switch, the circuit was closed and vehicle passage was recorded on paper charts used in the event recorder. Almost 3 km (2 miles) of wire were required to connect the switches to the recorder. To reduce the quantity of wire required, speeds in one area of the zone were measured by radar. Three-meter (10-ft) tape switches placed perpendicular to the lane were used to record lane volumes, speeds, and headways. In the transition area of sites 1, 2, and 3, 0.6-m (2-ft) switches laid end to end were used to record information on the lateral placement of vehicles. Data from these switches were used to determine vehicle encroachments on the highway centerline.

The tape switches were installed by the project crew. During installation of the switches, traffic was controlled by a flagman and a sequential flashing-arrow trailer. The switches were secured to the pavement by duct tape. When the switches were in place and tested, the study began. Each experiment was conducted for day and night conditions. The daytime experiments were conducted from 12:00 n. to 5:00 p.m. Night studies were conducted between 7:00 p.m. and 12:00 m.n.

Two observers were present during each of the experiments. One of the observers made the radar speed measurements. The other observer was stationed in the transition area of the zone and recorded vehicle conflicts and erratic maneuvers. He also noted (on the event recorder by a special switch) the passage of a bus, which would assist in later data reduction. The erratic maneuver and conflict counts and the radar speed measurements were made for 15-min periods. The length of each experiment (day or night) was 2.5 to 3 h, which was sufficient to obtain at least ten 15-min periods of conflict data. This length of study period for conflicts data is equivalent to the conventional sample for intersectional conflict counts.

Five types of conflicts plus erratic maneuvers were monitored at each site, although all were not analyzed at each site because of the small numbers encountered. A slow-moving-vehicle conflict occurs when a vehicle is forced to brake or swerve to avoid a rear-end collision with a slower vehicle in the transition area. A weave conflict occurs when a vehicle changes lanes into the path of another vehicle, which causes the offended vehicle to brake or swerve. A slow-to-weave conflict occurs when a vehicle must brake or swerve to avoid another vehicle while changing lanes. A right-turn conflict occurs when a vehicle must brake or swerve to avoid collision with a vehicle that is turning right. A previous conflict occurs when a vehicle is forced to brake or swerve to avoid collision with another vehicle and in so doing causes a third vehicle to brake or swerve. An erratic maneuver occurs when a single vehicle brakes or swerves on the approach to the transition area. Unlike a conflict, an erratic maneuver does not require the presence of a second vehicle that causes the braking or swerving maneuver.

The observed sample sizes and speed measurements of vehicles are given below (the number of buses at site 1 was not noted separately):

Site	Trucks	Buses	Total	Number of Vehicle Speed Measurements
1	197	–	9120	5928
2	55	8	3076	1393
3	16	9	4751	3045
4	72	0	2772	2634

DATA REDUCTION

The field data were reduced by reading the paper charts from the 20-pen event recorder. The data determined from the charts included traffic volumes, traffic speeds, and vehicle classifications.

Traffic volumes were determined for each 15-min period by counts of the number of times the tapeswitch was actuated. Separate switch actuations for each axle make it possible to classify vehicle types as truck (three or more axles) and passenger automobile or bus (two axles). Buses were distinguished from passenger automobiles at sites 2, 3, and 4 by a manual actuation of one recorder pen for each bus that passed through the taper area.

Speed measurements from the event recorder charts were made by use of an overhead opaque projector to show the chart image on a rear-projection screen. The projected image was enlarged four times to permit accurate measurement of the distance on the chart that represents the time between closures of switches spaced at a known 30-m (100-ft) interval.

ANALYSIS AND RESULTS

Four general measures of effectiveness were considered in the taper length studies: speeds, traffic conflicts, erratic maneuvers, and centerline encroachments. The mean speeds, erratic maneuver rates, traffic conflict rates, and encroachment rates for each experiment are presented in Table 2. The statistical analyses and the evaluation of the relationship between each measure of effectiveness and taper length are summarized below. For further details of the analyses, refer to the recent report by Graham and Sharp (2).

Speeds

Site 1 involved a closure of the right lane of an urban, four-lane, undivided arterial street. Speeds at site 1 were measured at three locations: location 1 (L1) was 0.8 km (0.5 mile) upstream of the lane closure, location 2 (L2) was 310 m (1000 ft) upstream of the lane closure, and location 3 (L3) was at the center of the barrel taper. At this site the effect of taper length was studied in conjunction with the effect of the presence or absence of funneling and lane-width reduction. These effects were separated by an analysis of variance of the following factors and levels (1 km = 0.62 mile; 1 m = 3.28 ft):

Factor	Level
Taper length (T)	T1 = Proposed formula T2 = Standard formula
Funneling and lane width reduction (F)	F1 = Not present F2 = Present
Time of day (t)	t1 = Day t2 = Night
Location (L)	L1 = 0.8-km upstream L2 = 310-m upstream L3 = Center of taper

The results of this analysis of variance are given in Table 3. This table illustrates the format of the analysis of variance results that were obtained throughout the study. The location effect was highly significant, which indicates that (as expected) vehicle speeds decreased on the approach to the taper. However, neither the taper length effect nor any of the other effects or interactions shown in Table 3 had a significant effect on vehicle speed.

Similar speed data were obtained at site 2, except that the influence of funneling and lane-width reduction was not considered. The construction activity at site 2 involved a lane closure followed by a traffic diversion (crossover) through a median opening of a multilane, divided urban arterial street. Speed data were collected at two locations in the crossover taper: L1 = 8 m (25 ft) from the beginning of the crossover taper and L2 = 23 m (75 ft) from the beginning of the crossover taper. An analysis of variance of the factors' taper length, time of day, and speed-measurement location found that the mean speed when the proposed taper was used was significantly higher than the mean speed when the standard taper was used. However, the absolute difference in mean speeds was less than 1.6 km/h (1 mph) [39 km/h (24.40 mph) versus 37 km/h (23.38 mph)].

At site 3, the proposed taper was compared with the existing, shorter-than-proposed taper. The construction activity at site 3 was similar to that at site 2, which involves a crossover through a median opening of a multilane, divided urban arterial street. Speed measurements were made at three locations: L1 was approximately 152 m (500 ft) before the taper, L2 was approximately 131 m (430 ft) before the taper, and L3 was at the crossover point. An analysis of variance of the speed data found that the three major factors (taper length, time of day, and location of speed measurement) were all significant, as were several of the interaction terms. The speeds with the existing shorter-than-proposed taper were significantly greater than the speeds with the proposed taper [55 km/h versus 59 km/h (34.33 mph versus 36.53 mph)]. The other significant factors and interactions indicate that the pattern of speed change on the approach to the zone is influenced by both the taper length and the time of day.

A final comparison of speeds for the standard and proposed taper lengths was made at site 4. This site involved the closure of the two left lanes of one direction of travel on a 6-lane divided arterial street. Speed measurements were made at three locations: L1 was approximately 275 m (900 ft) before the taper, L2 was near

Table 2. Summary of results.

Site	Design Speed (km/h)	Experiment	Time of Day	Mean Speed (km/h) Location 1	Location 2	Location 3	Erratic Maneuver Rate (Erratic Maneuver/100 Vehicles)	Conflict Rates (conflicts/100 vehicles) Slow Moving Conflicts	Previous Conflicts	Slow-To-Weave Conflicts	Right Turn Conflicts	Weave Conflicts	Encroachment Rate (Encroachments/100 Vehicles)
1	50	US-1—Proposed taper length, no funneling or lane width reduction	Day	71.0	66.3	56.2	0.19	0.19	8.00*	1.52*	0.13*	1.33*	NA
			Night	67.8	65.5	54.9	0.097	0.29	0*	0.68*	0.19*	1.84*	NA
		US-3—Proposed taper length, funneling and lane width reduction present	Day	71.3	66.8	52.8	7.63	7.02	1.49*	1.96*	0*	1.22*	NA
			Night	70.0	62.8	52.6	18.06	4.46	3.08*	2.44*	0*	0.96*	NA
		US-5—Standard taper length, no funneling and lane width reduction	Day	71.3	67.8	58.9	2.06	7.62	1.03*	0.79*	0*	1.11*	NA
			Night	70.0	66.3	57.6	8.68	5.71	0.95*	0.95*	0*	0.12*	NA
		US-6—Standard taper length, funneling and lane width reduction present	Day	69.7	66.0	54.9	4.79	7.14	2.00*	1.05*	0.17*	0.61*	NA
			Night	70.0	67.1	55.0	11.7	6.67	1.55*	0.48*	0*	0*	NA
2	45	1—Proposed taper length	Day	39.1	39.1	NA	7.46	2.99	1.09	3.49*	0.73*	3.17*	0.08*
			Night	39.3	39.6	NA	9.93	2.20	1.06	2.99*	0*	1.87*	0.56*
		2—Standard taper length	Day	36.2	38.0	NA	4.58	5.13	0.74	0*	0.96*	0*	0*
			Night	37.8	38.5	NA	9.12	4.21	0.90	0*	0.76*	0*	1.14*
3	50	1—Proposed taper length	Day	61.0	56.0	48.8	1.25	4.23	0.76	4.23	0.21	2.36*	0*
			Night	60.0	52.3	45.7	2.23	3.55	1.11	3.85	0.20	1.93*	0*
		2—Existing taper length	Day	57.0	62.0	48.5	13.58	3.78	1.22	0.07	0.54	0*	0*
			Night	58.6	58.7	46.8	12.72	4.16	0.59	0.12	0.12	0*	1.43*
4	72	1—Proposed taper length	Day	68.6	74.0	62.3	1.68	1.85*	0.43*	0.45	0*	0.96	NA
			Night	70.5	72.9	63.7	1.83	1.69*	0*	0.87	0*	0.19	NA
		2—Standard taper length	Day	71.8	74.7	59.6	2.16	2.03*	0.63*	1.34	0*	0.61	NA
			Night	69.5	69.4	60.2	3.26	1.70*	0.19*	0.75	0*	0.39	NA

Note: 1 km/h = 0.62 mph.
*These data were not included in the analyses of variance.

Table 3. Site 1: analysis of variance of speeds.

Source	df	Sum of Squares	Mean Squares	F-Ratio
Time (t)	1	4.17	4.17	9.07
Taper (T)	1	4.00	4.00	8.70
Funneling (F)	1	3.84	3.84	8.35
Location (L)	2	345.64	172.82	375.70*
tT	1	0.88	0.88	1.91
tF	1	0.32	0.32	<1
tL	2	0.81	0.41	<1
TF	1	0.14	0.14	<1
TL	2	2.43	1.22	2.65
FL	2	3.88	1.94	4.22
tTF	1	0.74	0.74	1.61
tTL	2	0.42	0.21	<1
tFL	2	0.34	0.17	<1
TFL	2	0.48	0.24	<1
Residual (tTFL)	2	0.91	0.46	

*Significant at the $\alpha = 0.05$ level.

the beginning of the taper, and L3 was near the end of the taper. In the analysis of variance, all three factors were significant, as were all three two-way interactions. The proposed taper speeds were significantly greater than the standard taper speeds [67 km/h versus 68 km/h (41.95 mph versus 42.67 mph)]. Again, the absolute difference in speeds between the two tapers was less than 1.6 km/h. In addition, the significant difference was due almost entirely to the speeds at location L3. The speeds for the standard and proposed tapers were statistically indistinguishable at locations L1 and L2.

Traffic Conflicts and Erratic Maneuvers

A comparison of the slow-moving-vehicle conflict rates for the standard and proposed taper lengths was made at site 1. The following table illustrates the comparative conflict rates per 100 vehicles at this site. The analysis found extremely low conflict rates both during the day and at night for the proposed taper formula in the absence of funneling or lane-width reduction. The other combinations tested (which included either the standard taper formula or the presence of funneling or both) had much higher traffic conflict rates.

Taper Length	Conflicts/100 Vehicles Day	Night
Proposed formula, no funneling	0.19	0.29
Proposed formula, funneling	7.02	4.46
Standard formula, no funneling	7.62	5.71
Standard formula, funneling	7.14	6.67

The erratic maneuver rates measured at site 1 are summarized below. The results of the erratic maneuver analysis are similar to the results of the conflict analysis for site 1. The erratic maneuver rate is very much lower for the proposed taper length in the absence of funneling than for any other combination.

Taper Length	Erratic Maneuvers/100 Vehicles Day	Night
Proposed formula, no funneling	0.19	0.10
Proposed formula, funneling	7.63	18.07
Standard formula, no funneling	2.06	8.68
Standard formula, funneling	4.79	11.68

At site 2, the standard and proposed tapers were compared with respect to both slow-moving-vehicle and previous conflict rates. An analysis of variance found no significant difference in previous conflict rate between the standard and proposed taper lengths, but the slow-moving-vehicle conflict rate was significantly greater for the standard taper length than for the proposed taper length.

The erratic maneuver rate at site 2 was found to be significantly higher at night than during the day. However, no statistically significant difference was found between the erratic maneuver rates for the standard and proposed taper lengths.

At site 3, the existing shorter-than-proposed taper length was compared with the proposed taper length with respect to slow-moving-vehicle conflicts, slow-to-weave conflicts, right-turn conflicts, and previous conflicts. The only statistically significant difference was found for the slow-to-weave conflicts, which were significantly higher with the proposed taper than with the shorter taper [4.0 versus 0.1; $F(1, 26) = 75.34$]. In contrast, the erratic maneuver rate was much greater with the existing taper than with the proposed taper [13.6 versus 1.7; $F(1, 26) = 205.06$]. It appears that at this site the proposed taper length eliminates erratic maneuvers but only at the expense of causing heretofore nonexistent slow-to-weave conflicts.

The standard and proposed taper lengths were compared at site 4 with respect to slow-to-weave conflicts, weave conflicts, and erratic maneuvers. No statistically significant differences were found.

Encroachment Rates

The effect of taper length on vehicle encroachments on the highway centerline was investigated at sites 2 and 3. After the proposed taper length was in place at site 2, 36 trucks and 1767 passenger vehicles were observed. Of these vehicles, only 1 truck and 4 passenger vehicles encroached on the highway centerline. During the daytime testing of the standard taper length at site 2, no encroaching passenger vehicles or trucks were recorded on the placement switches. During the night experiments, the lateral placement switches were not in use, but random visual observations of 200 vehicles revealed that approximately 4 percent of the vehicles traveled on or over the centerline. No encroaching trucks or buses were observed.

At site 3, no encroaching vehicles were observed during the daytime or nighttime periods when the proposed taper length was in place or during the daytime when the existing, shorter-than-proposed taper length was in place. During the nighttime sampling period of the existing taper length, 12 of 841 passenger vehicles were recorded encroaching on the centerline. None of the trucks and buses observed at this site was recorded as encroaching.

SUMMARY AND CONCLUSIONS

The study presented in this paper dealt with a proposed new taper length formula that yields shorter tapers than the standard formula, $L = WS$. Concern has been expressed that the new formula would result in more hazardous traffic operations. Speed and other measurements were performed at four sites, day and night, for a variety of design speeds and taper lengths. Altogether, nearly 20 000 vehicles (including 340 trucks and 17 buses) were observed and 13 000 speed measurements were obtained.

In general, speeds were slightly higher for the shorter-length tapers. At site 1, speeds did not differ significantly between the two tapers. At site 3, the very short taper produced significantly higher speeds than did the proposed taper. The speeds with the very short taper also showed a sudden decrease near the end of the taper. The more moderate decrease in speeds for the longer taper lengths could be associated with the fact that at sites 2 and 3 the longer tapers restricted traffic to one lane sooner than did the shorter tapers. In all cases, where a significant difference in traffic speeds was found, the absolute difference in mean speed was less than 1.6 km/h (1 mph).

Results of the erratic maneuver analysis were mixed. At site 2, the erratic maneuver rate did not vary between the standard and proposed taper lengths. At site 3, the very short taper had a higher erratic maneuver rate than did the proposed taper length. This result is compatible with the sudden drop in speed observed at site 3. At site 1, the taper effect was dependent on both the level of funneling and time of day. No site showed that the proposed taper created more erratic maneuvers than did the standard-length taper.

At site 1, only the proposed taper length in combination with the absence of funneling depressed the slow-moving conflict rate. At site 2, the slow-moving conflicts were greater under the standard taper length. This result is compatible with the lower speeds under the standard taper. Sites 3 and 4 showed no significant effects on slow-moving conflict rates. No site showed that the proposed taper created more slow-moving conflicts than did the standard-length taper.

Only site 3 showed a significant slow-to-weave conflict rate effect. Use of the proposed taper length rather than the shorter taper length increased slow-to-weave conflicts. This result may have been due to the fact that one more lane was closed during the proposed taper experiment than during the existing condition experiment. Also, at site 3, the increase in slow-to-weave conflict rate was accompanied by a significant decrease in erratic maneuver rate.

The placement switches at sites 2 and 3 did not indicate that trucks or buses were encroaching on adjacent lanes under the proposed taper. The number of encroaching passenger vehicles did increase under the very short existing taper at site 3 during the night measurements. No encroaching vehicles were observed during the day measurements, but 12 of 841 vehicles were observed encroaching during the night measurements.

In summary, the analyses do not imply that the proposed taper lengths are more hazardous than the standard taper lengths. In no instance was the erratic maneuver rate significantly higher with the proposed taper than with the standard or existing taper and at one site it was less (this site had an existing shorter-than-proposed taper). Likewise, slow-moving conflict rates were never greater with the proposed taper. Only at one site were slow-to-weave conflicts higher under the proposed as compared with the existing taper—at the site with the shorter-than-proposed taper. At three sites average speeds differed significantly with taper length, but by small magnitudes. At each of these three sites, speeds were higher when the shorter tapers were used. There was no indication that the proposed taper lengths resulted in a greater number of passenger vehicle or truck encroachments on adjacent lanes.

As a result of this field evaluation of the operational effects of taper length, the Federal Highway Administration (FHWA) and the National Advisory Committee on Uniform Traffic Control Devices (NAC) have approved the proposed taper formula for inclusion in the MUTCD (3). The proposed taper formula [$L = WS^2/157.5$ ($L = WS^2/60$, when S is in mph)] should be used to compute taper length on urban, residential, and other streets where the posted speeds are 65 km/h (40 mph) or less. The standard taper length formula is retained for freeways, expressways, and all other roadways having a posted speed of 72 km/h (45 mph) or greater. Sections 3B-4, 3B-8, 3B-13, and 6C-2 of the MUTCD will be revised accordingly.

ACKNOWLEDGMENT

This paper is based on a study conducted by Midwest Research Institute as part of a study for the Federal Highway Administration. The findings and conclusions of this paper are our own, however, and do not necessarily represent the views of the Federal Highway Administration.

REFERENCES

1. Manual on Uniform Traffic Control Devices for Streets and Highways. U.S. Department of Transportation, 1971.
2. J. L. Graham and M. C. Sharp. Effects of Taper Length on Traffic Operations in Construction Zones. Federal Highway Administration, Rept. FHWA-RD-77-162, Dec. 1977.
3. M. R. Norman. Traffic Control Devices. ITE Journal, Vol. 48, No. 7, July 1978, p. 50.

Effect of Longitudinal Edge of Paved Surface Drop-Offs on Vehicle Stability

Roger L. Stoughton, Douglas M. Parks, J. Robert Stoker, and Eric F. Nordlin,
Division of Construction, California Department of Transportation

The effect of edge of pavement drop-offs on vehicle stability is reported for 50 tests of professional drivers handling small-, medium-, and large-sized automobiles and pickup trucks off, along, and back onto drop-off heights of 38 mm (1.5 in), 89 mm (3.5 in), and 114 mm (4.5 in) at about 26.8 m/s (60 mph). Tests of two- and four-wheel drop-offs were conducted from an existing asphalt concrete shoulder onto both compacted soil and asphalt concrete surfaces. The drop-off heights had little effect on vehicle stability: steering wheel angles were generally 60° or less; vehicle roll angles were 10° or less. A significant jolt and accompanying front-end noise were experienced by the driver at the larger drop-off heights; there were no problems with vehicle alignment. Less than one wheel revolution was required for the first wheel to mount the drop-off heights. Varying amounts of front-wheel wobble caused mainly by an irregular drop-off edge were detected. There was virtually no deviation in vehicle trajectory as the vehicles remounted the drop-off edges, and the vehicles did not encroach into adjacent traffic lanes. Two nonprofessional drivers participated in a few supplementary tests. They had no difficulties driving over all three drop-off heights at 17.9-20.1 m/s (40-45 mph). The results of these tests were used to help evaluate the California maintenance standards in effect in 1974.

In 1974, the California Department of Transportation studied some highway accident cases in which a drop-off at the longitudinal edge of pavement was cited as a possible contributing factor.

This project was initiated

1. To determine the effects of longitudinal drop-offs along a highway and on the stability and controllability of vehicles traveling over the drop-offs at high speeds,
2. To establish maximum tolerable heights for drop-offs,
3. To verify current maintenance standards for allowable drop-off heights.

No attempt was made to study the surprise element in driver reactions to an unexpected drop-off condition.

A longitudinal drop-off exists along a highway when there is a difference in height between two adjacent surfaces, either between

1. Surfaces of a paved shoulder and the unpaved area alongside it,
2. Surfaces of a paved traveled way and an unpaved shoulder,
3. Surfaces of a paved traveled way and a paved shoulder, or
4. Surfaces of a portion of an existing traveled way with a newly paved blanket overlay and the remaining portion of the existing pavement.

Drop-offs created during construction, when new traffic lanes are added to existing traveled ways, were not considered for this study. These drop-offs generally exceed the maximum heights of 114 mm (4.5 in) used for this project, and sometimes approach several meters, depending on soil conditions at the construction site.

Drop-offs are generally caused by erosion and traffic wear. However, during a pavement blanket overlay operation, a drop-off is frequently caused because the paving equipment cannot pave the full width of the traveled way or traveled way and shoulder at one time. There is often a delay before all of the existing pavement can be brought up to the grade of the new pavement blanket.

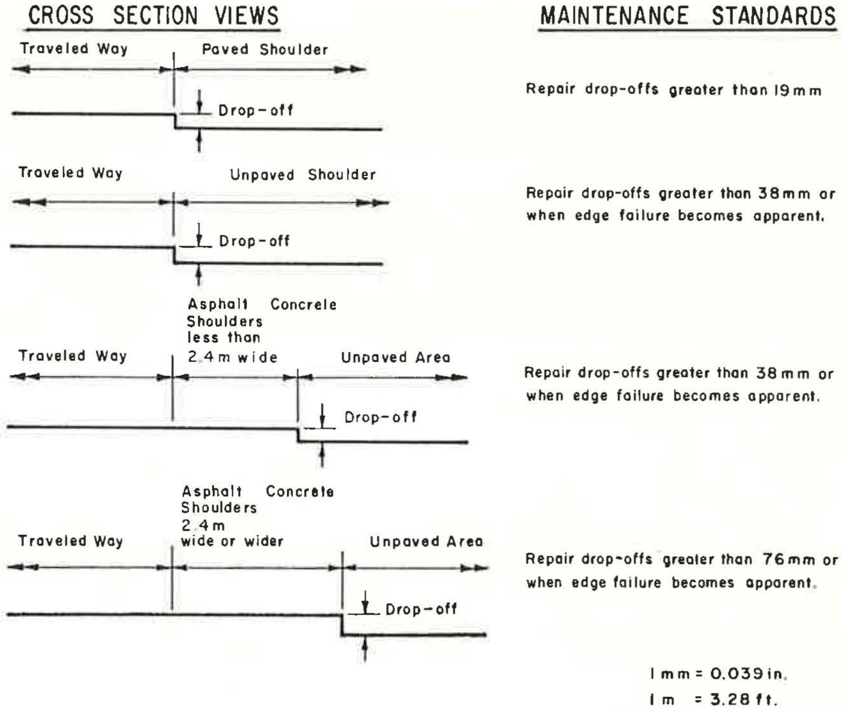
Portions of the California Department of Transportation maintenance manual dated May 15, 1974, specified California's drop-off standards and are illustrated in Figure 1.

The highway departments from the states of Illinois, New York, Oregon, Texas, and Washington were contacted during the course of this project for their allowable drop-off standards and accident experience records. New York permitted drop-off heights ranging from 25 mm (1 in) maximum for expressways with volumes over 500 vehicles/h to 51 mm (2 in) maximum for state highways having one-way design volumes of less than 200 vehicles/h. The other states either had no published standards, required shoulders to be flush with the traveled way, or allowed maximum drop-offs of 51-76 mm (2-3 in). Only Oregon had accident records related to drop-off conditions. The records from Oregon combined all accidents due to chuckholes and drop-offs.

A Highway Research Information Service (HRIS) literature search was made prior to the initiation of this project. Before 1974, none of the research reported had been conducted to determine whether longitudinal drop-offs cause vehicle stability problems.

Full-scale tests have been conducted by the California Department of Transportation (1, 2) and the Texas Transportation Institute (3) on the effects of vehicles climbing up over curbs at various angles. These tests were conducted on curbs with heights ranging from 152-305 mm (6-12 in) and also included a few tests over a sloping 102-mm (4-in) high curb. It was concluded that these tests did not apply to drop-off conditions of interest in this study, which was concerned with near-vertical drop-off heights less than 125 mm (5 in).

Figure 1. 1974 maintenance standards in California.



Fifty tests, using professional drivers, were conducted to investigate the following basic parameters:

1. Drop-off heights of 38 mm (1.5 in), 89 mm (3.5 in), and 114 mm (4.5 in);
2. Four different vehicles—a small-, medium-, and large-sized automobile and a pickup truck;
3. Vehicles driven by a professional driver from an existing asphalt-concrete (AC) shoulder onto either an AC or a soil surface and returned to the AC shoulder at velocities of 26.8 m/s (60 mph) and angles less than 10°; and
4. Tests with either two wheels of the vehicle or four wheels of the vehicle dropping off an existing AC shoulder.

The driver, a former race-car driver, is a private consultant who conducts vehicular impact tests and other automotive research.

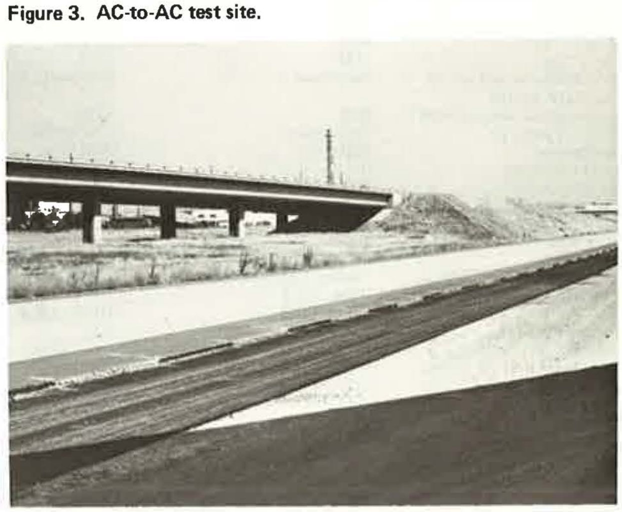
TEST SITE LOCATION AND CONSTRUCTION

The test site was located on an unopened portion of I-80 between Del Paso Park Separation and Overhead and Longview Drive Overcrossing in Sacramento County near Sacramento, California (Figure 2).

Drop-off heights of 114 mm (4.5 in), 89 mm (3.5 in), and 38 mm (1.5 in) were constructed along the edge of an existing 1.5-m (5-ft) wide AC shoulder adjacent to a 15.3-m (50-ft) wide unpaved median. Each drop-off height was maintained for a 153-m (500-ft) length with short spaces between the three 153-m test strips. Field measurements of drop-off heights were taken at 3.1-m (10-ft) intervals. Each 153-m strip was used for both series of tests, asphalt-to-soil and AC-to-AC. After the AC-to-soil tests were completed, an additional 25-51 mm (1-2 in) layer of soil was removed from each strip and replaced by a layer of AC so that the AC-to-AC drop-off tests could be conducted (Figure 3). Originally it was planned that a 140-mm (5.5-in) drop-off height be used. However,

Figure 2. Test site.

Figure 3. AC-to-AC test site.



due to the 147-mm (5.8-in) minimum ground clearance on the small automobile we decided that 114 mm was the maximum height that could be used without the automobile bottoming out on the edge of pavement at the drop-off. The longitudinal profile grade for the portion of I-80 used for this project was 0.54 percent, or nearly level.

Two control tests were conducted at sites where there were no drop-offs. Test 39, with a medium-sized vehicle, was performed entirely on the existing portland cement concrete (PCC) pavement adjacent to the drop-off test sites. Test 45, with a large-sized vehicle, was conducted entirely on soil on the other side of the median adjacent to the 38-mm drop-off site.

The tests were conducted from September to October 1974. The test strips were dry and the weather was good for all tests. Figure 4 shows a layout of the test site, test-site widths, and typical cross sections for the existing roadway used for this project.

TEST EQUIPMENT AND PROCEDURE

Four different types of vehicles were used for the test series. The vehicle specifications are included in Table 1. Each vehicle was tuned and aligned before

Figure 4. Test site and typical camera layout.

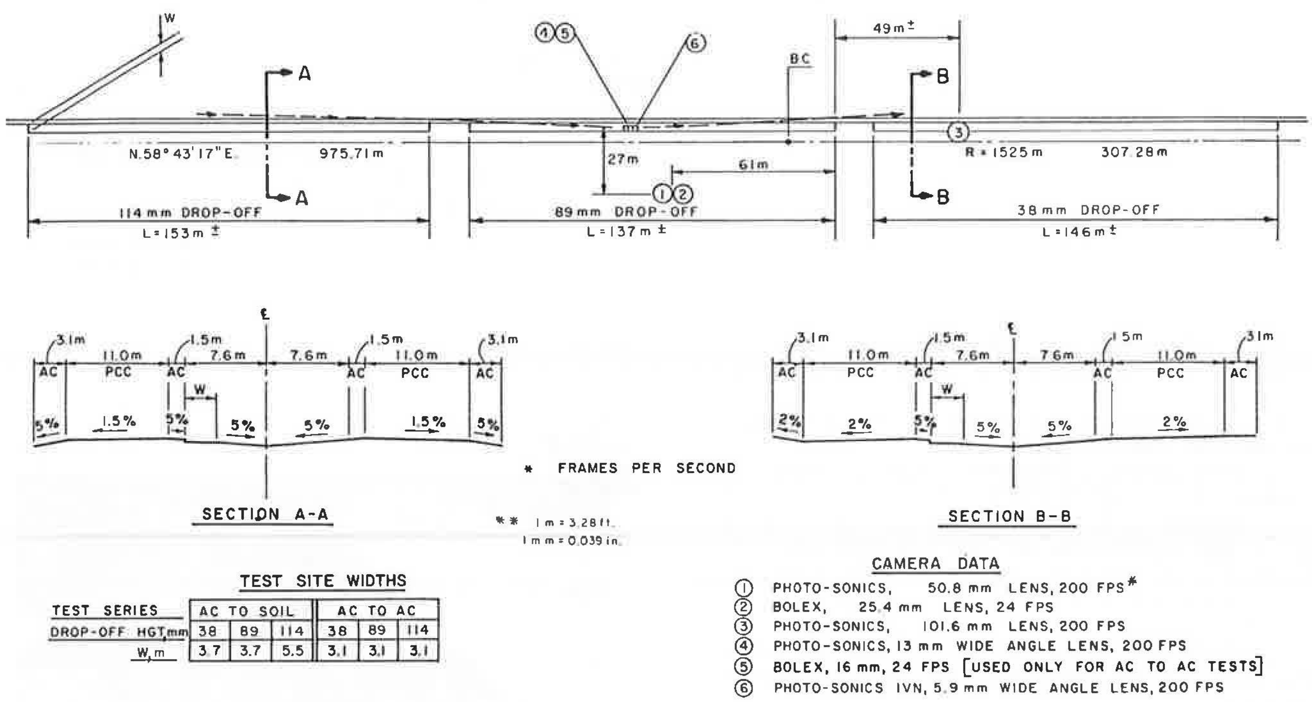


Table 1. Vehicle specifications.

Feature	Automobile: Small	Medium	Large	Pickup Truck
Year	1971	1971	1970	1973
Make	Ford	American Motors	Chevrolet	Dodge
Model	Pinto	Matador 4-door sedan	Brookwood station wagon	D100 454 kg
Mass[a] (kg)	1144	1743	2170	1851
Transmission and no. of forward speeds	Automatic 3	Automatic 3	Automatic 3	Automatic 3
Engine displacement (cm^3)	2000	4980	5740	5210
Shock absorbers	Telescoping	Telescoping	Telescoping	Telescoping
Suspension	Ball joint	Ball joint	Ball joint	Ball joint
Power steering	No	Yes	Yes	No
Steering ratio[b]	22.1	19.4	19.3	30.0
Brake type/power	Drum/no	Drum/no	Drum/yes	Disc, front and drum, rear/yes
Air conditioner	No	No	Yes	Yes
Tire size	B78×13	E78×14	H78×15	G78×15
Tire type	B. F. Goodrich custom long miler 4 ply polyester	B. F. Goodrich Silvertown HT 4 ply polyester	B. F. Goodrich Silvertown HT 4 ply polyester	Goodyear custom belted 2+2
Average tread depth (mm)	RF 6, LF 8 RR 8, LR 8	RF 6, LF 4 RR 9, LR 8	RF 10, LF 10 RR 8, LR 6	RF 8, LF 8 RR 7, LR 6
Recommended tire pressure (kPa)	221	221	221	221
Wheelbase (m)	2.29	3.00	3.02	3.00
Front tread (m)	1.37	1.53	1.61	1.68
Rear tread (m)	1.40	1.53	1.61	1.63
Distance (km)	65 092	77 629	110 048	13 713
Minimum ground clearance (mm)	147	178	203	203

Note: 1 mm = 0.039 in; 1 m = 3.28 ft; 1 km = 0.62 mile; 1 cm^3 = 0.06 in^3; 1 kg = 2.21 lb; 1 kPa = 0.145 lbf/in^2.

[a] Mass includes 91 kg for the driver and 100 kg of instrumentation. [b] Overall.

being used for the drop-off tests. The alignment was checked after each test run by measuring the wheel track of the vehicles with an adjustable gauge. Toe-in and toe-out alignment problems could be detected by this method. These problems are early indicators of more extensive alignment problems.

The sidewalls of the tires on the test vehicles were painted before each drop-off test so that tire scuff marks caused by the interaction of the tire with the drop-off edge could be photographed. Tire pressure was checked before each test day and was kept at recommended levels. A gravity-flow drip system delineated the path of the right rear wheel of the vehicle with a colored dye for each drop-off test.

Figure 5. Vehicle interior showing taped steering wheel and large speedometer.

Figure 6. Large-sized vehicle.

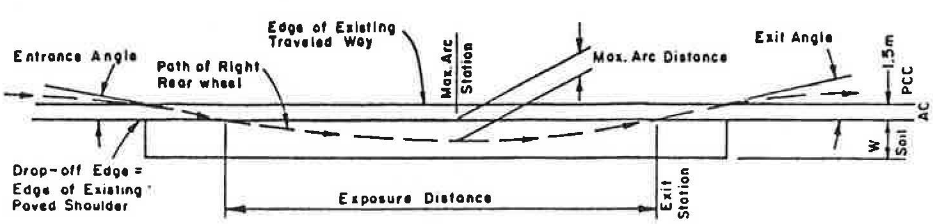
Table 2. Trajectory measurements—AC-to-soil drop-off test series.

Nominal Drop-Off Height (mm)	No. of Wheels Dropping Off	Test No.	Vehicle Size[a]	Vehicle Trajectory: Entrance Angle (degrees)	Max. Arc: Distance (m)	Max. Arc: Station (m)	Exit Angle (degrees)	Exit Station (m)	Exposure Distance (m)
38	2	7	S	3.2	1.1	58	3.4	93	82
		1	M	3.4	0.9	31	1.1	95	100
		17	L	2.3	1.0	55	2.9	91	80
		23	P	2.0	1.4	55	2.4	99	85
	4	8	S	4.0	2.2	67	5.2	113	90
		2	M	4.0	2.6	70	4.0	133	130
		18	L	2.6	3.1	61	3.1	120	110
		24	P	4.3	3.1	61	3.1	131	131
89	2	10[b]	S	4.6	1.0	37	5.7	69	58
		10[c]	S	4.6	0.7	46	4.9	70	59
		10	S	3.1	0.8	37	3.7	69	52
		4	M	4.9	1.1	58	4.3	90	66
		16	L	4.6	1.1	37	2.9	78	74
		22	P	4.6	1.2	46	4.3	99	85
	4	9	S	3.7	2.4	55	4.6	102	85
		3	M	4.0	2.7	46	4.0	107	88
		15	L	2.3	2.0	64	2.3	102	92
		21	P	5.4	3.1	58	2.9	102	88
114	2	11[d]	S	4.7	1.4	37	7.8	56	56
		5	M	5.2	0.5	88	4.6	104	87
		13	L	4.0	1.2	4.0	2.9	110	102
		19[d]	P	4.6	1.7	37	4.2	70	60
	4	12	S	4.6	2.8	67	4.0	120	103
		6	M	3.5	2.6	43	1.4	96	89
		14	L	4.0	2.9	49	2.9	120	107
		20	P	4.6	3.2	58	4.0	120	110
0	4	45[e]	L	6.8	2.6	64	1.4	118	94

Note: 1 mm = 0.039 in; 1 m = 3.28 ft.

[a]S = small automobile; M = medium automobile; L = large automobile; P = pickup truck.
[b]No camera coverage.
[c]No camera coverage of driver.
[d]Three wheels dropped off.
[e]Control test.

Figure 7. Trajectory measurements—AC-to-soil drop-off test series.

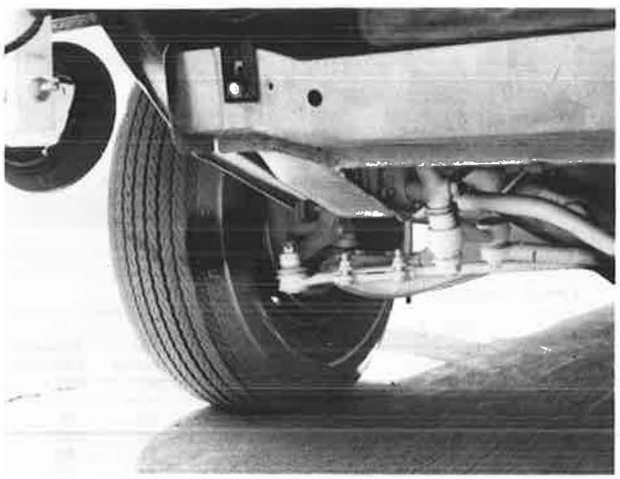


Note: 1 m = 3.28 ft; 1 mm = 0.039 in; W = 3.7 m for 38-mm and 89-mm sites; and W = 5.5 m for 114-mm site.

The perimeter of the steering wheel in each test vehicle was taped every 15°. A black vertical reference line was marked on the white background of a sheet-metal angle bracket taped to the dashboard of the vehicles. When the interior camera was bore-sighted, the vertical reference line was adjusted to line up with the tape on the steering wheel corresponding to a zero steering wheel angle. These taped angle markings were used to measure the angles through which the steering wheel was turned during each test. An interior view of the automobile is shown in Figure 5.

A typical view of a test vehicle straddling a drop-off edge is shown in Figure 6. Entrance angles (Table 2, Figure 7) were purposely small to simulate a driver drifting off the edge of the traveled way. Curb jump tests (1-3) showed that vehicles easily traverse curbs 152 mm (6 in) high and greater when impacting at high speeds and larger angles.

Figure 8. Bumper-mounted camera.



Four high-speed movie cameras and a normal-speed movie camera were used to document each drop-off test. The camera positions are shown in Figure 4. Cameras 1 and 2 were mounted on the ground and panned the action. Camera 3 was mounted on the ground downstream of the test and viewed the action parallel to the drop-off edge. Camera 4 was mounted inside the vehicle to view the driver, the rotation of the steering wheel, and a large speedometer mounted on the dash (Figure 5). Camera 6 was mounted on the front bumper of each vehicle to view the action of the vehicle's right-front or left-front wheel as the wheel dropped off and then mounted the drop-off edge (Figure 8). This camera was moved from the right side to the left side of the vehicle, depending on whether two- or four-wheel drop-off tests were conducted. Camera 5, mounted inside the vehicle, viewed the driver and steering wheel rotation for the AC-to-AC drop-off tests in addition to the other cameras.

Over 3350 m (11 000 ft) of movie film was exposed during the tests. Selected tests have been incorporated in a 30-min silent film report, which summarizes the test series.

TEST RESULTS

Test parameters, trajectory measurements, maximum vehicle roll angles, maximum steering wheel angles, and vehicle velocities are tabulated for the AC-to-soil drop-off tests in Tables 2, 3, and 4 and Figures 9 and 10. Data for the AC-to-AC drop-off tests were similar and are not included in this paper, but are included elsewhere (4).

Steering wheel angle (SWA) (Table 4) is defined as the angular displacement of the steering wheel measured from the straight-ahead position (position corresponding to zero average steer angle of a pair of steered wheels) (5).

Table 3. Vehicle roll angles—AC-to-soil drop-off test series.

Nominal Drop-Off Height (mm)	No. of Wheels Dropping Off	Test No.	Vehicle Size[a]	Vehicle Roll Angles (degrees): Going Off Drop Off	Coming Back on Existing Paved Shoulder	After All Wheels on Traveled Way
38	2	7	S	5	4	0
		1	M	3	3	0
		17	L	3	2	-1
		23	P	4	4	0
	4	8	S	5	6	–[b]
		2	M	3	3	-1
		18	L	3	3	-2
		24	P	4	4	0
89	2	10[c]	S	6	9	0
		10[d]	S	7	7	-1
		10	S	7	8	-1
		4	M	7	7	-2
		16	L	6	6	-2
		22	P	5	6	-1
	4	9	S	7	9	0
		3	M	5	7	-1
		15	L	6	8	-3
		21	P	5	6	-1
114	2	11[e]	S	9	9	-2
		5	M	8	7	-2
		13	L	7	7	-1
		19[e]	P	7	7	-3
	4	12	S	7	6	0
		6	M	7	7	0
		14	L	7	7	0
		20	P	5	6	-1
0	4	45[f]	L	0	0	0

Note: 1 mm = 0.039 in.

[a] S = small automobile; M = medium automobile; L = large automobile; P = pickup truck.
[b] No film coverage.
[c] No camera coverage.
[d] No camera coverage of driver.
[e] Three wheels dropped off.
[f] Control test.

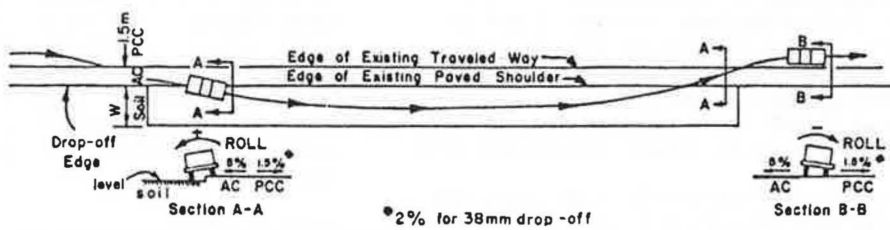
Table 4. Steering data—AC-to-soil drop-off test series.

Nominal Drop-Off Height (mm)	No. of Wheels Dropping Off	Test No.	Vehicle Size[b]	SWA[a]/Vehicle Velocities: SWA Off[c] (degrees)	Velocity Off (m/s)	Corrective SWA Off[d] (degrees)	SWA On (degrees)	Velocity On (m/s)	Corrective SWA On (degrees)
38	2	7	S	No film coverage					
		1	M	30R	26.8	60L	15L	26.8	8R
		17	L	23R	26.8	53L	38L	26.8	0
		23	P	30R	26.8	45L	38L	26.8	23R
	4	8	S	15R	29.1	30L	30L	29.1	23R
		2	M	38R	26.8	38L	30L	26.8	15R
		18	L	38R	26.8	53L	45L	26.8	23R
		24	P	45R	26.8	53L	38L	26.8	23R
89	2	10[e]	S	No film coverage					
		10[f]	S	30R	24.6	30L	23L	24.6	45R
		10	S	30R	26.8	30L	15L	26.8	45R
		4	M	38R	26.8	38L	30L	26.8	45R
		16	L	30R	24.6	45L	30L	24.6	45R
		22	P	60R	26.8	53L	45L	29.1	53R
	4	9	S	30R	26.8	30L	30L	26.8	45R
		3	M	30R	24.6	60L	60L	22.4	45R
		15	L	45R	29.1	60L	83L	26.8	60R
		21	P	75R	26.8	75L	68L	26.8	75R
114	2	11[g]	S	38R	26.8	45L	75R	24.6	30R
		5	M	45R	24.6	45L	45L	24.6	45R
		13	L	38R	26.8	53L	30L	26.8	-[i]
		19[g]	P	68R	26.8	83L	120R	24.6	45R
	4	12	S	30R	26.8	30L	23L	26.8	68R
		6	M	45R	24.6	30L	23L	24.6	30R
		14	L	45R	24.6	53L	30L	24.6	45R
		20	P	75R	26.8	68L	30L	26.8	75R
0	4	45[h]	L	45R	24.6	45L	38L	24.6	38R

Note: 1 mm = 0.039 in; 1 m/s = 2.24 mph.

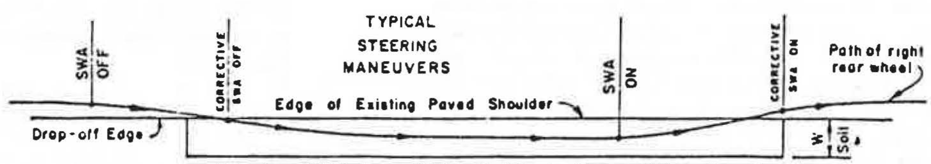
[a] Maximum degrees, reduced from high-speed film.
[b] S = small automobile; M = medium automobile; L = large automobile; P = pickup truck.
[c] R = clockwise rotation of steering wheel.
[d] L = counterclockwise rotation of steering wheel.
[e] No camera coverage.
[f] No camera coverage of driver.
[g] Three wheels dropped off pavement.
[h] Control test.
[i] No film coverage.

Figure 9. Vehicle roll angles—AC-to-soil drop-off test series.



Note: 1 m = 3.28 ft; 1 mm = 0.039 in; W = 3.7 m for 38-mm sites; and W = 5.5 m for 114-mm site.

Figure 10. Steering data—AC-to-soil drop-off test series.



Note: 1 m = 3.28 ft; 1 mm = 0.039 in; W = 3.7 m for 38-mm and 89-mm sites; and W = 5.5 m for 114-mm sites.

Coefficients of friction for the existing PCC traveled way, the existing AC shoulder, and the AC surface used for the AC-to-AC drop-off tests were measured along the three drop-off test strips with the California portable skid tester.

Average values for the coefficients of friction for the three paved surfaces were 0.42 for the PCC traveled way, 0.44 for the AC shoulder, and 0.39 for the AC surfaces used for the AC-to-AC test. These correspond to American Society for Testing and Materials (ASTM) skid numbers of 49, 51, and 47, respectively.

CONCLUSIONS

This paper does not attempt to define vehicle stability and controllability rigorously. For the purposes of this study, they were described as follows:

1. Stability—All of the mechanical systems and parts of the vehicle responded in a predictable, non-erratic manner and were undamaged. This is meant to imply that there was no skidding; no excessive rocking, rolling, or vibration; no deviation from the intended path of travel; and no loss of contact with the pavement.
2. Controllability—Steering did not require undue physical effort, excessive or tricky steering wheel input was unnecessary, and the drivers were not unduly bounced or thrown around in their seats.

The following specific observations and conclusions were reached as indicators of the stability and controllability of the test vehicles as they traveled over the drop-offs:

1. Steering—Relatively small steering wheel angles were measured during these maneuvers, usually 60° or less. The driver for these tests handled the steering wheel with minimal effort, which included control with

the thumb and forefinger only of both hands in some tests. At no time did the driver lose control of the steering wheel.

2. Vehicle roll—Vehicle roll angles did not increase significantly in relation to the height of the drop-offs. A maximum value of 10° was recorded, which is far from an impending rollover condition. The driver for these tests did not become disoriented or feel any discomfort during vehicle roll.

3. Noise—There is a significant jolt and accompanying noise associated with driving off and mounting drop-off heights of 89 mm (3.5 in) and 114 mm (4.5 in). The driver did not experience any noticeable disturbances during the 38-mm (1.5-in) drop-off tests.

4. Vehicle alignment—Front wheel alignment was not measurably affected during the drop-off tests.

5. Tire scuff—When the vehicles remounted the drop-off edge, the first vehicle wheel to contact the drop-off edge mounted each drop-off height without delay. Photographs of the tire scuff marks taken during the test series show that it takes less than one revolution of the first wheel contacting the edge of the drop-offs before the vehicle climbs back onto the pavement. Results were similar for two-wheel and four-wheel tests.

6. Wheel wobble—Varying amounts of front-wheel wobble occurred as the first vehicle wheel mounted the 89-mm and 114-mm drop-off heights. The major cause of wheel wobble (side-to-side motion) was the interaction of the sidewall of the tire with an irregular pavement drop-off edge. Wheel wobble did not affect the trajectory path of the vehicles during any of the tests.

7. Nonprofessional drivers—Although a professional conducted all of the tests documented on film for this project, two nonprofessional drivers stated they also did not experience any steering difficulties or stability problems while driving the three drop-off heights at about 17.9-20.1 m/s (40-45 mph). No data were taken from these tests, which were not part of the work plan.

8. No encroachment—During all of the tests, the drivers steered their vehicles back onto the pavement and back into their original 3.7-m (12-ft) lane of travel, nearest the shoulder, without encroaching into the other adjacent traffic lanes.

9. Three-wheel off tests—The events which came closest to causing any loss of vehicle control occurred during tests 11 and 19 (114-mm drop-off, AC to soil) when there was some rear wheel sideslipping and three wheels dropped off instead of the intended two. However, the driver was able to drive the vehicle back onto the roadway surface without losing control and without any abnormal difficulty. The lower coefficient of friction for the soil drop-off surface as compared to the AC drop-off surface made it easier for the vehicles to slip. Loose material on a shoulder should be considered a shoulder problem, not a drop-off problem. Vehicle roll angles for these tests (9° and 7°, respectively) were not excessive.

10. Curved roadway—The 38-mm (1.5-in) drop-off test strip was constructed on a 1525-m (5000-ft) radius curve to the left along the test site (Figure 3). The vehicles were not affected by this gradual curve during any of the two- or four-wheel drop-off tests conducted at this height.

11. Power steering—The medium- and large-sized vehicles used for this test series were equipped with power steering, and the small-sized vehicle and the pickup truck were equipped with manual steering. Even though steering torques were not measured during this test series, there were no trends in the test results to indicate that power steering affected vehicle stability in any of the tests.

12. Recent tests—Three tests involved a professional driver in a pickup truck traveling 26.8 m/s (60 mph). They were conducted in March 1978, to investigate vehicle stability and controllability while traversing a crumbling edge, 51-mm (2-in) high drop-off (nominal) on an AC shoulder next to a muddy shoulder. One test was a control test with no drop-off encountered; in the other two tests the two right wheels of the truck traversed the drop-off. It was concluded that there were no changes in the conclusions from the original series of 50 drop-off tests (6).

13. Summary statement—For the test conditions studied, the edge of pavement drop-offs per se did not throw the vehicles out of control or into an unstable condition or require any unusual control methods by the driver to get the automobile off and on the drop-off.

MAINTENANCE STANDARDS

Before setting overall drop-off standards or standards for specific sites, consideration should be given to variables not included in this project, such as vehicles in poor mechanical condition, driver inexperience or unpreparedness, adverse weather conditions, roadway and shoulder geometry, roadside obstructions, or hazards. Hence, the test results alone were insufficient to establish a maximum tolerable drop-off height for all conditions.

Based on the test conditions for this project, the 1974 California Department of Transportation maintenance standards concerning drop-offs were considered to be quite reasonable and conservative. Since 1974 the approach to maintaining the lateral support at the edge of pavement and shoulder maintenance has been changed somewhat in California, and no specific maximum allowable drop-off heights are included in the maintenance standards.

REFERENCES

1. J. L. Beaton, H. A. Peterson, and R. N. Field. Final Report of Full Scale Dynamic Tests of Bridge Curbs and Rails. Materials and Research Department, California Department of Transportation, Aug. 1957.
2. E. F. Nordlin and others. Dynamic Full Scale Impact Tests of Cable Type Median Barriers, Test Series IX. Materials and Research Department, California Department of Transportation, June 1965.
3. R. M. Olson and others. Effect of Curb Geometry and Location on Vehicle Bahavior. NCHRP, Rept. 150, 1974, 150 pp.
4. E. F. Nordlin and others. The Effect of Longitudinal Edge of Paved Surface Drop-Off on Vehicle Stability. California Department of Transportation, CA-DOT-TL-6783-1-76-22, March 1976.
5. Vehicle Dynamics Terminology. Society of Automotive Engineers, SAE-J670c, Nov. 1974.
6. R. L. Stoughton and others. The Effect of a Broken AC Pavement Drop-Off Edge and Muddy Shoulder on Vehicle Stability and Controllability. California Department of Transportation, Memorandum Rept. TL 656909, July 1978.

Notice: The Transportation Research Board does not endorse products or manufacturers. Trade and manufacturers' names appear in this paper because they are considered essential to its object.

Field Evaluation of Traffic Management Strategies for Maintenance Operations in Freeway Middle Lanes

Stephen H. Richards and Conrad L. Dudek, Texas Transportation Institute,
Texas A&M University, College Station, Texas

This paper presents results of field studies conducted in Houston, Texas, to evaluate the performance of two innovative approaches for managing traffic during maintenance operations in the middle lane of an urban freeway. The two approaches, traffic shifting with use of the shoulder and traffic splitting, were used by District 12 of the Texas State Department of Highways and Public Transportation during pavement repairs on Interstate 45. The results of the studies indicate that, compared to the multilane closure strategy commonly used at middle-lane work sites (closure of an exterior lane and one or more adjacent middle lanes), both approaches significantly increased work-zone capacity. The studies revealed that (a) traffic shifting could be used to manage traffic at relatively long work sites on freeways with discontinuous shoulders and (b) shoulder use at sites where this strategy was employed was greatly influenced by traffic demand. Traffic splitting around an isolated, middle-lane work site, on the other hand, was used effectively at a relatively short work site on a freeway section that did not have shoulders.

The Texas Transportation Institute is conducting research into traffic management at urban freeway work zones. The research is an attempt to develop more effective traffic-control systems for temporary work zones on urban freeways.

Project field studies have been conducted in Texas to evaluate standard and innovative practices associated with handling traffic through urban freeway work areas. The results of these studies indicate that managing traffic during maintenance activities in the middle lanes (on freeways with three or more lanes in each direction) is a difficult task and that problems often arise related to insufficient work-zone capacity. On the positive side, the studies have revealed innovative management strategies for handling traffic at middle-lane work sites that minimize the reduction in work-zone capacity and thus alleviate many of the inherent problems.

BACKGROUND

Maintenance work in the median lane or shoulder lane of an urban freeway is accommodated by closure of a single lane. Closure of either of these exterior lanes is relatively easy to achieve and, compared to more extensive management strategies (i.e., detours, crossovers, and multilane closures), this approach has minimum negative effects on freeway traffic operation.

The multilane closure strategy illustrated in Figure 1 is commonly used to accommodate work on the middle lane of an urban freeway. The multilane closure strategy involves closing an exterior lane and one or more adjacent middle lanes (1).

The major disadvantage of the multilane closure strategy presented in Figure 1 is the resulting loss of freeway capacity. The extent of the capacity reduction is illustrated by the data in the table below, which are based on studies made by Forbes and others on Los Angeles area freeways (2). The observed capacity data were collected on four-lane sections (in each direction) that were reduced to two lanes to accommodate maintenance work.

Type of Work	Observed Capacity (vehicles/h)
Median barrier or guardrail repair	3200
Pavement repair, mudjacking, or pavement grooving	3000
Resurfacing, slide removal, or striping	2600
Pavement marker installation	2400

From the table, note that capacity flow on four-lane freeway sections reduced to two lanes to accommodate maintenance work ranged between 2400 and 3200 vehicles/h (vph). These flow rates are 30.0-40.0 percent of the theoretical capacity (8000 vph) of a four-lane freeway section. In addition to the work done by Forbes and others, we have evaluated the capacity of three-lane freeway sections reduced to one-lane operation during resurfacing activities. The observed capacity of the one open lane was approximately 25 percent of the theoretical capacity (6000 vph) of a three-lane freeway section. Capacity flow in the open lane ranged between 1550 and 1700 vph.

INNOVATIVE STRATEGIES

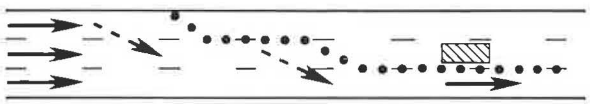
In recent years traffic on urban freeways in Texas has increased rapidly. On many facilities closing more than one lane to perform middle-lane maintenance work results in severe congestion, even during off-peak periods. The Texas State Department of Highways and Public Transportation is considering innovative traffic management strategies that increase capacity at middle-lane work sites, including two approaches used by District 12 during pavement repairs on a 4.0-km (2.5-mile) section of I-45 in Houston.

In April of 1978, state forces repaired potholes with asphaltic concrete in the northbound and southbound middle lane of a six-lane section of I-45 (three lanes in each direction). A work crew of approximately 10, along with several maintenance vehicles (i.e., asphalt trucks and a steel-wheeled roller), occupied the middle lane during work activities. As the work progressed, the location of the activity within the work zone moved slowly downstream at approximately 2 km/h (1 mph). The work took several days to complete.

Site Characteristics

The area of the repair work was a six-lane urban freeway facility with three 3.7-m (12-ft) lanes in each direction. It has 3.0-m (10-ft) outside shoulders and 2.4-m (8-ft) inside shoulders; however, the shoulders are discontinued at overpasses and bridge structures. Access

Figure 1. Multilane closure strategy used for middle-lane maintenance operations.



to and from the freeway is provided by right-hand slip ramps, which connect to parallel continuous frontage roads. In the study area, I-45 has an approximate average daily traffic (ADT) of 120 000.

Traffic Management Plan

The primary feature of the traffic management plan was the use of innovative approaches to increase work-zone capacity. To accommodate work on sections with shoulders, traffic was shifted out of the median and middle lanes and encouraged to use the shoulder lane and outside shoulder as travel lanes. To accommodate work on bridges and overpasses without shoulders, the middle lane was closed. Traffic was split around the middle-lane work area and motorists were permitted to travel in the median and shoulder lanes. The two strategies (traffic shifting with use of the shoulder and traffic splitting) were not used at the same time.

To reduce demand on the main traffic lane, entrance ramps were closed in the 4.0-km (2.5-mile) work area. Generally, two to four ramps were involved. Motorists who normally use these ramps had to remain on the frontage road and enter the freeway downstream of the work area. Frontage road signalization was not modified.

Traffic Control at Locations with Shoulders

Figure 2 shows the traffic-control devices used to manage traffic in the main lane during work on sections that have shoulders. All signs shown in the figure were temporary work-zone signs and had a black legend on an orange background.

The approach illustrated in Figure 2 made use of a typical multilane closure (presented in Figure 1) to remove traffic from the work-occupied middle lane. Motorists, however, were encouraged, by the use of special signs and cones, to use the outside shoulder as an additional travel lane. This management approach was fashioned after a similar approach developed by the California Division of Highways (2). Experience in California indicated that the strategy increased work-zone capacity significantly on a four-lane freeway section that has continuous shoulders, and motorists tended to use the shoulder only when congestion existed.

Comparison of the situation in the Houston studies to that studied in California reveals a major difference: The shoulders at the Houston sites were not continuous. Motorists on the shoulder were moved off the shoulder in the vicinity of bridge and overpass structures, then encouraged to use the shoulder again immediately downstream of the structures.

Traffic Control at Locations Without Shoulders

Figure 3 shows the traffic-control scheme used to manage traffic during work on sections of I-45 that do not have shoulders. All signs shown in the figure were temporary work-zone signs. They had a black legend on an orange background, except for the flashing arrow board and specially fabricated symbolic signs, which warned motorists of the traffic split. The symbolic traffic split signs had a black legend on a yellow background.

The management approach illustrated in Figure 3 is not a new concept, but it does have an innovative feature. Note in the site layout that cones were placed on the lane line between the middle and shoulder lanes to discourage lane weaving. These cones extended 150 m (492 ft) upstream of the taper closing the middle lane. Sight distance to the actual closure was approximately 400 m (1312 ft).

FIELD EVALUATION

Methods for evaluation of the innovative traffic management approaches used by District 12 were developed in response to the actual work performed. No attempt was made to control site variables or alter the work activities; therefore, the type and amount of data collected are somewhat limited. Nevertheless, sufficient data were collected to report significant findings.

Data were collected at work sites in the northbound and southbound middle lanes of I-45, where the shifting strategy was used. Manual traffic counts were made at several locations, and traffic operation was documented on videotape and 8-mm movies. Approximately 4 h of data were collected during a two-day study period.

Shoulder Use

Lane distribution data were collected during work activities. Figure 4 shows the location of the lane distribution count stations relative to the lane closures and shoulder-use signing. Station 1 was located downstream of the median-lane closure, near the first shoulder-use sign (CARS MAY USE SHOULDER 500 FT AHD). Station 2 was located approximately 150 m (492 ft) downstream of the point where use of the shoulder was first encouraged. The middle lane was still open to traffic at this point. Station 3 was located near the work activity and downstream of the start of the middle-lane closure.

Table 1 summarizes the lane distribution data. From the table, note that no drivers were observed traveling on the shoulder at station 1. Only a few drivers were observed using the shoulder (1.2-2.9 percent) at station 2. By the time drivers reached station 3, however, a significant number were using the shoulder. For example, during studies conducted when the flow rate through the work site was approximately 2400 vph, 38.1 percent of the traffic in the main lane was observed using the shoulder at station 3.

The increased shoulder use at station 3 is probably due to two factors: (a) The middle lane was closed at station 3, resulting in greater lane volumes and more congestion at this location compared to station 2; and (b) drivers had more time to read the signs and observe the action of other drivers by the time they reached station 3.

Influence of Shoulder Use on Capacity

Traffic counts of the main lane were made in the vicinity of the work activity to assess the ability of the strategy to increase work-zone capacity. Table 2 presents data on counts made during the period of heaviest observed flow. The data represent demand flows rather than capacity flows; demand never exceeded work-zone capacity. Nevertheless, the data in the table indicate that driver use of the shoulder as a travel lane increased work-site capacity well above capacities observed at similar sites where the multilane closure strategy (Figure 1) was used.

Note in the table that flow rates based on 5-min counts ranged from 2160-2616 vph. These flow rates represent a substantial increase over observed capacities (1550-1700 vph) at sites where the multilane closure strategy was used.

As reported in the California studies, the shifting strategy successfully increased work-zone capacity. In California the strategy permitted average hourly lane flows up to 1333 vph (total flow averaged over two available lanes and the shoulder). Lane flows up to 1308 vph were observed in the Houston studies (total flow averaged over one available lane and the shoulder).

Figure 5 illustrates the influence of traffic demand on shoulder use. Note from the figure that more drivers

Figure 2. Traffic-control strategy used during work on sections with shoulders.

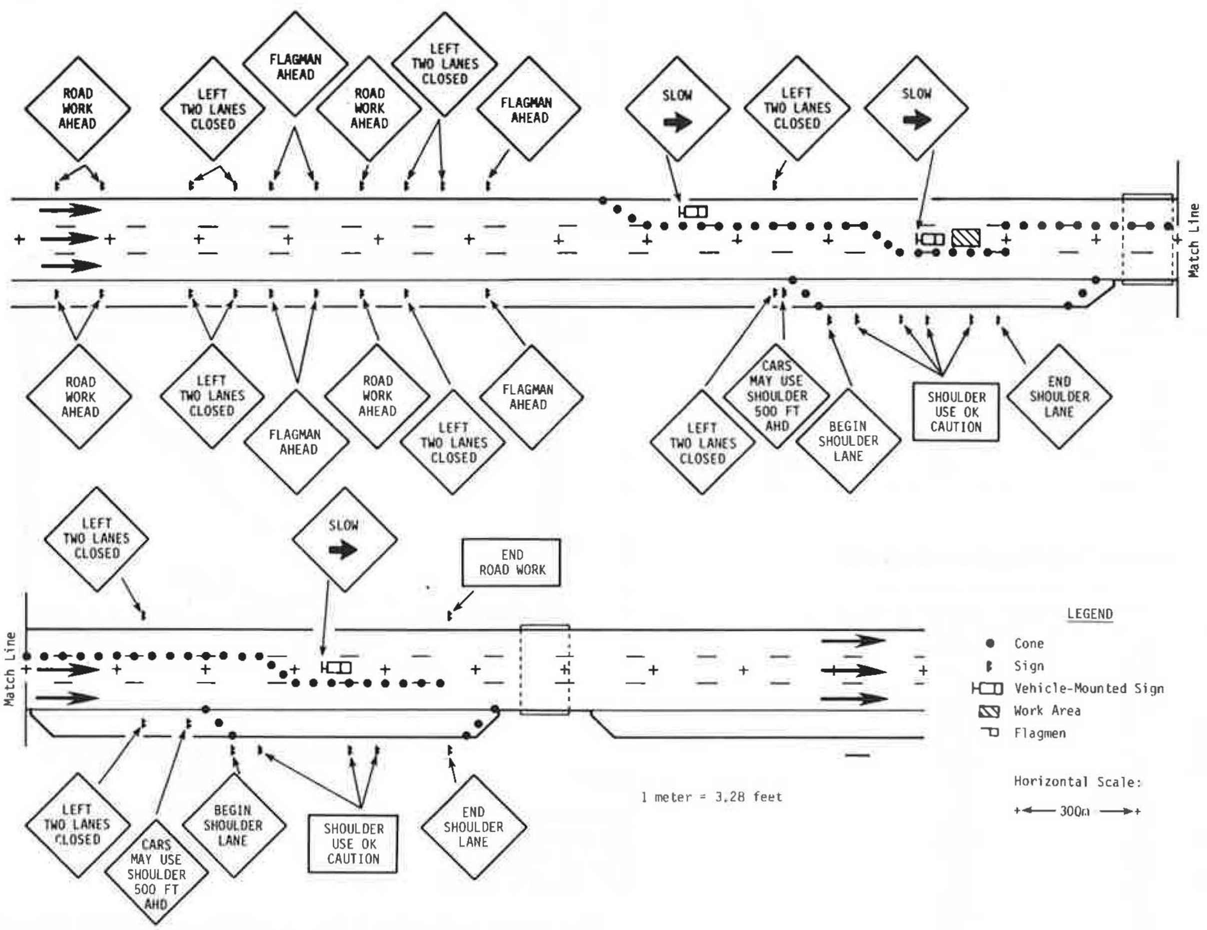


Figure 3. Traffic-control strategy used during work on sections without shoulders.

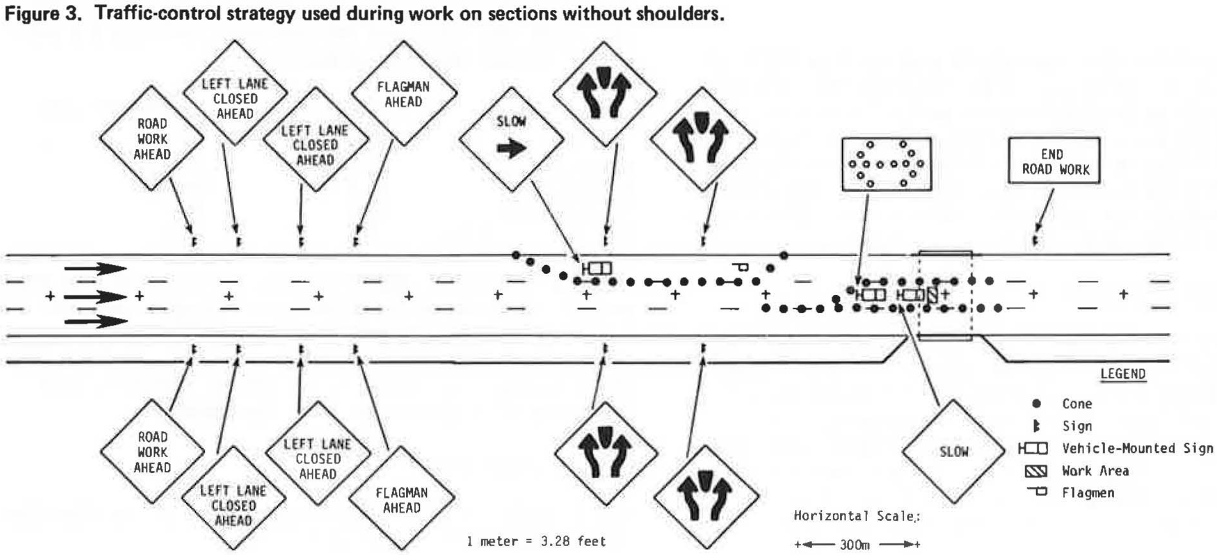


Figure 4. Lane distribution count stations—traffic shifting with use of the shoulder.

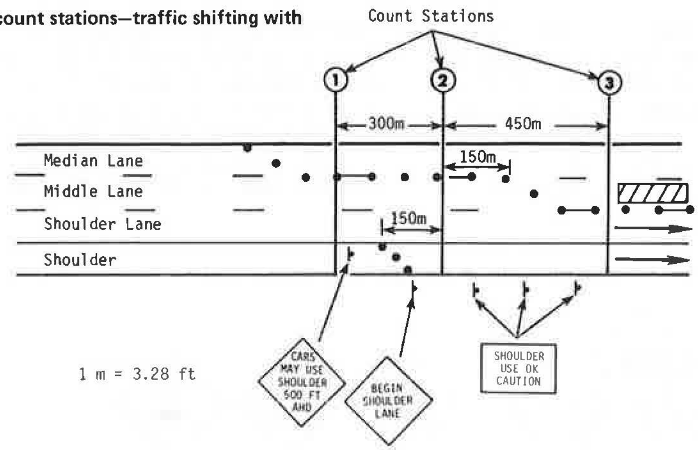


Table 1. Lane distribution data—traffic shifting to use the shoulder.

Flow Rate (vph)	Station	Study Time (min)	Total Traffic Middle Lane (%)	Total Traffic Shoulder Lane (%)	Total Traffic Shoulder (%)
1600	1	-	N/A	N/A	N/A
	2	10	20.6	78.2	1.2
	3	15	-	91.6	8.4
2400	1	70	42.1	57.9	0
	2	30	34.7	62.4	2.9
	3	75	-	61.9	38.1

Table 2. Volume data—traffic shifting to use the shoulder.

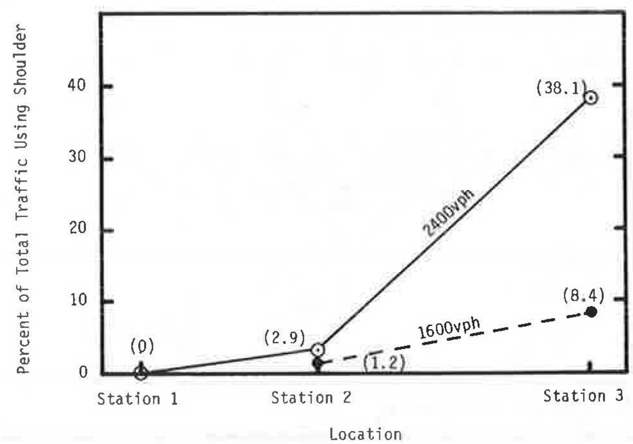
Time Period	Observed Volume	Hourly Flow Rate
11:00-11:05 a.m.	192	2304
11:05-11:10 a.m.	184	2208
11:10-11:15 a.m.	188	2256
11:15-11:20 a.m.	200	2400
11:20-11:25 a.m.	212	2544
11:25-11:30 a.m.	211	2532
12:15-12:20 p.m.	209	2508
12:20-12:25 p.m.	186	2232
12:25-12:30 p.m.	218	2616
12:30-12:35 p.m.	195	2340
12:35-12:40 p.m.	196	2352
12:40-12:45 p.m.	215	2580
12:45-12:50 p.m.	200	2400
12:50-12:55 p.m.	193	2316
12:55- 1:00 p.m.	180	2160
Average	199	2388

used the shoulder when the traffic demand was 2400 vph than when it was 1600 vph. This supports the finding of the California study that little or no traffic will use the shoulder until some degree of congestion develops. The difference in the shoulder-use rates at the two levels of volume (1600 vph and 2400 vph) was most pronounced at station 3, where the middle lane was closed. There was more congestion associated with this location.

Shoulder Use by Vehicle

The advance sign installed to encourage use of the shoulder read CARS MAY USE SHOULDER 500 FT AHEAD, which implied that only automobiles should use the shoulder. This message was selected to discourage truck traffic from using the narrow and structurally inadequate shoulder. The data presented in the tables below indicate that the subtle message did influence driver action; however, many trucks were observed on the shoulder.

Figure 5. Influence of traffic demand on shoulder use.



Vehicle	Shoulder Used (%)	Shoulder Not Used (%)
Automobile	9.3	90.7
Pickup and van	8.1	91.9
Truck and bus	2.6	97.4
All vehicles combined	8.4	91.6

The above table gives the percentage of traffic by vehicle type that used the shoulder when main-lane demand was approximately 1600 vph. From the table, 9.3 percent of all automobiles used the shoulder, 8.1 percent of pickup trucks and vans used the shoulder, but only 2.6 percent of trucks used the shoulder.

Vehicle	Shoulder Used (%)	Shoulder Not Used (%)
Automobile	40.2	59.8
Pickup and van	41.6	58.4
Truck and bus	25.3	74.7
All vehicles combined	39.1	60.9

This table presents the same information collected during periods when the demand increased to approximately 2400 vph. Under these higher-volume conditions, 40.2 percent of the automobiles used the shoulder, 41.6 percent of the pickup trucks and vans used the shoulder, and 25.3 percent of the trucks used the shoulder.

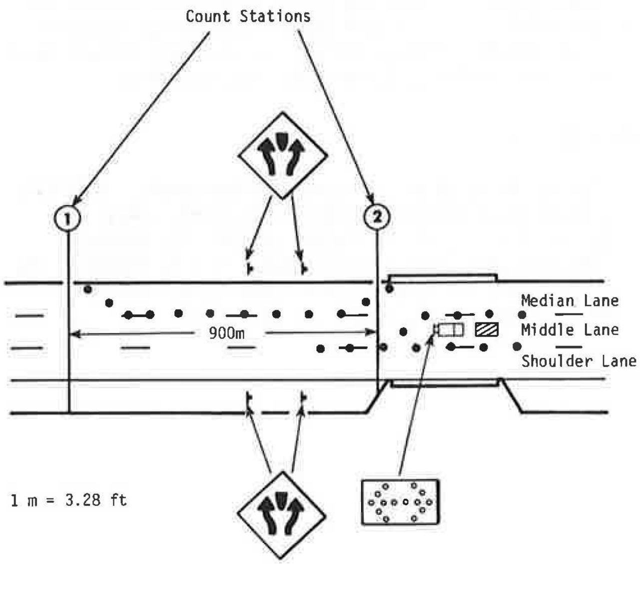
Other Observations

The following special observations were made about the performance of the strategy:

Table 3. Volume data—traffic splitting.

Time Period	Observed Volume	Hourly Flow Rate
10:35-10:40 a.m.	206	2472
10:40-10:45 a.m.	192	2304
10:45-10:50 a.m.	214	2568
10:50-10:55 a.m.	218	2616
10:55-11:00 a.m.	180	2160
11:00-11:05 a.m.	244	2928
Average	209	2508

Figure 6. Lane distribution count stations—traffic splitting.



1. Shoulder-use signing was not erected until after the median and middle lanes had been closed for 15 min. Therefore, for a brief time, all freeway traffic was funneled into the shoulder lane. Some queueing resulted, but the queue quickly dissipated after the shoulder-use signing was erected.

2. On the first day of operation, freeway demand was greatly reduced by closing several entrance ramps in the vicinity of the work. The result was extremely light flow on the freeway and severe congestion on the parallel frontage road. During this period, use of the shoulder was minimal and several vehicles drove across the outer separation to gain access to the freeway and avoid lengthy delays at signalized frontage-road intersections. State forces later took corrective action and reopened a high-volume entrance ramp.

3. The freeway cross-section included acceleration-deceleration lanes in the vicinity of ramps. Motorists attempted to form an additional travel lane at these locations, even though the added pavement width eventually was discontinued.

Traffic Splitting

The work schedule permitted data collection for a one-day period at a middle-lane work site on I-45 where the splitting strategy was employed. Manual traffic counts were made at two locations and traffic operation was documented on videotape and 8-mm movies during the one-day study.

Counts were made in the main traffic lane in the vicinity of the work activity to determine flow rates through the work site. A summary of the count data is presented in Table 3. The volume data in the table represent demand flows rather than capacity flows; however, field observations suggest that the work site was operating just below capacity. From the table, flow rates based on 5-min volume counts ranged from 2160 to 2928 vph. These flow rates represent a substantial increase over observed capacities (1550-1700 vph) at similar sites where the multilane closure strategy (Figure 1) was used. The average flow rate during a 30-min study period in which demand was highest was 2508 vph.

Figure 6 shows the location of the two-lane distribution count stations. Station 1 was approximately 150 m (492 ft) upstream of the median lane closure. Station 2 was at the beginning of the middle lane closure. The table below summarizes the lane distribution data. It shows that 59.8 percent (7.7 + 52.1 percent) of the traffic was in the median and middle lanes at station 1 and 40.2 percent was in the shoulder lane. The distribution did not change greatly at station 2, where 56.5 percent of the observed traffic was in the middle lane (the median lane was closed) and 43.5 percent of traffic was in the shoulder lane.

		Total Traffic		
Station	Study Time (min)	Median Lane (%)	Middle Lane (%)	Shoulder Lane (%)
1	30	7.7	52.1	40.2
2	30	-	56.5	43.5

The number and direction of lane changes that occurred in a section 300 m (984 ft) immediately upstream of the middle-lane closure were recorded. During the study period, only seven vehicles, or 1.5 percent of the total vehicles, changed lanes within this critical zone. Three of the seven lane changes were from the middle lane to the shoulder lane and four of the lane changes were from the shoulder lane to the middle lane.

Based on observed lane-change maneuvers and flow past the work site, the strategy appeared to provide an adequate level of safety to both motorists and work crew. This fact, combined with the increased work-zone capacity achieved, indicates that traffic splitting is a useful strategy for managing traffic at relatively short, middle-lane work sites where no shoulders exist.

During the installation and removal of middle-lane-closure devices (i.e., cones and arrow board), the three-lane freeway section was reduced to one-lane operation. Some congestion and motorist confusion resulted.

SUMMARY OF FINDINGS

Results of the field studies indicate that both approaches, traffic shifting with use of the shoulder and traffic splitting, increased work-zone capacity significantly. Specific findings associated with the approaches are enumerated below.

Traffic Shifting with Use of the Shoulder

1. Demand flows up to 2616 vph were accommodated without traffic queueing by use of this strategy on the three-lane study section. This flow rate represents a substantial increase over observed capacities (1550-1700 vph) at sites where the multilane closure strategy (Figure 1) was used. It also indicates that the strategy can be effective on freeway sections that have discontinuous shoulders.

2. More traffic used the shoulder at locations where only the shoulder lane and shoulder were open to traffic than at locations where the middle lane was also open.

3. During study periods when the demand flow was approximately 2400 vph, up to 38.1 percent of the amount of traffic in the main lane used the shoulder. At flow rates of 1600 vph, however, the percentage dropped to 8.4 percent.

4. The shoulder-use signing implied that only automobiles could use the shoulder. This implication may have caused drivers of trucks and buses to be more hesitant in the use of the shoulder than drivers of automobiles, pickups, and vans. Despite the reference only to automobiles, however, up to 25.3 percent of the trucks and buses used the shoulder.

Traffic Splitting

1. This strategy accommodated demand flow rates at the study site ranging from 2160-2928 vph. As flow rates approached the upper value (2928 vph), some queueing was observed, which indicates that capacity flow was approximately 3000 vph. This flow rate represents a substantial increase over observed capacities (1550-1700 vph) at sites where the multilane closure strategy (Figure 1) was used.

2. Lane distribution did not change significantly in the vicinity of the work site. Only 1.5 percent of the vehicles approaching the work zone changed lanes in a 300-m (984-ft) section immediately upstream of the taper that closed the middle lane.

3. Traffic cones were placed on the lane line between the middle lane and shoulder lane for a distance of 150 m (492 ft) upstream of the taper closing the middle lane. These cones appeared to be effective in reducing the number of sudden lane changes and other erratic maneuvers within this critical area.

ACKNOWLEDGMENT

The studies documented here were sponsored by the Texas State Department of Highways and Public Transportation in cooperation with the Federal Highway Administration. We would like to express our appreciation to the numerous individuals who made significant contributions to the conduct of the research reported here. The assistance provided by R. G. Biggs and Hunter Garrison of District 12, Texas State Department of Highways and Public Transportation, in planning and conducting the field studies is especially acknowledged.

REFERENCES

1. Traffic Control for Freeway Maintenance. NCHRP, Synthesis of Highway Practice 1, 1969, 47 pp.
2. C. E. Forbes, J. H. Smith, and W. E. Schaefer. Reducing Motorist Inconvenience Due to Maintenance Operations on High-Volume Freeways. HRB, Special Rept. 116, 1971, pp. 181-188.